Preface:

Sustainable development and its extensive domain affect every aspect of specialized activities including Civil Engineering with its vast and extensive scope. In this respect, protection of investments on concrete constructions on the one hand, and production and utilization of concrete materials in construction industry and its related issues like Analysis and Design, Durability, Strengthening, Maintenance, Quality Control, Environmental Considerations, Modern Constructional Technologies, Nano-Concrete, Concrete Constitutive Laws, on the other, have attracted the attention of researchers, planners and management authorities in various societies in considering concrete as an effective factor in the creation of sustainable development. Such characteristics have shaped a renowned, comprehensive and indispensable material from concrete, which plays an important role in various aspects of human life such as production, employment, environment and development in a rather significant manner. Behavioral complexities of this material, e.g. varying mechanical and physical properties exposed to diverse conditions, its behavior under different loading circumstances and development of properties like creep, shrinkage, dual action, bond between concrete and steel, slip, durability, corrosion etc, have all created a wide scope of research based on this material.

An in-depth knowledge and awareness of various scientific and technological developments on concrete and its related issues is conceivable only through an exchange of scientific and technical information by researchers and engineers across the globe. The Building & Housing Research Center, affiliated to the Ministry of Housing & Urban Development is honored to hold the 3rd International Conference on Concrete & Development, based on its functions and objectives. The Conference Secretariat at first stage received 195 Abstracts out of which 160 were accepted for preparation of full papers. At the second stage 143 Papers submitted by researchers from within the country and overseas, of which 101 (including 87 in English and 14 in Persian languages) were accepted by the Scientific Committee to be presented at Conference. The Conference CD-Proceeding totally includes 123 papers, out of which 17 Papers in Persian and 106 in English.

It is hoped that the proceedings will provide a fruitful background for upgrading the Civil Engineering branch, with an emphasis on concrete materials and structures through an appropriate exchange of information between all researchers, scientists and industrial scholars.

We would like to take this opportunity to thank our esteemed keynote speakers and distinguished authors, as well as members of the Reviewing Committee for their invaluable contribution to the proceedings. A special note of acknowledgement is also due to the staff and personnel of the BHRC who worked tirelessly in laying the groundwork and carrying out the numerous tasks involved in the organization of the Conference.

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A METHOD TO EVALUATE SEISMIC DEMANDS OF LOW-RISE RC BUILDINGS

Nanako Marubashi¹, Santiago Pujol², Toshikasu Ichinose³, Mogens P. Nielsen⁴
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ABSTRACT
A new methodology for design of low-rise RC building structures is proposed. This methodology uses base shear strength as the sole variable in the estimation of the maximum displacement demand. The formulation is applicable to ‘asymmetric’ ground motions exhibiting a large difference between the peak accelerations in opposite directions of motion, which result in larger inelastic displacement than symmetric ground motions. Conventional methods such as The Capacity Spectrum Method do not yield adequate results for such ‘asymmetric’ ground motions. The proposed method is effective if the period at the yield strength is smaller than 0.5 s, the base shear coefficient of the building is smaller than 0.4, and the ground motion is strong (PGA > 0.5g and PGV > 0.5 m/s).

Keywords: base shear strength, displacement demand, asymmetric ground motions

1. INTRODUCTION
The damage potential of ground motion is, arguably, estimated best in terms of the magnitude of the deformations induced on structures. In this article, we show results from analyses confirming that the correlation between an intensity measures commonly used to rank ground motion records, namely PGV, and displacement demand is poor. As a result, we concentrate on direct estimation of displacement demand, which is not only a good measure of damage potential but also the key parameter in the design and evaluation of building structures. Because there is much uncertainty in the problem, especially regarding the intensity of future ground motion, a simple method for estimation of displacement demand is preferable over a complex method. But simplicity ought not to compromise the quality of the answer. Several studies have proposed simplified procedures to estimate the maximum deformation of nonlinear single and multi-degree-of-freedom (SDOF and MDOF) systems. One of the most popular methods to estimate deformation demand of nonlinear structures was conceived by Freeman in 1975 [1]. It is known as The Capacity Spectrum Method and is used in design and evaluation provisions in the US (ATC-40, 1996 [2]) and Japan (Kuramoto, 2006 [3]). Although analytical studies have indicated the contrary, the available experience has not shown the results of the method to be inadequate. Its positive
record and its simplicity are the main reasons for its popularity among both
academicians and practitioners. In this paper we take another look at the results
obtained with the Capacity Spectrum Method (CSM). We do so motivated by the
recent abundance of available ground motion records. Our computations show that
the method is not sensitive to an attribute of strong ground motion that we have
coined “asymmetry” for lack of a better term. By asymmetry we refer to the
difference between the “positive” and “negative” peaks of a ground acceleration or
velocity record. We quantify it using the absolute value of the ratio of the positive
to the negative peaks. Our computations indicate that the response of nonlinear
SDOF systems with short periods of vibration (T<0.6s) is in fact sensitive to
asymmetry in the acceleration record.
In this article we propose a new design spectrum. The spectrum can be used to
capture the effect of record asymmetry on structural response. It differs from the
spectrum used in the CSM in that the one we propose refers to nonlinear response,
rather than linear response. And it differs from other spectra for nonlinear response
(Blume et al., 1961 [4]; Chopra 2006 [5]; Priestley 2007 [6]) in that we use base-
shear strength, not initial period or frequency, as the independent variable (as the
horizontal axis). We do not mean to suggest that response is not sensitive to initial
period. What we are suggesting is simply that, for short-period structures, response
is more sensitive to strength than to initial period.

2. METHODOLOGY
We study the potential effects of ground motion on structures by using SDOF
systems (Figure 1a). Three hysteresis rules are considered as depicted in Figures.
1b, 1c and 1d. Figure 1b shows the force-displacement response for an elasto-
plastic system, where \( Q_y \) is the yield force and \( K \) is the stiffness. The stiffness \( K \) is
calculated using Eq. 1,

\[
T = 2\pi \sqrt{\frac{m}{K}}
\]

(1)

where \( m \) is the mass and \( T \) is the fundamental period. The force-displacement
response of a rigid-plastic system is shown in Figure 1c. A rigid plastic system is
simply a particular case within the elasto-plastic systems in which \( K \) is assumed to
be infinite. We consider this case because the solution of the differential equation
of motion for a system with infinite stiffness is simpler than the solution for a
system with finite stiffness. Figure 1d shows the force-displacement response
assumed for an oscillator with decreasing stiffness. Stiffness variations are
computed using the formulation proposed by Takeda (1970) [7]. Fundamental
period (\( T \) in Eq. 1) is computed the secant stiffness \( K \) shown in Figure 1d. The
initial viscous damping factor is assumed 2% for the initial stiffness of each model.
The damping coefficient is assumed to reduce in proportion to the tangential
stiffness.
3. ASYMMETRY

The positive and negative peaks in ground acceleration and velocity records are different (Figure 2a, Erzincan 1992 record). The signs are a matter of convention. They are necessary but otherwise unimportant. We adopt the convention that accelerations and velocities are positive in the direction of the absolute acceleration or velocity maxima. The difference between the positive and negative peaks is usually ignored. To gain insight on the magnitude and the possible relevance of this difference, which we refer to as “asymmetry,” we studied 2715 horizontal ground motion records published by PEER (2000) [8]. Figure 2b shows the ratio of the peak ground acceleration \( a_1 \) to the maximum acceleration in the opposite direction \( a_2 \) plotted against \( a_1 \). Ten percent of the records with \( a_1 \) exceeding 6 m/s\(^2\) have a ratio \( a_1 / a_2 \) larger than 2.

![Figure 2. Erzincan 1992 record](image)

To understand the effects of the asymmetry in records on system response we start by considering the simplified periodic motions shown in Figure 4. We consider three idealized acceleration records with equal (absolute) peak ground acceleration \( a_i \): A1, A1.5, A2.5. The numbers following the letter A, which stands for “acceleration,” are ratios of peak accelerations \( a_i / a_2 \). Figure 2b indicates that ratios between 1 and 1.5 are common and a ratio of 2.5 is close to the upper bound for the records we have. Figures 4b and 4c show ground velocities and displacements computed by integrating the idealized signals A1, A1.5, and A2.5. Note that peak ground velocity and displacement decrease as we increase of the ratio \( a_i / a_2 \).
Figure 5 shows linear response spectra computed for the idealized periodic motions shown in Figure 3 for a damping ratio of 10%. The spectra indicate that relative displacement and absolute acceleration tend to decrease with increases in the ratio of peak accelerations. In other words, the spectra would lead us to think that the “asymmetry” in the records we are considering is not detrimental, and in fact, one could argue that the spectra show it is beneficial.
4. COMPUTED RESPONSE TO IDEALIZED MOTIONS

Figure 6a shows absolute peak relative displacements computed for synthetic motions A1, A1.5 and A2.5 and for elasto-plastic oscillators with $T = 0.2$ s with base shear coefficients $C_b$ ranging from 0 to 0.4. The computed relative displacement maxima are larger for the asymmetric records A1.5 and A2.5 than for record A1 for base shear coefficients exceeding approximately 0.1. Record A1, the symmetric record, has the same PGA, and larger PGV and response spectra (Figure 3b and 5a) than records A1.5 and A2.5. The peak ground velocity of record A2.5 is approximately half of the peak ground velocity of record A1, but the relative displacement computed for A2.5 is 1.5 times the displacement computed for A1 at $Cb = 0.1$. This observation indicates that there is no proportionality between peak ground velocity and displacement response for short-period structures. Elastic response spectra also fail to capture the effects of the asymmetry of the records considered (Figure 5). Estimates of displacement demand based on elastic spectra would, therefore, not include the effects of asymmetry.

Figure 6b shows the relationship between the base shear coefficient $C_b$ and the absolute maximum relative displacements computed for elasto-plastic oscillators with $T = 0.6$ s. The asymmetry of the record seems not to impact negatively the computed relative displacements. We conclude that response is sensitive to asymmetry in the acceleration record for $T < 0.6$ s.

On the other hand, response is not as sensitive to asymmetry in the velocity record as it is to asymmetry in the acceleration record.

5. COMPUTED RESPONSE TO RECORDED MOTIONS

In section 4 we considered the response of oscillators to idealized ground motion records. We now consider response to actual records. For illustration purposes we consider, initially, two records: 1992 Erzincan, Turkey, NS component, and 1992 Landers, Lucerne Station, component 275. Table 1 lists intensity measures for each record. Notice that the record for the Landers Earthquake has larger PGA, PGV and PGD. But, at the same time, the record for the Landers Earthquake has small
asymmetry while the record from Erzincan has large asymmetry as indicated by the ratios of PGA to maximum ground acceleration in the direction opposite to that of PGA. Figure 7 shows computed displacement peaks for elasto-plastic oscillators with $T = 0.2$ and $0.6$ s, and base shear coefficients ranging from 0 to 0.4. In all cases, the relative displacements computed for the record from Erzincan are larger than those for Landers. This trend is opposite to what would have been expected on the basis of the differences in PGA, PGV, PGD, and linear response spectra. This observation indicates that asymmetry is an important attribute in an acceleration record.

![Figure 7. Peak relative displacements computed for the Erzincan and Landers records](image)

6. HOW ASYMMETRY IN THE ACCELERATION RECORD AFFECTS DISPLACEMENT RESPONSE

We think of the response of a sliding block (Figure 8a: a physical representation of a rigid-plastic system) to a simple idealized ground acceleration record (Figure 8c). The record is asymmetric and periodic, and features acceleration “pulses” in two directions ($a_1 = 3$ and $a_2 = -1$, positive values represent movement to the left). We assume that the block will move relative to the ground if the frictional resistance in terms of acceleration reaches its strength ($\mu g = 2$ in Figure 8b). When the ground starts to move, with an acceleration of $a_2 = -1$, the block will simply move with the ground, and the frictional resistance is $a_2 = -1$ (Figure 8d). When the ground experiences the first “positive” acceleration pulse $a_1 = 3$ (point 1 in Figure 8c), the block will try to follow the ground. But its “strength” is limited ($\mu g = 2$ in Figure 8e). So there will be sliding. We can compute the relative acceleration simply by subtracting the acceleration of the block from the acceleration of the ground: $a_1 - \mu g = 3 - 2 = +1$ (Figure 8b, the positive sign indicates relative motion to the right). When the ground returns to an acceleration of $a_2 = -1$ (point 2), the relative acceleration becomes $a_2 - \mu g = (-1) - 2 = -3$. The relative acceleration reverses. If we integrate relative acceleration we obtain relative velocity (Figure 8f). And we observe that the relative velocity will increase during the first positive acceleration pulse (that is, between points 1 and 2) and will start to decrease when the ground
acceleration of the ground returns to \( a_2 = -1 \) and the relative acceleration changes in sign (point 2). Eventually the relative velocity returns to zero (point 3). At this point, the relative motion between block and ground ceases, and, therefore, the relative acceleration returns to zero. The acceleration of the block becomes equal to the acceleration of the ground \( (a_2 = -1) \) once again. The result of this process is a spike in the relative velocity record, which is zero elsewhere. This spike results in a permanent offset of the block with respect to its initial position \( (\Delta_1 \text{ in Figure 8g}) \). The magnitude of this offset is equal to the area under the spike in the relative velocity record. If a second positive acceleration pulse takes place (points 4 and 5 in Figure 8c), it will result in a second spike in the relative velocity and an additional offset (Figure 8g). The ground may go back to its original position (as in Figure 3c), but the block does not. The relative displacement \( (\Delta_1 \text{ in Figure 8g}) \) is maximum if the resistance is equal to \( a_2 (\mu g = |a_2|) \), as we observed in Figure 6a, unless the asymmetry is small \( (|a_1 / a_2| \approx 1) \). If the ground acceleration pulses did not have a preferred direction, the block would slide back and forth without accumulation of successive offsets. This analogy shows why asymmetry in the acceleration record may cause large relative displacements in inelastic systems.

7. THE STRENGTH SPECTRUM

We have shown that linear response spectra do not capture the effects of what we have called “asymmetry” in acceleration records. And we have shown that the effect of asymmetry can be significant for short-period structures \( (T < 0.6 \text{ s}) \). To solve this conundrum we propose the direct use of nonlinear spectra. And to ease the transition from linear spectra to nonlinear spectra we propose a simplification. Figure 9 shows computed displacement maxima for rigid plastic and elasto-plastic oscillators with periods ranging from 0 to 0.6 s and base shear coefficients from 0 to 0.6. The values shown were computed for the Erzincan and Landers records. From these four Figures it is apparent that, for \( T < 0.6 \text{ s} \), response is more sensitive to base shear strength than to initial period of vibration. We therefore suggest that, for design or evaluation purposes, it would be sufficient to work with spectra computed for oscillators with a fixed average period (say \( T = 0.2 \text{ s} \)) and varying base shear strength. We call such spectra (graphs showing maximum displacements of systems with \( T = 0.2 \text{ s} \) vs. base-shear coefficient) “Strength Spectra.”

In the following discussion, we refer to results obtained for the decreasing-stiffness system defined in Figure 1d with a base shear coefficient of 0.2. Those results were obtained for the 59 ground motion records with PGA > 0.5g and PGV > 0.5 m/s. Figure 10a compares the maximum displacements for systems with \( T = 0.2 \text{ s} \) with maximum displacements for systems with \( T = 0.1 \text{ s} \) and 0.5 s. The correlation coefficient is 0.97, indicating that the systems with \( T = 0.2 \text{ s} \) provide a good estimate of the maximum displacement of systems with different periods (not exceeding 0.5 s). As we increased the base shear coefficient up to 0.4, the correlation coefficient decreased but was larger than 0.8. In contrast, Figures 12b
and c show how intensity indicators, PGA and PGV, correlate with the computed displacement demand. It is clear that PGA is not a good measure of displacement demand. PGV is not very good, either. Note that the records in Table 2 include various ground motions with various dominant periods. We therefore conclude that the Strength Spectrum with $T = 0.2 \text{ s}$ can be used as an indicator of potential for seismic damage in low-rise buildings ($T < 0.5 \text{ s}$) regardless of the characteristics of the ground motion if the ground motion is strong (PGA > 0.5g and PGV > 0.5 m/s) and the base shear coefficient of the building is smaller than 0.4. The same observation is also true for elasto-plastic systems.

Figure 8. Response of a rigid-plastic oscillator to an idealized asymmetric record
Figure 9. Computed displacement maxima for rigid-plastic and elasto-plastic oscillators

Figure 10. Maximum displacement of decreasing-stiffness oscillators with 0.1 < T < 0.5 s (r: correlation coefficient)

Figure 11. Strength spectra for decreasing-stiffness SDOF systems with T<0.5s
Figure 13 shows maximum displacements of decreasing-stiffness oscillators with $T=0.2$ s and various base-shear strengths. The 59 records satisfying PGA $> 0.5g$ and PGV $> 0.5$ m/s were used again to generate this plot. The thick continuous line shows the tenth largest value of displacement response at each base shear coefficient considered. We consider that the data above this line are exceptionally large and should not govern the design. The thick broken line was obtained using the formulation proposed by Ozturk (2003) [9] for $T = 0.2$ s and PGV = 1.1 m/s, which is the mean plus one standard deviation for the peak ground velocities of the considered records. It is interesting that the line we computed matches reasonably well the one obtained using Ozturk’s formulation although Ozturk calibrated his formulation using records from Turkey exclusively while we have used records from a number of locations.

We suggest that the spectra in Figure 13, which have been computed for $T = 0.2$ s, can be used to estimate the displacement demand for building structures with different initial periods not exceeding 0.5 s. For $0.05 < C_b < 0.4$ and $T < 0.5$ s, we propose this envelope to be:

$$S_d = 0.6 - C_b$$  \hspace{1cm} (2)

Where $S_d$ is displacement demand for strong motion in meters and $C_b$ is base shear coefficient. This demand may be reduced according to the soil condition and the seismicity of the region; matters outside the scope of this manuscript.

8. CONCLUSION

(1) Asymmetry in ground acceleration -as defined here- can cause large inelastic displacements in low-rise RC buildings.
(2) Elastic spectra do not capture the effects of asymmetry.
(3) A Strength Spectrum (a graph showing maximum displacements of nonlinear SDOF systems with $T$ at yield = 0.2 s and various base shear coefficients) can be used as an indicator of potential for seismic damage in low-rise buildings ($T$ at yield $< 0.5$ s) if the base shear coefficient of the building is smaller than 0.4 and the ground motion is strong (PGA $> 0.5g$ and PGV $> 0.5$ m/s). The Strength Spectrum captures the effects of asymmetry in the ground motion.
(4) Strength Spectra can, therefore, be used as a new paradigm for design and evaluation of low-rise RC buildings ($T$ at yield $< 0.5$ s).

References
WORLD RECORD: THE LONGEST CONCRETE PLATE WITHOUT JOINTS, WITHOUT CRACKS

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ABSTRACT
The German electron synchrotron DESY (Deutsches Elektronen-Synchrotron) in Hamburg is a centre for research into the structure of matter. DESY uses the accelerator PETRA (Positron-Elektron-Tandem-Ring-Anlage) as an intense synchrotron radiation sources for a broad spectrum of tests with electromagnetic radiation. The tests must take place on an extremely stiff and smooth concrete plate. The plate had to be 280 m long, 24 m wide and 1 metre thick. The plate should not get any joints and has to remain free of cracks because of the resistance demanded. The plate has to be produced monolithically, that is on behalf of the huge dimensions and properties world record. The positioning of the plate on a sliding bitumen layer and the use of steel fibre concrete have been the prerequisites for a successful performance under such challenging requirements.

1. INTRODUCTION
The German electron synchrotron DESY in the Helmholtz-Community in Hamburg, Germany, is one of the leading accelerator centres in the world. The accelerator PETRA is being improved to be the most brilliant worldwide storage ring-based X-ray radiation sources worldwide. This new light source of the superlative offers excellent conditions for top research with particularly intensive and sharp joint X-ray radiation. The decisive advantage is the capillary X-ray radiation of an especially high brilliance: Also, tiny tested materials can be examined and their pictures highly resolved in the order of their atoms. Almost 300 m of the 2.3 km long Petra ring must be completely modified and a new experimentation hall had to be constructed. The basis of the hall is a 1 m thick concrete slab which carries the accelerator and the experiments. The slab is 24 m wide and 280 m long. It follows the circular shape of the accelerator ring. The concrete slab protects the highly precise measurement equipment from mechanical vibrations. The slab is decoupled from the building. It had to be built without joints and cracks. Therefore the slab had to be concreted without interruption. The evenness of the final floor slab was better than 4 mm/10 m. At a concentrated load of 1kN the floor slab may deform vertically only by 1 µm. The unusually high requirements called for correct planning, faultless production, highly sophisticated quality assurance, and an experienced contractor.
2. PHILOSOPHY OF THE ENGINEERING DESIGN

The dimensioning of the floor slab was planned with the following considerations: A minimal deformation hindrance has been gained by applying a bituminous sliding layer despite not knowing initially the necessary minimum thickness. The slab starts shortening by cooling off after its maximum temperature gained by hydration heat. As a consequence it has to suffer a maximum tensile stress in its central cross section through each of the approximately 150 m long ends. If these tensile stresses remain below the tensile strength of the concrete, an additional mild reinforcement could be saved. The shortening of the slab has been expected to start three days after the beginning of concreting. For this circumstance a concrete mix had to be designed which produced a hydration temperature as low as possible while reaching a tensile strength of at least 2 N/mm² as early as possible. The mixture had to be proven in internal performance tests by the construction firm. Because not all conditions and may be impacts could be foreseen for this special construction task nevertheless no crack has been allowed leading to a gaping joint in the concrete slab. Concerning these requirements the planned tensile strength of the young concrete had to be achieved for certain. The achievable tensile strength can only be guaranteed when steel fibres are mixed into the concrete. Steel fibres do not increase the tensile strength of the concrete, however, they guarantee it. This guarantee is reached only by a fibre content of approximately one volume per cent. For this reason we suggested 75 kg of steel fibres per cubic metre of concrete.

Micro cracks arise during the hydration process in the very young concrete. Late cracks arise by shrinkage at a higher age, by restraint and internal tensions. Micro cracks caused by autogenous shrinkage or by tensile stresses through hydration heat act later as crack starters when the plate is subjected to stresses by external loads. These fine micro cracks do not further enlarge if they are held small and locked by well bonding scraped fibres. To cope with the shrinkage crack formation in the very early age we chose scraped fibres with the best bonding performance in young concrete. The amount was 40 kg scraped fibres per cubic metre of concrete. For cracks arising later we chose crack bridging fibres with double bended ends, a
length of 50 mm, diameter 0.8 mm, and 35 kg per cubic metre of concrete. Cracks have to be avoided primarily in the surface. Near the surface of a concrete the influences are more various and more intense than inside the structure. Therefore steel fibre concrete has been planned for the top 50 centimetres, for the upper half of the slab. The lower area got conventionally reinforced with a single reinforcing layer calculated for preventing crack widths bigger than 0.3 mm. (Figure 2.)

![Figure 2. Layers of the floor slab](image)

The centre of the 300 m long concrete slab was designed as a fixed symmetry centre. A 1 m deep trench was executed monolithically in the central section of the slab. Concreting had to be started from here in the two directions simultaneously.

![Figure 3. Test setup at the TU Berlin to check the sliding resistance of the bituminous sliding layer](image)
3. SOLUTION METHOD

Long building structures traditionally get subdivided into sections by expansion joints. Because of the high requirements for the bending stiffness of the slab it had to be executed without joints. For a floor slab of such a huge length deformations are adding up in the decimetre range by loss of hydration heat and shrinkage. The foundation basement of the slab leads to a resistance against shortenings. Forces by friction or adhesion are caused in the plate. The not avoidable cracks normally get limited in width by a crack distributing reinforcement. As the requirements for this floor slab were extremely high this would have required an exceptionally high amount of steel reinforcement with a time consuming forming and placing. A special cracking risk is gained by the hydration heat of the cement. Maximum temperatures of more than 50°C were expected. During cooling the concrete contracts and will crack if its deformation is hindered. To not hinder the deformation, we proposed to place the slab on a 3 mm thick sliding bituminous layer. Laboratory tests with a 1.5 mm thick bituminous layer proved the sliding performance to be not sufficient. A temperature difference of only 10 K between surface and centre of the concrete slab theoretically leads to a temperature related crack in the surface. Water loss by drying of unprotected concrete yields to a shortening by shrinkage of about 0.4mm/m. Therefore the concrete slab had to be covered against drying after smoothing with a foil and had to be protected against cooling with a mat of foam with a chosen thickness of 10mm.

To optimize the slab construction not only technically but also economically we proposed to construct the slab in two fresh concrete layers. The 49 cm thick upper layer has been made with steel fibre reinforced concrete with 1 Vol% fibre content with the concrete strength class C 30/37 (F1,6
XC1XM1 D\text{max}=16) according to EN 206-1 „Concrete – Part 1: Specification, performance, production and conformity“. The 50 cm thick bottom layer was made with reinforced concrete strength class C 30/37 (XC1 D\text{max}=32) according to EN 206-1 with a weak reinforcement for crack width smaller than 0.3 mm at the bottom according to DIN 1045-1 „Concrete, reinforced and prestressed concrete structures – Part 1: Design and construction“. Each concrete layer had to be cast in place while proceeding into both directions without interruption and without joints in one sequence. The top layer with steel fibre concrete was cast in place nearly 25 metres behind the front of the bottom layer.

In the steel fibre concrete an electromagnetic compatibility shielding (EMC Shielding) had to be installed with steel mats in a depth of 15 cm under the top edge of the floor slab. A 1 cm thick layer for wear made of epoxy resin produced the finish of the slab.

4. PRELIMINARY TESTS
4.1. Tests with Sliding Bitumen Layer
The bitumen sliding layer had to be executed as thick as necessary but as thin as possible so that no surplus bitumen would flow out at the sides of the floor slab. For the regulation of the different sliding resistance of differently thick bitumen layers corresponding tests were carried out at different temperatures at the TU Berlin.

The result of the experiments was a friction coefficient µ_R of 0.1 at 20°C with a 3 mm thick sliding layer made of bitumen 50/70 to EN 12591 "Bitumen and bituminous binders – Specification for paving grade bitumen". For a 1 mm thick layer a sliding resistance of µ_R > 0.4 was found.

On the construction site a bitumen sliding layer with an amount of 3 litres of bitumen per square meter got mechanically sprayed on the sub-concrete. This volume produced a nearly 3 mm thick layer. The extraordinary smoothness of the layer is shown in Figure 5.

![Figure 5. The smoothness of the bitumen layer on top of the sub-concrete](image-url)
For the bitumen sliding layer a friction coefficient $\mu_R=0.35$ was set for the further static calculations.

4.2. Mix Design
In addition to constructive measures concrete technological parameters had to be assumed in order to reduce the crack width provoked by heat of hydration or by long term shrinkage. The high steel fibre content of 1 V-% in the upper part of the floor slab was an indispensable component of the quality concept. The slab should remain crack free in its early age and for its lifetime.

The concrete slab should be able to suffer shortening deformations three days after production. Consequently cement had to be chosen which builds up its tensile strength fast. In addition, the steel fibres had to be intensively connected to the hardened cement paste. Therefore minimum binder content is required. This demand stands contrary to the reduction of hydration heat. However, a rapid strength development is more important than a slightly elevated hydration temperature. A mix of CEM III A and fly ash was recommended.

For the assessment of the longitudinal deformations of the slab, the time dependent temperature development of the concrete was examined in polystyrene cubes under adiabatic conditions. The target consistency was 50cm spread on a flow table according to EN 12350-5 “Testing fresh concrete–Part 5: Flow table test”. Preliminary tests were performed by the contractor Züblin for different concrete mixtures. The results of the selected mixture are shown in Table 1. Table 2 shows the main characteristic values of the selected mix.

The measurements of the temperature development led us to expect a temperature rise of about 40 K in the centre of the floor slab.

Table 1: Mixture

<table>
<thead>
<tr>
<th>C 30/37</th>
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</thead>
<tbody>
<tr>
<td>CEM III/A 42.5 N 340 kg/m$^3$</td>
</tr>
<tr>
<td>Fly ash 112 kg/m$^3$</td>
</tr>
<tr>
<td>Maximum grain size 16 mm</td>
</tr>
<tr>
<td>$w/(c +0,4 f) = 0.43$</td>
</tr>
<tr>
<td>Fibre mix 35 kg/m$^3$ DE 50/0,8 (Harex)</td>
</tr>
<tr>
<td>40 kg/m$^3$ SF 01-32 (Harex)</td>
</tr>
</tbody>
</table>

To make the concrete pumpable, the content of ultra fines had to be sufficiently high. The components below 0.25 mm, (cement, fly ash, fine sand) should therefore be at least 400 kg/m$^3$. In preliminary tests the pumpability was proven also for the relatively high steel fibre content. The pumping of such a concrete is representing a special challenge.
Table 2: Test results (main characteristic values).

<table>
<thead>
<tr>
<th></th>
<th>Compressive strength $f_{c,\text{cube}}$ in N/mm²</th>
<th>Young's modulus in N/mm²</th>
<th>Fibre concrete class</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 d</td>
<td>7.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2 d</td>
<td>14.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3 d</td>
<td></td>
<td></td>
<td>1,0/0,6</td>
</tr>
<tr>
<td>7 d</td>
<td>33.0</td>
<td>28 000</td>
<td></td>
</tr>
<tr>
<td>28 d</td>
<td>52.0</td>
<td>33 900</td>
<td>1,6/1,0</td>
</tr>
</tbody>
</table>

5. EXECUTION

5.1. Quality Assurance

Before the beginning of the challenging concreting task the companies involved trained their employees for the special site requirements. The concreting concept and the quality system of the building contractor regulated the quality control during the execution. The quality was monitored additionally beyond this at particularly relevant stages upon request of the owner-builder. Controls of the preparations and equipment in the mixing plants before the beginning of the work as well as spot tests during the addition of the steel fibres were included.

Each mixing truck was checked on the construction site once again. Rejection criteria for concrete trucks were fresh concrete temperatures falling below 10°C and a spread on a flow table of more than 55 cm. Special emphasis was put on the compaction of the concrete embedding the EMC mats and on curing measures.

5.2. Concrete Production

The concrete supplier Holcim had to guarantee delivery of 160 m³ per hour for the four concrete pumps. Holcim used four ready-mix concrete factories and one alternative factory with a corresponding stockpiling of the base materials. Twenty eight ready-mix trucks of 8m³ each were used. An interruption of the concreting process was not foreseen and was not allowed under any circumstances.

The concrete factories put special emphasis on a high regularity of concrete production. Coping with the moisture content of the sand and the medium-sized aggregates was important. The target value of the spread at delivery was 52.5 cm in order to not fall below the demanded consistency of 50 cm on the site.

The steel fibres were weighed and pre-packaged beforehand. The scraped fibres SF 01-32 were added to the empty transport vehicle by an elevator. After this the concrete was loaded. The steel fibres DE 50/08 were blown with an air stream device into the loaded ready-mix vehicle (Figure 6). A prolonged mixing time for the steel fibre concrete had to be taken into account in the delivery concept.

Vehicles with steel fibre concrete were indicated by a green arrow in the
windshield to avoid false placement. The site foreman checked every vehicle again at the pump for its consistency by visual inspection.

![Figure 6. Blow in of the steel fibres into the mixing vehicle](image)

5.3. Placing the Concrete
The walls and the roof of the hall were completed except for the end walls before placing the concrete. Four concrete pumps and one reserve pump were used. Two pumps concreted the lower layer with normal reinforced concrete; the other two pumps concreted the steel fibre concrete for the top layer of the slab 20 m behind the leading edge of the lower layer. The concrete was pumped at the same time at both sides starting in the middle of the slab. The 7000 m³ of concrete were placed within approx. 60 hours. The compaction of the concrete was carried by immersion vibrators. The surface was levelled with a special levelling plank. After reaching the required degree of hardness the concrete surface was smoothed with a double slab machine. As soon as the concrete was sufficiently set, it was covered with foil and a 10 mm thick insulating mat. The curing time was 31 days. The concrete was left in the side formwork for the complete curing time.

6. RESULTS
Immediately after the completion of work the monitoring of the strength and deformation behaviour of the slab began. The floor slab behaved as calculated. The ends were shortened with a speed of 30 - 40 nm/s after three days. Each end shortened by about 40mm.
The temperature from hydration heat and heat conduction was recorded continuously starting at the beginning of concreting in different places in the following four heights:

- Sliding layer ± 0 cm (bottom contact zone)
- Sliding layer + 10 cm
- Sliding layer + 50 cm (slab centre)
- Sliding layer + 90 cm (10 cm below slab surface)

Compared with the values determined in the laboratory the temperature rise inside the concrete slab increased at most 32 K. The reason was the low fresh concrete
temperature and the cool weather during the execution. Figure 9 shows the typical temperature developments at different measuring points in one cross-section.

![Figure 9. Temperature development in the concrete slab](image)

After the length deformation stopped the floor slab was measured in a grid of 2.0m×2.0m. Such a high level of evenness was achieved that a planned additional levelling screed could most likely be foregone. Nevertheless the epoxy-layer has been performed.

![Figure 10. The glass like surface of the finished concrete slab](image)

Partners:
Züblin AG, Direction North, Hamburg
Holcim Concrete and Aggregates GmbH, Hamburg
GuD Consult, Berlin
IFDB, Berlin
REFERENCES
SEISMIC UPGRADE OF CONCRETE COLUMNS

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ABSTRACT

This paper presents a summary of the work from an extensive research program underway at the University of Toronto on strengthening deficient and repairing damaged concrete columns with fibre reinforced polymer (FRP) jackets. The specimens consisted of either a 305 mm square or 356 mm diameter and 1.47 m long column connected to a 508 × 762 × 813 mm stub. Each specimen was tested under lateral cyclic displacement excursions and simultaneous constant axial load to simulate seismic forces. Results indicate that added confinement with FRP at plastic hinge locations significantly enhanced ductility, energy dissipation capacity and strength of columns. Efficiency of FRP confinement was much superior in circular columns than in square columns. A procedure is presented for the design of confining FRP reinforcement in columns required to achieve a certain ductility performance given the applied axial load and the properties of the FRP.

Keywords: columns, confinement, ductility, earthquake, energy dissipation, retrofitting.

1. INTRODUCTION

Large inelastic deformation capacities of individual members allow entire structures to resist severe ground motion while dissipating significant levels of seismic energy. To ensure overall structural integrity, plastic hinge formation associated with lateral displacement excursions is preferred in beams and girders rather than in columns [1, 2]. However, plastic hinge development can occur in columns, particularly at the bases of multi-storey frames and bridges. Ductile behavior is essential at these crucial sites to prevent complete structural collapse under sustained loading. Many of the existing reinforced concrete structures do not have adequate amount of confinement reinforcement in potential plastic hinge regions of the columns and may result in brittle structural response during earthquakes. Destruction from the 1994 Northridge, 1995 Kobe and 2005 Kashmir earthquakes [3, 4, 5] has highlighted the worldwide vulnerability of reinforced concrete columns exposed to inelastic conditions. To provide additional confinement to these deficient columns, retrofitting with a fiber-reinforced polymer (FRP) jackets provides a very attractive solution due to their lightweight, high strength, and excellent corrosion resisting capabilities. The current design code provisions [1, 2] require large amounts of steel reinforcement placed at small spacing in critical regions of columns, which, quite often, makes construction very
cumbersome and at times impractical. Use of external FRP shells with fibers aligned in the circumferential direction of the column can provide confinement and a stay-in-place formwork for new structures [6]. Many experimental studies [7–14] have demonstrated that the confinement provided by the FRP wraps can significantly increase the energy absorption capacity and ductility of the columns under combined axial, flexural and shear loads, thereby increasing their seismic resistance. This paper presents selected results from an extensive experimental program in which similar large size square and circular columns were tested under simulated seismic load in exactly the same manner to provide comparable results to investigate different variables and design parameters. A design procedure is also presented which can be used to calculate the amount of confining FRP required for a certain ductility performance given the axial load on the column and the properties of the FRP. Initial focus of this research program was on square columns due to the fact that it presented a more challenging scenario due to inferior confinement efficiency in these columns compared with circular columns. A large number of well-instrumented square columns were tested in a similar manner under simulated earthquake loads and extensive data was used to develop analytical models and design procedures. Experimental work on circular columns has recently been completed on similar lines and analytical work is in progress. Results indicate that the amount of FRP reinforcement required in circular columns is about half of what is required in square columns for similar improvement in ductility.

2. EXPERIMENTAL RESULTS AND DUCTILITY PARAMETERS
To develop a procedure for the design of confining FRP for concrete columns, an extensive review of the available test results was conducted. This review indicated that the results available were seriously limited. Although there exists a consensus that confining FRP can significantly enhance the seismic performance and ductility of circular, square or rectangular concrete columns, the test setups, loading histories, instrumentations, specimen details and ductility parameters used in the available experimental investigations were different from one program to another, making it difficult to evaluate these results on a common platform.
A test program was thus undertaken in which all the column specimens were similar and tested in exactly the same manner. Each specimen was comprised of a 305×305×1473 mm or 356 mm circular column connected to a 508×762×813 mm stub. The column part of the specimens was 1.47 m long. The corners of all square columns were rounded using concave wood sections, with a 16 mm radius, placed inside the forms during casting to facilitate FRP wrapping. The columns were characteristic of field members located in multi-storey building frames or in bridges between the points of maximum moment and contraflexure. All square specimens contained eight 20M longitudinal bars ($\rho_l = 2.58 \%$) uniformly distributed around the column core creating a core area that was 77 % of the gross column area. Perimeter hoops laterally supported the four corner bars and internal hoops enclosed the four middle bars. The circular columns contained six 25M longitudinal bars ($\rho_l = 3.01 \%$) and the ratio between the core area and the gross
column area was 74%.

Each specimen was tested horizontally in the loading frame shown in Figure 1 under a constant axial load and applied lateral cyclic displacement excursions simulating earthquake forces. The specimens were subjected to transverse displacement excursions (Figure 2) using a displacement-control mode of loading until the specimen was unable to sustain the applied axial load. The retrofitted specimens were externally confined by different amounts of continuous CFRP or GFRP wraps. Table 1 lists the details of the square specimens considered in this study, while the test data of circular specimens is presented in Table 2. Throughout this experimental program the same types of CFRP and GFRP materials were used. The FRP properties varied only slightly from one batch to the other. The ultimate tensile strength and rupture strain of the CFRP fabric ranged from 912 to 962 N/mm width per layer and 0.0123 to 0.0142, respectively, while these values for the GFRP fabric were 518 to 647 N/mm width per layer and 0.0197 to 0.0228, respectively.

![Figure 1. Schematic test setup](image1)

![Figure 2. Lateral displacement excursions](image2)

![Figure 3. Ductility parameters](image3)
2.1. Ductility Parameters

The design approach was developed on the pattern of the procedure for the design of steel confining reinforcement [15]. The parameters used for evaluating the ductile performance of a column included section and member ductility, energy dissipation capacity and the number of standard displacement excursions a column could sustain before failure. In evaluating the seismic performance of the columns and studying the effects of different variables, ductility and toughness parameters defined in Figure 3 were used [15]. These included curvature ductility factor $\mu_{80}$, cumulative ductility ratio $N_{80}$, and energy-damage indicator $E$. Subscripts t and 80 indicate, respectively, the value of the parameter until the end of the test (total value) and the value until the end of the cycle in which the moment has dropped to 80 percent of the maximum value. The energy parameter $e_i$ represents the area enclosed in cycle $i$ by the M-\(\Phi\) loop. Terms $L_f$ and $h$ represent the length of the most damaged region measured from the test and the depth of the column section, respectively. All other terms are defined in Figure 3. Table 1 and Table 2 list these ductility and toughness parameters for the columns considered in this analysis.

Sheikh and Khoury [15] observed that different ductility and toughness parameters were interrelated (Figure 4). In columns internally confined with steel, for $\mu_{80}$ of 16, the values for $N_{80}$ and $E_{80}$ were found to be 64 and 575, respectively. A column with this level of deformability was defined as highly ductile. The section with a $\mu_{80}$ value of 8 to 16 was defined as moderately ductile and the low ductility column had $\mu_{80} < 8$. Figure 4 also shows the relationships between different ductility parameters of FRP-confined columns. Data from nine steel-confined columns reported by Sheikh and Khoury [16] and ten FRP-confined columns as listed in Table 1 are used to construct the figure.

It is clear from Figure 4, that up to $\mu_{80}$ of approximately 8, the relationships between various parameters for steel-confined and FRP-confined columns are very similar. For values of $\mu_{80}$ larger than 8, the parameters $N_{80}$ and $E_{80}$ of the FRP-confined columns are significantly higher than those of the steel-confined columns with similar $\mu_{80}$ values. For dissipating equal amount of energy, the FRP-confined
columns require smaller curvature ductility factors than comparable steel-confined columns. This can be attributed to the different curvature distributions in the plastic hinge region and different plastic hinge lengths in these two types of columns. The FRP-confined columns require a curvature ductility factor of only 13.2 to dissipate the amount of energy as the steel-confined columns dissipate at $\mu_{d80}$ of 16. Similar trend is observed for cumulative curvature ductility ratio. The behavior of a FRP-confined column with $\mu_{d80} = 13$ can thus be considered as highly ductile. The section with a $\mu_{d80}$ value of 8 to 13 is defined as moderately ductile and the low ductility column has $\mu_{d80} < 8$.

2.2. Effects of Different Variables on Column Performance

Based on the experimental results listed in Table 1 and Table 2, the most important variables identified to affect a column’s ductility are the amount of FRP confining reinforcement, type of FRP and the level of axial load. The effect of these variables on column behavior is discussed in the following.

Amount of confining FRP – The effect of the amount of confining CFRP can be evaluated by comparing the moment vs. curvature behavior of two sets of columns, as presented in Figure 5. The first set (Figure 5a) includes square columns AS-1NS, ASC-2NS and ASC-6NS that were tested under an axial load of 0.33 $P_o$. All the columns in the second set (Figure 5b), AS-1NSS, ASC-4NS, ASC-3NS, and ASC-5NS, were tested under an axial load of 0.56 $P_o$. The ductility parameters in Table 1 and the responses shown in Figure 5 clearly demonstrate the enhanced cyclic performance of the FRP-retrofitted columns. The behavior of columns improved progressively as the amount of confining CFRP increased. While Specimen AS-1NS tested under axial load of 0.33 $P_o$ failed following the 7th cycle, Specimen ASC-2NS and ASC-6NS confined by one and two layers of CFRP, respectively endured 15 and 20 load cycles, respectively. The columns tested under higher axial load, ASC-4NS, ASC-3NS, and ASC-5NS, were able to sustain 8, 11, and 15 load cycles, respectively while the control specimen AS-1NSS failed in the fourth cycle. The enhancements in curvature ductility of the columns were approximately proportional to the amount of confining FRP provided. Comparisons of the behavior of the GFRP-confined square columns and the control specimens as shown in Figure 6 and Table 1 also lead to the same conclusion. Figure 7, 8 and 9 show the behavior of eleven circular columns tested under an axial load of 0.27$P_o$, 0.40$P_o$ or 0.56$P_o$. Seven columns were confined by CFRP or GFRP and others only had steel as lateral reinforcement. The test data of circular columns also demonstrated the enhancement of ductility by FRP confinement. Moreover, addition of FRP confinement provided improvements in the performance of circular columns that were significantly better than those observed in square columns. It should be noted that minimal lateral steel reinforcement was used in all the FRP-confined columns with spacing approximately equal to the size of the column core. The confinement effectiveness of such reinforcement is known to be insignificant and is obvious from the behavior of control specimens AS-1NS,
Axial load level – Another important variable that determines the behavior of a column particularly with respect to ductility is the level of axial load applied. The effect of this parameter can be evaluated by comparing the behavior of several pairs of circular or square columns. As an example, Specimens ASC-2NS and ASC-4NS, which were almost identical in every aspect except for the different levels of axial load applied. An increase in axial load from $0.33P_0$ in ASC-2NS to $0.56P_0$ in ASC-4NS resulted in significantly less ductile behavior. The column resisting a high axial load experienced a decline in ductility ratio of approximately 60% and dissipated energy of about 75%. Its excursion limit was also reduced to 8 cycles from 15 for the specimen under lower axial load. Comparison of specimens ASG-2NSS, ASG-4NSS and ASG-3NSS shows that the effects of the higher axial load can be countered by an increase in the lateral FRP confinement. Specimen ASG-3NSS was strengthened with 4 layers of GFRP and tested at high axial load of $0.56P_0$. Moment curvature responses of ASG-2NSS and ASG-3NSS are very similar, with ASG-2NSS displaying a little more ductile behavior. Similar observations can also be found by comparison among CFRP confined circular columns P27-1CF-3, P40-1CF-8 and P56-2CF-13. The former two columns were confined by one layer of CFRP, while the last column was confined by two layers of CFRP. As the axial load increased from $0.27P_0$ in P27-1CF-3 to $0.40P_0$ in P40-1CF-8, the curvature ductility factor decreased from 25.6 to 9.6. On the other hand, P56-2CF-13, tested under a high axial load of $0.56P_0$ but confined by two layers of CFRP, showed a ductile behavior similar to P40-1CF-8.

### Table 1: Ductility parameters of FRP-confined square columns

<table>
<thead>
<tr>
<th>Researchers</th>
<th>Specimen</th>
<th>$f'_c$ (MPa)</th>
<th>Lateral steel</th>
<th>Layers &amp; type of FRP</th>
<th>Axial load level $P/P_0$</th>
<th>$\mu_\phi$</th>
<th>$N_{\phi_0}$</th>
<th>$N_{\phi_1}$</th>
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</table>

a Control steel-confined specimens.
b Reduction in capacity less than 20% for completed cycles.
### Table 2: Ductility parameters of FRP-confined circular columns

<table>
<thead>
<tr>
<th>Researchers</th>
<th>Specimen</th>
<th>$f'_c$ (MPa)</th>
<th>Lateral steel</th>
<th>Layers &amp; type of FRP</th>
<th>Axial load level $P/P_0$</th>
<th>Ductility ratio</th>
</tr>
</thead>
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<td></td>
<td></td>
<td>Size/Spacing (mm)</td>
<td>$\rho_s$ (%)</td>
<td>$\mu$</td>
<td>$\Phi$</td>
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$^a$ Control steel-confined specimens.
$^b$ Reduction in capacity less than 20% for completed cycles.

(a) Specimens tested under an axial load of 0.33 $P_0$
Figure 5. Behavior of CFRP-confined square specimens

(a) Specimens tested under an axial load of 0.33 $P_0$

(b) Specimens tested under an axial load of 0.56 $P_0$

Figure 6. Behavior of GFRP-confined square specimens

(a) Specimens tested under an axial load of 0.33 $P_0$

(b) Specimens tested under an axial load of 0.56 $P_0$
Figure 7. Behavior of circular columns under $0.27P_o$.

Figure 8. Behavior of circular columns under $0.40P_o$. 
Type of confining FRP- The relative effectiveness of CFRP and GFRP in strengthening deficient columns can be evaluated by comparing the behaviour of four sets of square specimens: ASC-2NS and ASG-2NSS, ASC-3NS and ASG-3NSS, ASC-4NS and ASG-4NSS, and ASC-5NS and ASG-6NSS. The two columns in each set of specimens are similar in every aspect except that one column was confined by CFRP whereas the other one was confined by GFRP. The layers of the GFRP were twice as many as those of the CFRP. Comparisons of the ductility parameters given in Table 1 and the moment-curvature relationships in Figures 5 and 6 show that both columns in each set behaved in a similar manner and had comparable ductility parameters, indicating that the confinement effectiveness of two layers of GFRP is similar to that of one layer of CFRP. This behaviour was also observed in tests of circular columns. The specimens in each of the two pair of columns: ST-2NT and ST-3NT, and P27-1CF-3 and P27-2GF-4, are identical in all aspects except that one was confined by one layer of CFRP and another was confined by two layers of GFRP. As shown in Table 2 and Figure 7, the two columns in each set displayed very similar behaviour and had similar ductility levels and energy dissipation capacities. It is worth noting that in these tests, the ultimate tensile strength of the CFRP fabric was approximately 70% higher than that of the GFRP fabric. The stiffness of CFRP measured in terms of N/mm width per layer was about three times larger than that of GFRP. From these test results, it appears that the effectiveness of FRP in enhancing column ductility closely relates to its ultimate tensile strength.

3. DESIGN OF FRP CONFINEMENT
From the results discussed above, it can be concluded that the column ductility and
energy dissipating capacity increase as the amount of FRP confining reinforcement increases, whereas an increase in axial load level reduces column ductility. These effects are similar to those in steel-confined columns reported by Sheikh and Khoury [15]. While there are similarities between steel-confined and FRP-confined columns, major differences exist between these two types of columns that must be taken into account while designing confinement reinforcement. While the core concrete in steel-reinforced column is confined, the cover concrete outside the lateral steel is not. As the thickness of the cover concrete increases, the area of the confined concrete decreases and larger amount of confinement reinforcement is required to achieve a certain ductility performance. In FRP-confined columns, however, the entire cross section of the column is confined with the FRP wraps that are used externally. The thickness of the cover concrete thus has no effect on column behavior and is not a design parameter. Another important difference is the nature of the lateral confining pressure exerted by steel and FRP. In steel-confined columns subjected to large deformations, the confining pressure remains practically constant while the steel yields under hoop tension. In columns confined by FRP, on the other hand, the lateral confining pressure keeps increasing up to the rupture of fibers due to the linear elastic stress-strain characteristic of the FRP.

For square columns confined by FRP, Sheikh and Li [17] developed a procedure for the design of FRP confining reinforcement on the lines of a procedure Khoury and Sheikh [15] proposed for steel confined columns. In this procedure, the amount of required confinement is related to the level of axial load, lateral reinforcement configuration and ductility demand in terms of curvature ductility factor. The equation to calculate the number of layers of FRP wraps is given below (Eq.1).

$$n \cdot f_u = \beta \cdot h \cdot f'_c \cdot \left\{ 1 + 13 \left( \frac{P}{P_o} \right)^5 \right\}^{1.15} \frac{\mu_{\phi,80,in}}{29}$$

where

$$\mu_{\phi,80,in} = \mu_{\phi,80} - \mu_{\phi,80,con}$$

where

- $n$ = number of layers of FRP
- $f_{FRP}$ = tensile stress in FRP
- $h$ = cross sectional dimension of column
- $f_u$ = the ultimate tensile strength of the FRP obtained from tensile coupon tests
- $\beta$ = confinement efficiency parameter; equal to 0.25 for square columns
- $\mu_{\phi,80}$ = curvature ductility factor of the FRP-confined specimen; and
- $\mu_{\phi,80,con}$ = curvature ductility factor of the control reinforced specimen.

The simplified version of the above equation was given as
The experimental curvature ductility factors and analytical values obtained from Eq.1 and Eq.3 are compared in Figure 10(a). The average of the analytical curvature ductility factors using Eq.1 is roughly equal to the average of the experimental values and the standard deviation from the mean is about 6%, whereas Eq.3 is more conservative and slightly underestimates the curvature ductility of the columns in most cases.

Equations 1 and 3 are applicable to square normal strength concrete columns confined by continuous FRP wraps with continuous longitudinal rebar in plastic hinge regions. For circular columns, following design equations were derived based on similar procedure and existing test data [18].

\[
\begin{align*}
n \cdot f_u &= \beta \cdot h \cdot f_c' \left(6 \frac{P}{P_o} - 1.4\right) \frac{\mu_{g80, in}}{18} \\
&\geq \beta \cdot h \cdot f_c' \frac{\mu_{g80, in}}{18}
\end{align*}
\]  
(3)

The simplified version of Eq.4 was given as

\[
\begin{align*}
n \cdot f_u &= 0.07 \cdot d \cdot f_c' \left\{8.8 \frac{P}{P_o} - 1.2\right\} \frac{\mu_{g80, in}^{1.15}}{29} \\
&\geq 0.07 \cdot d \cdot f_c' \frac{\mu_{g80, in}^{1.15}}{29}
\end{align*}
\]  
(4)
\[ n \cdot f_c = 0.07 \cdot d \cdot f_c' \left( 9.2 \frac{P}{P_o} - 1.4 \right) \frac{\mu_{\phi_{80, in}}}{18} \] (5)

where, \( d \) = diameter of circular column, and all the other parameters are the same as defined for Eqs. 1 and 2. The validity of the design equation for circular column has also been shown by the comparison of analytical and experimental curvature ductility factors in Figure 10(b).

4. APPLICATION OF THE PROPOSED DESIGN APPROACH

The proposed method is applied to a 450 mm square column and a 500 mm diameter circular column, respectively. Both columns, with similar area of cross section, are reinforced with eight longitudinal bars of 25 mm diameter. The concrete compressive strength, steel yield strength and FRP rupture strength are assumed to be 35 MPa, 400 MPa and 900 N/mm width per layer, respectively. Figure 11 shows the number of layers needed as function of the column axial load for two values of ductility enhancement. Assuming that the original columns are capable of displaying a ductility factor of 4, enhancements of \( \mu_{\phi_{80}} \) by 4 and 9 would make the columns moderately and highly ductile, respectively. Addition of one layer of FRP would make the square column moderately ductile if the axial load is 0.5 \( P/P_o \) or lower. For this range of axial load, between 2 and 3 layers of FRP are needed to make the square column highly ductile.

As stated earlier, the results shown in Figure 11 demonstrate clearly that about half the number of FRP layers would be required for circular columns for similar ductility enhancements compare to equivalent square columns. It should be noted from Figure 11 that the simplified equation is significantly more conservative compared to the original equation particularly for high axial load levels.
5. CONCLUDING REMARKS
Selected results from an extensive experimental and analytical research program on seismic upgrade of concrete columns with FRP underway at the University of Toronto are presented. Columns tested under simulated earthquakes were either 305 mm square or 356 mm diameter circular in cross section and 1.47 m long. The experimental results show that variables that affect the ductility parameters of a column include confinement configuration, the level of axial load and the type and amount of confining reinforcement. Circular confinement is more efficient than square confinement. An increase in the level of axial load significantly reduces ductility and energy dissipation capacity of a column. The column performance in terms of ductility improves almost proportionally with an increase in the amount of FRP confinement while strength improvement is less than proportional. A performance-based approach is also briefly presented for the design of confining FRP reinforcement externally applied to square and circular concrete columns. The required amount of confining FRP increases with an increase in ductility demand, an increase in the level of axial load applied and reduced FRP strength.

ACKNOWLEDGMENTS
The authors wish to express their gratitude to Natural Sciences and Engineering Research Council of Canada and ISIS Canada, an NSERC Network of Center of Excellence for financing this work. The experimental work was carried out in the Structures Laboratories of the University of Toronto. Assistance of the technical staff is gratefully acknowledged.

REFERENCES


SIMULATION AND BEHAVIOR OF CORROSION DETERIORATED REINFORCED CONCRETE MEMBERS

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¹School of Civil Engineering, Iran University of Science and Technology, Tehran, Iran
²Moshanir Consultant Engineers, Park Prince Buildings, Tehran, Iran

ABSTRACT
Several reinforced concrete (RC) infrastructures are now crumbling from corrosion of steel bars in concrete. The paper presents the recent advancements in analytical simulation of corrosion aftereffects on behavior of RC members. The model juxtaposes the experimental findings with analytical relationships. The implementation of the model into a nonlinear finite element formulation as well as the experimental and analytical backgrounds are discussed. The abilities of the resulted program have been studied by modeling some experimental specimens showing a reasonable agreement between the analytical and experimental findings.

Keywords: reinforced concrete, corrosion, bond-slip, nonlinear finite element method, tension stiffening

1. INTRODUCTION
The integrity of many RC buildings and infrastructures are compromised due to some dangerous effects of the aggression of the corrosive agents. To evaluate the effects of these types of the damages on the total behavior of reinforced concrete structures, the nonlinear finite element models for reinforced concrete need an improvement to take the effects of corrosion of the steel bars into account. A survey on the literature reveals that there is a knowledge gap in this area of researches; relatively few studies addressed explicitly analytical modeling of corroded reinforcements in RC members. The amalgamation of the available analytical models is presented by Table 1. All of the reviewed models have their own advantages and disadvantages. Some of them sound to be more valuable from engineering point of view while the others seem to be more complicated and suitable at elemental level. The common point of these models is application of especial elements between concrete and reinforcement to represent the bond-slip behavior and associated damages as results of the corrosion of reinforcements. Corrosion of steel reinforcements in the RC structures diminishes the total load bearing capacity of RC structures. This happens not only by means of depletion of rebar cross-sectional area, but also by bond deterioration as reported by some of the researchers, e.g. [6]. Tension-stiffening phenomenon in reinforced concrete is developed as a result of steel and concrete bond that occurs between the tensile cracks. Therefore, degrading effects of corrosion to the bond between steel and concrete could be taken into consideration more effectively by a proper tension
stiffening model. This would be a more practical method to solve the problem than utilization of link elements between steel bar and concrete. For this purpose at first step, a comprehensive experimental program including 58 cylindrical reinforced concrete specimens under various levels of corrosion is conducted. Some of the specimens (44) are located in large tub containing water and salt (5% salt solution). An electrical supplier has been utilized for the accelerated corrosion program. Afterwards, the tensile behavior of the specimens was studied by means of the direct tension tests. For each specimen, the tension-stiffening curve is studied at various load levels. Average crack spacing, loss of cross-section area due to corrosion, the concrete contribution to the tensile response for different strain levels, and maximum bond stress developed at each corrosion level are studied, and their appropriate relationships are proposed. Afterwards based on the experimental program and some analytical relationships, a new bond-slip-tension-stiffening model considering the effects of corrosion of reinforcement was developed. It is implemented into nonlinear finite element relationships as a part of a hypoelastic model of reinforced concrete. Finally, the performance of the program in handling nonlinear analysis of corroded reinforced concrete members is validated.

<table>
<thead>
<tr>
<th>Researchers</th>
<th>Subject of simulation</th>
<th>Framework for constitutive relations</th>
<th>Corrosion and bond-slip representation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coronelli and Gambarova [1]</td>
<td>RC beam</td>
<td>Incremental stress–strain relation for concrete, with smeared rotating cracks</td>
<td>Link element</td>
</tr>
<tr>
<td>Dekoster et al. [2]</td>
<td>RC beam</td>
<td>Elasto-plastic and damage Mechanics</td>
<td>Special link element called &quot;rust&quot;</td>
</tr>
<tr>
<td>Lundgren [3]</td>
<td>Direct tension test specimens</td>
<td>Theory of plasticity</td>
<td>Special layer of elements between steel and concrete</td>
</tr>
<tr>
<td>Lee et al. [4]</td>
<td>RC beam</td>
<td>Incremental stress–strain relation for concrete</td>
<td>Bond element</td>
</tr>
</tbody>
</table>

2. EXPERIMENTAL PROGRAM
A total of 58 specimens have been prepared for the experimental program. Specimens were divided into 7 types according to their sizes (see Table 2). From each type, 2 specimens (totally 14) were used as control and were not placed in the corrosive conditions. The rest of the specimens (totally 44) have been kept in the corrosive environment until the expected level of corrosion achieved. Subsequently, all of the specimens including non-corroded samples were tested by direct tension test on the embedded rod. Medium strength concrete (26 MPa) was used. The mean value of physical and mechanical properties of each type of rebars and concrete were measured (see [7] for details). Concrete cylinder specimens had a constant 500 mm length and variable diameter (60, 100, 150 mm). One deformed
steel reinforcement has been embedded in the middle of the concrete cylinder. This steel bar was extended adequately outside the two ends of the specimen. The specimen diameter and reinforcement diameter have been chosen in a manner to facilitate the feasible study of the effects of some important parameters such as clear concrete cover, ratio of clear concrete cover to rebar diameter, and ratio of rebar diameter to reinforcement ratio. For the construction of the specimens, 500 mm long rubber molds have been used. The molds have been set on a special chassis vertically. Steel reinforcement has been placed in the middle of the specimens to pass through the existing socket on an especial chassis at the end area. This set has been placed on a vibration table and the ready mixed concrete has been cast in mold layer by layer. After 24 hours, the molds have been opened and the specimens were cured in the ambient temperature for 28 days. The exposed parts of the reinforcement and the end areas of the RC specimens at the top and bottom surface were coated by epoxy. The extended steel bar outside the concrete was covered by two layers of tape, electrical tape followed by duct tape. The specimens have been immersed into a fiberglass tub containing a solution of water and salt (5%). An electric supplier has been utilized to subject the specimens to voltage of 24 V and current density of 8 A. The direction of the electric current was set so that the reinforcement served as the anode while the bare metal wire which was spread over the specimens served as cathode (see Figure 1). The duration of the accelerated corrosion procedure was estimated by Faraday's law to reach the required corrosion levels; with periods ranging from one day to one month. The actual values of degree of corrosion were calculated by breaking the specimens to retrieve the reinforcing bar after completion of the tests. The reinforcement bar for each specimen was cleaned and carefully scrubbed with a wire brush to assure that the bar was free from any adhering corrosion products. Special attention had been paid not to alter the base metal. The reinforcing bar was then carefully weighed to determine the actual corrosion degree.

Two special metal plates were fabricated, and each one was affixed to the top and the bottom of the specimens by three screws for measurement of specimen axial deformation. The LVDT (Linear Voltage Differential Transducer) system was employed to measure axial deformations. One gage was attached to the top plate and another one to the bottom plate for this purpose; the values of axial deformation of the specimen in each stage of loading were recorded by connecting them to the data logger apparatus. To measure reinforcing steel elongation, a displacement gage was affixed to the reinforcing bar at the top and another one at the bottom, connected to another LVDT system. The axial tensile forces were applied by a hydraulic jack and measured by a load cell connected to the top bracket. The axial load values were measured continuously by a data logger equipment (see Figure 2). The axial forces were increased to reach yielding capacity of the steel reinforcement; the tests were ceased by the onset of plastic deformations, and the number of transversal cracks and minimum and maximum crack spacing were recorded for each specimen. Eventually, actual degrees of the corrosion were measured for the corroded RC specimens as described earlier.
Figure 1. Accelerated corrosion program [7]

1: Displacement gage for specimen at top
2: Displacement gage for specimen at bottom
3: Displacement gage for bar at top
4: Displacement gage for bar at bottom

(a) Schematic representation (b) Photo

Figure 2. Details of the direct tension tests [7]

Table 2. The specimens overview [7]

<table>
<thead>
<tr>
<th>Type</th>
<th>Specimen</th>
<th>c (mm)</th>
<th>$\rho$</th>
<th>$c/d$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>S12-60</td>
<td>24</td>
<td>0.04</td>
<td>2.0</td>
</tr>
<tr>
<td>2</td>
<td>S12-100</td>
<td>44</td>
<td>0.0144</td>
<td>3.67</td>
</tr>
<tr>
<td>3</td>
<td>S18-60</td>
<td>21</td>
<td>0.09</td>
<td>1.167</td>
</tr>
<tr>
<td>4</td>
<td>S18-100</td>
<td>41</td>
<td>0.0324</td>
<td>2.278</td>
</tr>
<tr>
<td>5</td>
<td>S18-150</td>
<td>66</td>
<td>0.0144</td>
<td>3.67</td>
</tr>
<tr>
<td>6</td>
<td>S25-100</td>
<td>37.5</td>
<td>0.0625</td>
<td>1.5</td>
</tr>
<tr>
<td>7</td>
<td>S25-150</td>
<td>62.5</td>
<td>0.0278</td>
<td>2.5</td>
</tr>
</tbody>
</table>
The Corrosion levels, ultimate crack spacing, concrete stress contribution, bond strength, specimens' total applied tensile load versus average reinforcement strain and effect of corrosion on the cross-section area of the reinforcement are studied for specimens. The results of the experimental investigation reported elsewhere [7]. Some of the empirically obtained formulas are summarized by Table 3.

**Table 3: Review of some proposed relationships [7]**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Formula</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average Final Crack Spacing</td>
<td>$S_m = 2.35c + 1.533 - 0.3 \frac{c}{d_o} + 4.2\left(\frac{C_w d_o}{9c}\right)^2$ for $C_w = 0$ and $C_w &gt; 0$</td>
</tr>
<tr>
<td>Steel reinforcement yield strain</td>
<td>$\varepsilon_y = \frac{f_y}{E_s} = 0.0975 - 0.757 \frac{C_w d_o}{9c} + 0.0087 \frac{c}{d_o}$ for $C_w = 0$ and $C_w &gt; 0$</td>
</tr>
<tr>
<td>Reinforcement cross-sectional area</td>
<td>$A_i = A_{so} \left(1 - 0.35 \frac{C_w d_o}{9c} - 0.08 \frac{c}{d_o}\right)$ for $C_w = 0$ and $C_w &gt; 0$</td>
</tr>
<tr>
<td>Ultimate bond strength</td>
<td>$f_{bu} = \frac{0.4c}{d_o} \sqrt{f_c}$</td>
</tr>
</tbody>
</table>

3. MODELING STRATEGIES

In tension, the model adopts a macroscopic approach that is directly integrated into the concrete law. It simulates implicitly the reinforcing bar-concrete interaction using tension-stiffening factors adjustable according to the nature of specimen that vary as a function of the member strain, clear concrete cover, bond-slip behavior, degree of steel bar corrosion and amount of steel reinforcements. The tension-stiffening curve consists of two distinct states, namely “multiple cracking state” and “final cracking state.” Therefore, the uniaxial tensile stress-strain curve of a RC element could be divided into three states (see Figure 3-a): (a) “uncracked state” (path OA), (b) “multiple cracking state” (path AB) and (c) “final cracking state” (path BC). The numerical strategy of the proposed model is to discretize the tensile stress-strain curve by a set of discrete points called “principal points.” Those are connected by straight line to form a polygon similar to Figure 3-b. The number of “principal points”, $N$, is a constant value for a specific RC element during each analysis. This value probably differs from a RC element to another, depending on its characteristics; the minimum value of $N$ is 4; because at least for describing the reinforced concrete tensile stress-strain curve, three lines are necessary. The computed stress and strain values corresponding to “principal points” are stored in two separate vectors, namely: \{esm\} and \{scm\}; the dimensions of these two vectors are equal to the number of “principal points,” $N$. The value of tensile stress corresponding to the specific tensile strain could be
calculated by a linear interpolation. This concept and a sample for interpolation between the “principal points” are represented in Figure 3-b. For \( i = 1 \), the values \( esm(1) \) and \( scm(1) \) are equal to zero. When \( i = 2 \), the values \( esm(2) \) and \( scm(2) \) are equal to \( \varepsilon'_{cr} \) and \( f'_c \); when \( i \) exceeds 2 the “multiple cracking state” is started and this state lasts until the value of \( a \) becomes less than \( 0.5S_m \). The values of stress and strain corresponding to the “principal points” in “multiple cracking state” is calculated by the following formulas:

\[
esm(2 < i < N - 1) = \frac{1}{n\rho} \left( \cosh(ka) - \cosh(ka - 1) \right) - \sqrt{\frac{1 + 0.5 \cosh(2ka) - 0.75 \sinh(2ka)}{\cosh(ka) - 1}} \cdot \left( \varepsilon'_{cr} - \varepsilon_{cr}(\exp(-550 esm(i) - \varepsilon'_{cr})) \right)
\]

\[
scm(2 < i < N - 1) = \frac{1}{n\rho} \left( \cosh(2ka) - \cosh(2ka - 1) \right) \cdot \left( f'_c(\exp(-550 esm(i) - \varepsilon'_{cr})) \right)
\]

The derivation processes of above equations are presented in [8]. Parameter \( a \) is the half of the spacing between the two faces of two adjacent cracks in a tensile member. The value of \( 2a \) for the first “principal point” of “multiple cracking state” (\( i = 3 \)) is equal to element length perpendicular to crack direction, \( L \). For the second point of “multiple cracking state”, \( i = 4 \), the value of parameter \( 2a \) is bisected and it gets the value of \( L/2 \). For the next points this procedure will be continued until \( a \) becomes less than half of the average final crack spacing (\( S_m \)); at this point (\( i = N - 1 \)) which is called “final cracking point,” the “multiple
cracking” curve is completed. At “final cracking state,” the curve corresponding to this part is idealized by a line which is defined by two points, namely, “final cracking point” \( i = N - 1 \) and “ultimate tensile point” \( i = N \). At this stage, \{esm\} and \{scm\} vectors are computed by these two formulas:

\[
esm(i = N - 1) = \varepsilon_y - \frac{f_{bu} \Psi_{sm}}{A_i E_{sm} 2\sqrt{3}} \quad (3)
\]

\[
scm(i = N - 1) = 0.577 f'_{c} \exp(-550(\varepsilon_{sm} - \varepsilon'_{scm})) \quad (4)
\]

For simplicity and due to the lack of information about corrosion effect on “final cracking state,” the “final cracking state” is neglected for corroded RC elements, therefore, Eqs. \(3\) and \(4\) change to:

\[
esm(i = N - 1) = \varepsilon_y \quad , \quad scm(i = N - 1) = 0 \quad (5)
\]
and the “final tensile point” is calculated by:

\[
esm(i = N) = \varepsilon_y \quad , \quad scm(i = N) = 0 \quad (6)
\]

A comparison between Eqs. \(5\) and \(6\) shows that the elements \( i = N - 1 \) and \( N \) has the same values for the corroded RC elements, leading to removal of the “final cracking state.”

4. APPLICATION

The proposed tension-stiffening model is implemented into a nonlinear finite element analysis program which is called HODA. In this section, the abilities of the developed program on the analysis of field corroded RC beam specimens are verified.

The history, capabilities, element library, constitutive models and limitations of HODA nonlinear finite element analysis program used in this study are discussed elsewhere [9]. This program can depict, through the entire monotonically increasing load range, the static and reversed cyclic response of any plain, reinforced or prestressed concrete structures that is composed of thin plate members. This includes beams, slabs (plates), shells, folded plates, box girder, shear walls, or any combination of these structural elements. Time-dependent effects such as creep and shrinkage can be also studied. The element library includes membrane, plate bending, facet shell, one-dimensional bar, and boundary elements. The facet element has been used for modeling the RC beams. The program employs a layered finite element approach. The structure is idealized as an assemblage of thin constant thickness plate elements with each element subdivided into a number of imaginary layers. Each layer is assumed to be in plane stress
condition, and can be in any state - uncracked, partially cracked, fully cracked, non-yielded, yielded, and crushed- depending on the stress or strain conditions. Analysis is performed using an incremental-iterative tangent stiffness approach, and the stiffness of the element is obtained by adding the stiffness contributions of all layers at each Gauss quadrature point. Appropriate convergence/divergence criteria are utilized to stop the iterations in each load step as soon as a required degree of accuracy has been attained. Concrete are assumed to be as a stress-induced orthotropic material. The hypoeelasticity constitutive relationship developed by Shayanfar has been used for modeling of the uncracked concrete. Smeared crack approach has been adopted for modeling of the cracked concrete. Thorenfeldt, et al. relationship which is able to accurately represent the family of stress-strain curves for different strength concretes including the high strength concrete is employed. In this research, the program has been modified to include tension-stiffening effect considering bond-slip and corrosion effects in reinforced concrete structures. The steel reinforcement is treated in HODA program as an elasto-plastic-strain-hardening material. A slightly modified form of the biaxial strength envelope curve developed by Kupfer, et al. is used in the program built up in the present study [9 and 10].

Three reinforced rectangular beams with $f'_c$ equal to 70.1 MPa –that are almost high strength type concrete- were tested by Lee, et al [4]. Three beam specimens of their tests, namely BCD1 $\frac{1}{2}$, BCD2 and BCD3 are investigated in this study. The beams were 250x200 mm$^2$ in cross-section and they were supported over a clear span of 2000 mm (see Figure 4). It was subjected to two concentrated loads. The details of the reinforcement layout and the geometry of the beams are shown in Figure 4. The material properties of the concrete and the steel reinforcement are given in Table 4. The rate of reinforcement corrosion of each specimen is available on Table 5. Because of symmetry of load and geometry of the beams, only one-half of the beams are modeled in the finite element idealization. The beam specimen is discretized into 120 facet shell elements as illustrated in Figure 4. Plane stress conditions are assumed, therefore only one layer of concrete is sufficient. The longitudinal reinforcements are modeled using discrete bar elements without any flexural stiffness and are lumped in single bars at the reference surfaces. A 4x4 Gauss quadrature is used for estimating the integrations involved. The vertical loads are applied in 30 load steps with smaller increments of loads being applied just before the beam reaches its ultimate load stage. It would improve the rate of convergence of the solution and the accuracy in predicting the failure load. The smeared fixed crack model is used for crack modeling. All of the elements classified into two groups according to their tensile behavior; first group consists of reinforced elements with “tension-stiffening” behavior according to the proposed model; second group, consists of elements without reinforcement and their tensile stress-strain curves are described by linear “tension-softening” behavior. The first group ultimate tensile strain is equal to reinforcement yielding strain, while for the second group, it was chosen near to the strain calculated by a simple formula proposed by Shayanfar et al. [11]; this formula defines the RC element ultimate
tensile strain as function of element size in a very simple manner. This is used to remedy mesh size dependency in a nonlinear finite element formulation for reinforced concrete structures.

The analytical and experimental load-deflection curves for the beams BCD1 to BCD3 are plotted in Figure 5. The analytical results are a little bit stiffer than the experimental results and in good agreement with experimental findings. The stiffer response of model can be related to non-uniform corrosion, pitting, and longitudinal cracking due to rebar corrosion and the other items that arise from haphazard nature of corrosion and cracking phenomenon in RC members.

Figure 4. BCD1, BCD2 and BCD3 experimental details and finite element idealization

Figure 5. Experimental and analytical comparison
Table 4: Material properties of RC beams

<table>
<thead>
<tr>
<th>Properties</th>
<th>Beams</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_i (mm^2)$</td>
<td>256.46</td>
</tr>
<tr>
<td>$f_c (MPa)$</td>
<td>70.1</td>
</tr>
<tr>
<td>$E_0 (MPa)$</td>
<td>38500</td>
</tr>
<tr>
<td>$\varepsilon_c$</td>
<td>0.002</td>
</tr>
<tr>
<td>$\varepsilon_{cu}$</td>
<td>0.004 (Assumed value)</td>
</tr>
<tr>
<td>$f'_c (MPa)$</td>
<td>3.67</td>
</tr>
<tr>
<td>$f_y (MPa)$</td>
<td>359.4</td>
</tr>
<tr>
<td>$E_s (MPa)$</td>
<td>197000</td>
</tr>
<tr>
<td>$E'_s (MPa)$</td>
<td>1300 (Assumed value)</td>
</tr>
<tr>
<td>$\varepsilon_{su}$</td>
<td>0.15</td>
</tr>
<tr>
<td>$E_b (MPa/mm)$</td>
<td>450 (Assumed value)</td>
</tr>
</tbody>
</table>

Table 5. Rate of corrosion in RC beams

<table>
<thead>
<tr>
<th>Beams</th>
<th>$C_w (%)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>BCD1</td>
<td>3.8</td>
</tr>
<tr>
<td>BCD2</td>
<td>7.9</td>
</tr>
<tr>
<td>BCD3</td>
<td>25.3</td>
</tr>
</tbody>
</table>

5. CONCLUSION AND SUMMARIES
The conclusions drawn from experimental investigation, which is presented with more details in [7], could be summarized as follows: The tensile behavior of reinforced concrete considering corrosion effects was studied experimentally. The specimen properties were chosen in a manner to reflect the effects of the governing parameters in the tensile behavior of the reinforced concrete specimens. The following conclusions are drawn:

1. (a) The ultimate crack spacing for non-corroded specimens mainly is related to the clear concrete cover of specimens. (b) This length will be increased for corroded specimens by increasing the degree of corrosion and appearance of longitudinal cracks. (c) This is merely related to the decline of the bond between concrete and steel reinforcement demanding for greater length to transfer tensile forces from the steel to the concrete.

2. The study of the specimens concrete stress contribution versus steel reinforcement average strain reveals that the tension stiffening of reinforced concrete is very sensitive to the degree of reinforcement corrosion. Severe corrosion results in bond breakdown between concrete and steel reinforcement; nearby no contribution of the concrete in the tensile response.
beyond cracking (tension cut-off) could be expected for specimens in such condition. The lower levels of the reinforcement corrosion have considerable impacts on the stiffness and the ultimate strain of the tension-stiffening curve. The ultimate tensile strain of these curves for non-corroded specimens is close to the reinforcement yielding strain, but it will be reduced nearby to the strain corresponding to the tensile strength of concrete by increasing the levels of corrosion. The study on total applied tensile forces versus the average reinforcement strain curves also shows that the corrosion will decrease concrete stress contribution in cracking states.

3. The study of ultimate average bond strength deterioration of specimens and loss of cross-section area of reinforcement as result of corrosion reveals that the ratio of clear concrete cover to the size of the reinforcement has an important role in controlling these two important effects of corrosion.

4. Some empirical formulas for prediction of some of the studied parameters (e.g. ultimate crack spacing) were proposed and compared with the experimental findings of a similar program. Acceptable correlations were observed between the results of these two research programs.

Moreover, a new semi-analytical model describing the tension-stiffening phenomena considering bond-slip behavior and corrosion is represented; see also [8]. The model splits the tension-stiffening curve into two states, namely: “multiple cracking state” and “final cracking state.” The proposed procedure predicts the “final cracking point” by an experimental criterion by setting a lower bound for the average final crack spacing parameter. Another novel aspect of the tension-stiffening model is the discretization of the tension-stiffening curve by a set of points called “principal points” and using linear interpolation technique for computing tensile stress corresponding to a specific tensile strain. This model has been implemented into the HODA program. This program utilizes the hypoelastic model and the smeared crack approach. The model has been tested by means of analyzing three field RC beams; the RC beams have the same geometry and material property but different rates of tensile rebar corrosion. The analytical responses using HODA program reveals good agreements with the experimental findings. The principal features of this paper in a quick view are:

1. Without using any special element between concrete and steel by only modifying the tension-stiffening curve depending on the rate of steel bar corrosion, the corroded RC elements can be modeled with a reasonable accuracy. This method is applicable for a vast variety of steel reinforcement corrosion rates.

2. A new bond-slip-tension-stiffening algorithm has been introduced.

3. Ductility and the failure points of the corroded reinforcements RC members have been predicted reasonably by the means of a simple nonlinear finite element model.

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INTERNATIONAL TRENDS IN BUILDING & CONSTRUCTION RESEARCH & DEVELOPMENT

W. Bakens,
Secretary General CIB, International trends in Building & Construction Research & Development

1. BACKGROUND AND INTRODUCTION
CIB—the International Council for Research and Information in Building and Construction—is the world’s foremost network of organizations and persons in this area. Its objective is to stimulate the worldwide exchange information and cooperation in international projects, on all aspects of building and construction,
- with activities in the areas: technology, design and process for buildings and the built environment
- with members from research, academia, industry and other built environment stakeholders
- operating through experts commission, priority themes, student chapters and special interest clubs

Two groupings of worldwide / international trends and developments:
- of a more general nature and related to developing governmental policies
- related to priority themes for building and construction R&D.

2. TRENDS IN B&C R&D OF A MORE GENERAL NATURE
1 In many developed countries, with a built environment that is more or less “ready”: Lessening of political interest in B&C R&D and withdrawing of government as subsidizer of collective research, resulting in:
   - growing impact of industry stakeholders
   - growing focus in R&D on industry requirements / applicability of R&D / innovation
   - increasing commercial pressures on B&C R&D institutes / slimming or even disappearing B&C R&D institutes.
   In CIB we see this reflected in the form of a growth since a few years especially of industry members and through a former emphasize in CIB Meetings on academic discussions being replaced by ones on practice oriented projects.

2 In Europe: R&D programs directed and financed from Brussels, resulting in:
   - EU cooperation and competition in programs, but now also between institutes beyond programs; resulting for example in regional cooperation / mergers or specialized (expensive) international R&D laboratories, with Brussels’subsidies.
In CIB we are realizing that we are in fact competing with such funded EU R&D programs and we notice that no longer our growth in membership is originating from Europe but from the other regions in the world.

3. WORLDWIDE B&C R&D PRIORITY THEMES—CIB PRIORITY THEMES

1 SBC-Sustainable Building and Construction CIB started with SBC as a Priority Theme in 1995, but after more than 10 years it is still that important worldwide as a “driver” of R&D. Dominant developments in the SBC area over these years include
- changing focus: from an emphasize on “simple” topics like energy, waste pollution and recycling, via a more holistic but also complex approach looking for balance between social, economic and environmental sustainability, now “back to a “relative simple” approach again with an emphasize on topics like energy (again, but far more ambitious) and climate change (both mitigation and adaptation).
- SBC becomes a strategic priority in other parts of the world, for example in China-SBC/Green Building becoming a necessity in mainstream construction practice, resulting for example in the need for assessment/labeling/certification systems.

CIB Priority Theme since 1995: SB Agenda, SB Conference series, various Commissions, partnership with the UN.

2 RC-Revaluing Construction There is an acknowledged need in many countries to develop towards a substantially different and high performing industry that is integrated and transparent in organization, that is pro-active, innovative and client and user oriented in attitude, that is perceived to be of high value to society and that attracts high quality and well educated employees.
- examples of national reform programs include: Rethinking Construction in the UK, PSIBouw in The Netherlands, CRC for Construction Innovation in Australia en various such programs in Scandinavian countries
- R&D themes in such national reform programs include: procurement and process (with assumed decisive roles for large public clients), with attention for example for LCC based procurement and performance procurement, and industrial production processes.

CIB Priority Theme since 2001: RC Agenda, various Commissions, RC Conference series

3 IDS-Integrated Design Solution in Construction After two decades of developments in international research now – since 1 or 2 years – a sudden exploding attention all over the world for the use of BIM - Building Information Models = all partners in a project (for example in one joint server) really sharing all information in one model throughout all phases of the project, based on IFC information standards. The technology for this seems to be more or less ready for application (or not yet?).
- the application of BIM will result in a substantially different process for
the planning, design, construction and management of buildings and infrastructural works; one that requires not only different information management, but also different liabilities, communication, procurement, culture etc.

-BIM will offer new possibilities for integrated CAD/CAM processes and create a new context for Automation and Robotics in Construction.

-the application of BIM is being made mandatory in the public procurement of buildings in some North European countries, but also by organizations like GSA in the USA.

IDS is a CIB Priority Theme since 2006 and is still in stage of program elaboration: first CIB IDS conference in Helsinki, Finland in June 2009

Summary trends in worldwide B&C R&D:
- in developed countries: growing role for the industry / growing emphasize on applicability of R&D outcomes, but also growing commercial pressures in some case resulting in slimming down or even disappearing B&C R&D institutes
- in Europe: development towards an international R&D market, with funding for collective R&D from Brussels = unique in the world
- surprisingly many similarities in defining Priority Themes, with today’s B&C R&D in many countries being driven by priorities i) still for Sustainable Building and Construction but now with amongst other a strong focus on issues related to climate change and the built environment, ii) Revaluing Construction with its often very ambitious industry reform programs and with a lot of attention for applying new organizational models and iii) the aim for Integrated IT based Design Technologies with in this context the ‘sudden” worldwide and often mandatory application of BIM.

Closing Remarks
Not one country has the resources to be leading all developments. For all countries the principle applies that 80 – 90% of needed new knowledge and technologies is being developed abroad.

State-of-the art knowledge and information on all such themes B&C R&D is being brought together in CIB and is being made accessible for all its members.

We live in fast changing world: one in which R&D has a great potential to be of value to all built environment stakeholders and one in which modern technologies can very substantially increase the performances of our industry.

It is important for the Iranian B&C R&D community to become engaged in CIB as actively as possible.
CONSTRUCTION INDUSTRY DEVELOPMENT IN DEVELOPING COUNTRIES; LESSONS AND OPPORTUNITIES

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ABSTRACT
This paper deals with the development of the construction industry, with a particular focus on developing countries. The paper brings together some of the lessons learned from the national programmes around the world, and explores the opportunities for enhancing the role of research infrastructure and national laboratories in supporting such change.

The paper notes that in those countries that have adopted national initiatives to develop the construction industry, opportunities exist for the research infrastructure (the universities and research laboratories) to make a meaningful contribution to the development of the industry. In fact, it is likely that any national activity cannot succeed without the active support and participation of the research infrastructure. Specifically, research organisations underpin the development of the construction industry in any country.

Keywords: construction industry development, research

INTRODUCTION
Ladies and gentlemen.
My address deals with construction industry development, with a particular focus on developing countries. My aim is to:

• bring together some of the lessons learned from the national programmes around the world; and
• explore some of the opportunities for enhancing the role of research infrastructure and national laboratories in supporting such change.

WHAT IS CONSTRUCTION INDUSTRY DEVELOPMENT?

• Construction industry development is the deliberate and managed process to optimise the contribution of the construction industry in:
  meeting national construction demand;
  • promoting national social and economic development objectives;
  • promoting industry performance and competitiveness; and
  • providing improved value to clients.

This definition was crafted by, amongst others, Prof George Ofori, Jill Wells and Spencer Hodgson at the 1st Conference on Construction in Developing Countries in Arusha, Tanzania, September 1998, organised by TG29 of the CIB.
PARTICIPANTS IN CONSTRUCTION INDUSTRY DEVELOPMENT

This definition of construction industry development addresses the role and the contribution of all participants who add value to the delivery process;

- from project inception to project handover and maintenance; and includes
- the public and private sector clients, build environment professionals, constructors, materials manufacturers and suppliers, training delivery institutions, regulatory bodies and research institutions.

Furthermore, this definition does not differentiate between the local industry and the foreign industry – but in developing countries a component must clearly include a focus on the development of the local (or indigenous) industry.

AN INTERNATIONAL PERSPECTIVE

Internationally, several countries have established formal or informal national programmes to support the objectives of construction industry development. The “drivers” that have led to the creation of these national initiatives show both common elements and local dimensions.

Common elements include (after Roger Courtney, 2002):

- a recognition that construction accounts for a significant proportion of national economic activity and that the effectiveness of the sector has implications for other industries and for public services;
- a perception that construction, in contrast to other industry sectors, has not improved its use of labour and its overall productivity as much as other sectors in recent decades and that as a consequence its outputs are becoming relatively more expensive;
- a view that a key factor in the allegedly poor performance of construction is the number of different parties who have responsibilities within the construction process and therefore a desire to bring about a more integrated process; and
- overall, a view that construction should, by integrating its internal processes and adopting new information and production technologies, seek to become more similar to manufacturing sectors.

Before looking at these factors in more depth, it is useful to look at construction industry development from a developing country perspective.

A DEVELOPING COUNTRY PERSPECTIVE

Although many of the challenges facing the construction industry in developing countries are similar to those in developed countries (and hence much can be learnt from the initiatives in developed countries), there are also significant differences between the developed and developing countries. In particular, Ofori (1999, 2001) notes that:

- although construction may account for a significant proportion of national economic activity in developing countries, in many cases the indigenous
industry is weak and under-developed, and much of the construction activity is undertaken by multinationals;

- the construction industry in many developing countries is facing reduced levels of demand as a result of adjustment programmes which invariably involve cuts in governments’ capital investment; and

- with public funds under severe strain and chronically short, ways must be found to structure funding strategies which are suitable for the developing countries.

DRIVERS FOR CHANGE; LOCAL ELEMENTS

Other factors, or local elements, have been prominent in many countries which have created the need for a national or industry focus on the development of the industry, including:

- in the UK, major clients (notably utility companies that had been previously public sector organisations) wished to achieve better value from their investments in construction and sought new relationships and procedures to obtain this;

- in the UK also, the poor image of construction, as perceived by prospective employees and the Stock Market, caused firms to consider how new forms of operation could both provide more attractive employment conditions and higher levels of profitability;

- in Singapore, there was a realisation that the industry was heavily dependent upon relatively unskilled operatives, many from outside Singapore, and a principal focus of the review was therefore skills requirements and means of making more effective use of labour;

- in South Africa, the need to create an enabling environment for the transformation of the industry to create economic opportunities for all participants;

- in Hong Kong, some prominent defects in new construction works, such as inadequate foundations, had revealed shortcomings in quality practices, and – in the extreme – corrupt practices. Institutional arrangements for both procurement and in the carrying out of the works (e.g. the use of multi-layer sub-contracting) became a principal focus of the review;

- a desire to increase the international competitiveness of the construction sector, so that it could secure a higher proportion of business from other countries, was a factor in Hong Kong, Singapore and Australia.

It is seen from these examples that the focus of the development of the industry is very context specific, and depends on the needs of the country at a specific point in time.

NATIONAL PROGRAMMES

Internationally, the national programmes to support construction industry development broadly fall into three categories, namely:

- those that have established national initiatives to develop the construction
sector, involving a wide range of actions (e.g. UK, Australia, Hong Kong, Singapore, South Africa, India, Denmark);

• those that have recognised many of the needs of construction industry development in research programmes and other activities, but have not brought these together in a national initiative (e.g. Finland, the USA, Sweden); and

• those that have no national programme, but have some activities that address construction industry development (e.g. France, Malaysia, the Netherlands, Chile, Japan).

These categories do not have sharp boundaries, and some countries (e.g. Denmark) might be placed in a different category.

My presentation to follow will only focus on national initiatives.

THE SOUTH AFRICAN EXPERIENCE

While the need for, and focus of, the development of the industry differs from country to country, it may be useful to reflect on some aspects from the South African experience in developing the construction industry.

The impetus to the start of the reform process in South Africa began before the democratic elections in 1994, in which political power was transferred from a minority to the majority.

But the passage of transfer of economic opportunities to the majority was not going to be easy. Prior to 1994, the black population was excluded from most economic and many social opportunities (including meaningful education and skills development). Amongst others, a clear need therefore existed to promote the development of black contractors and participants – most of which had been excluded from meaningful participation in the construction industry.

A DECLINING CONSTRUCTION INDUSTRY

While the need for transformation was clear – prior to 1994 (like many other countries), the demand for construction in South Africa was in decline, resulting in increased competition, shedding of labour and skills, limited recapitalisation of equipment, and so on.

Simply transferring economic opportunities from one sector to another was not a viable option, and the development of the industry depended on:

• growth in infrastructure investment - providing increased opportunities;

• growing the capacity of the industry to meet the increased demand – and particularly amongst the previously disadvantaged sector; and

• improving the performance of all participants to deliver value for money to clients and to meet socio-economic objectives.

THE SA GREEN AND WHITE PAPERS

The process for setting up the framework the development of the construction industry in South Africa included the development of a government Green Paper in 1997, a White Paper in 1999, and legislation for the establishment of the
The South African Green and White Papers parallel in many ways the frameworks developed in other countries, such as:

- the Latham and Egan Reviews in the UK;
- the Australian Construction Industry Development Agency (CIDA) and the National Building and Construction Committee (NatBACC) Action Agenda – Building for Growth; and, more recently
- Vision 2020 in Singapore.

THE CONSTRUCTION INDUSTRY DEVELOPMENT BOARD (SA)

The cidb was established in South Africa in 2000, reporting to the Minister of Public Works, and currently has a staff complement of about 120 people, grouped around:

- corporate functions;
- registration of contractors and projects;
- procurement and delivery management;
- growth and contractor development; and
- performance improvement.

Again, the cidb parallels in many ways the institutional structures developed in several countries, including:

- the Rethinking Construction, Constructing Excellence, Better Public Buildings and other initiatives in the UK;
- the performance improvement programmes of the Building and Construction Authority (BCA) of Singapore;
- the initiatives of the Construction Industry Development Board (CIDB) of Malaysia;
- the Australian Construction Policy Steering Committee (CPSC) Construct NSW and other initiatives of the government of New South Wales.

DRIVING CHANGE

Turning again to international experience, the ability and approach adopted to influence stakeholders under a national initiative varies significantly from country to country. However, there are many commonalities.

Firstly, internationally, many governments have assumed the lead responsibility for driving reform initiatives in the context of national socio-economic objectives.

Governments are more able to organise public sector clients around reform initiatives (albeit usually under the instructions of a higher authority), driving change within their own business functions (such as procurement reform), and driving change amongst their suppliers (largely through the procurement regime).

On the other hand, there are only a few successful examples of private sector clients collectively driving change and reform initiatives amongst themselves or amongst their suppliers – other than amongst a small number of forward looking
committed private sector clients.
Similarly, there are few successful examples of construction industry suppliers collectively driving change amongst themselves – with the most notable examples being the benchmarking initiatives in the USA.

AFFECTING CHANGE
International experience has shown that effecting change at a national or sector level is complicated, resource intensive, and can only be achieved over a relatively long period of time (in some cases up to 10 years or even longer). Notwithstanding the huge challenges, numerous reform initiatives have been initiated around the world to support the necessary development of the construction industry – and many of these have shown progress towards their objectives for reform. An assessment of these reform initiatives (some of which themselves have been in existence for 10 years or longer) highlights key criteria for the success of such reform initiatives, and collectively these point towards a structured framework that can be adapted to national performance initiatives.

The international reform initiatives reviewed by the authors include, amongst others:

- the Rethinking Construction, Constructing Excellence, Better Public Buildings and other initiatives in the UK;
- the performance improvement programmes of the Building and Construction Authority (BCA) of Singapore;
- the Australian Construction Industry Development Agency (CIDA) and the National Building and Construction Committee (NatBACC) Action Agenda – Building for Growth;
- the Australian Construction Policy Steering Committee (CPSC) Construct NSW and other initiatives of the government of New South Wales;
- the initiatives of the Construction Industry Development Board (CIDB) of Malaysia;
- the recently initiated Process and System Innovation in Building and Construction (PSIB) programme in the Netherlands; together with
- a range of government and client driven initiatives aimed at furthering sustainable development, and in particular environmentally sustainable development.

Some key elements of these initiatives are described in the following sections.

LEADERSHIP
Without exception, the role of leadership by individuals and/or organisations has been fundamental to the success of every one of the more successful international reform initiatives. Common forms of leadership that are observed in the international reform initiatives include:
Leadership by *government* (either individuals of government departments) – demonstrating commitment and willingness to the reform initiatives. Examples of such leadership by government include the procurement reform initiatives being carried out in the Office of Government Commerce (OGC) in the UK, the *Better Public Buildings* initiative in the UK, and the *Construction Client Charter and Demonstration Projects* initiatives in the UK.

Leadership by influential forward thinking and progressive *private sector* organisations, and in particular private sector clients, is relatively common internationally. Examples include those initial private sector clients participating on the UK *Construction Client Charter*, and members of influential organisations such as the *World Business Council for Sustainable Development*.

**OBJECTIVES CREATE THE FOCUS**

Performance improvement programmes are generally driven by high level goals and objectives. For example, the Singapore programme to promote buildability derives from the national objective to limit the need for imported labour by improving productivity. Clarity on priority reform objectives is of the utmost importance to ensure focus, and is usually informed by policy, legislation and industry reviews. The objectives usually have to be cascaded out from higher level objectives to more manageable lower level objectives, which can then be prioritised. For example, the *Construct NSW* agenda set out an integrated framework of 20 strategies and 85 supporting actions to enable the government to achieve best value for money from its construction procurement, to support its economic and social goals through construction procurement and to assist the industry to achieve its potential. These strategies were then grouped under 8 headings-analogous to objectives – including:

- strategic information for decision making;
- business ethics and practices;
- security of payment;
- management and workforce development;
- continuous improvement;
- towards an ecological sustainable industry; and
- encouragement and recognition.

**AWARENESS**

*Awareness creation and promotion* is fundamental to furthering the objectives of reform initiatives, so as to continually reinforce the reform message, and to broaden the awareness and understanding of the reform initiatives. There are numerous examples of successful (and unsuccessful) awareness creation and promotion activities internationally, including:

- targeted awareness creation in the popular and technical press;
- award systems, such as the Considerate Contractor Scheme and the Prime Minister's Better Public Building Award in the UK;
• forums, benchmarking clubs, and demonstration projects, and
• periodic reporting, on the state of the industry or industry reform.

INFORMATION AND TOOLS
The development and dissemination of appropriate information and tools to support the attainment of reform objectives is a further key success factor, and as illustrated below can take various forms. Note, however, that many of the systems outlined below are in fact enforced through various instruments in many of the reform initiatives around the world, but the systems themselves provide a tool together with information to equip various stakeholders for change:
• codes, standards and guidelines, both voluntary and enforced through legislation;
• best practices, applicable to almost every reform initiative around the world;
• management systems, together with supporting implementation tools, specifying processes to be adopted and reported on, varying from full ISO 9000 and 14000 accreditation (which is currently required on selected projects in Singapore), to the management systems developed to target specific issues – such as the NSW Australia OHS&R Management Systems and Environmental Management Systems;
• accreditation and rating systems together with supporting implementation tools, such as the LEED environmental design accreditation of design professionals in the USA, the NSW Contractor Best Practice Accreditation System, and accreditation systems for buildings – predominantly environmental and quality systems;
• triple-bottom line reporting schemes and methods, which are becoming increasingly common around the world.

CAPACITY BUILDING
Capacity building is key to several of the international reform initiatives, including:
• formal training programmes for public sector officials that support reform initiatives;
• the establishment of public sector Centres of Excellence (such as the OGC Programme and Project Management Centres of Excellence in the UK), whose aim is to achieve significant improvement to central government capability to deliver successful programmes and projects;
• the sponsorship of formal and informal training programmes for private sector participants impacted on by reform initiatives.

In addition, many of these capacity building programmes in the public sector are supported by the development and implementation of performance management systems for public sector officials that are aligned to the reform initiatives.

ENFORCEMENT AND COMPLIANCE
All reform initiatives around the world are dependent to a greater or lesser degree
on enforcement and compliance mechanisms. These mechanisms vary significantly, and include:

- **legislation** to seek compliance with minimum acceptable standards (such as safety and health, and certain environmental considerations). In NWS, legislation has also been introduced to effect the prompt payment of subcontractors;
- **procurement instruments**, which are one of the most powerful instruments used in all reform initiatives for effecting change amongst suppliers, i.e. clients (typically government clients) specifying their requirements (aligned with the reform objectives) for other parties wishing to do business with them – often requiring compliance with codes of conduct, standards, and guidelines, or the mandatory use of management systems;
- **registration and accreditation** of contractors, designers, etc. according to specified criteria for different types of activities – including “construction registers”, which is typically implemented through legislative or procurement means; and
- **commitment to voluntary compliance** together with review mechanisms, to charters, codes of practice and/or conduct, management systems, reporting, including:
  - the UK *Construction Client Charter Improvement Programme*, in which clients commit to continually improving their performance in 4 themes;
  - the numerous environmental and social responsibility charters – such as the *Equator Principles* developed by leading international financiers; and the *FIDIC Integrity Management System*, adopted by the World Bank and others.

**MONITORING, EVALUATION AND REVIEW**

Regular monitoring, evaluation and review is an essential requirement for the successful implementation of any strategy, and is a key element of all international reform initiatives – and takes place at both the “macro-level” and the “micro-level”. For example:

- at the “macro-level” the UK has instituted the *Construction Industry Indicators* and the *Quality of Life Indicators* – setting high-level performance targets for the industry together with ongoing monitoring against these targets; and
- at the “micro-level”, the UK has initiated the OGC “Gateway” review process for acquisition programmes and procurement projects, and the CABE “Design Review” for buildings that will have a significant impact on their environment, while Singapore and Australia require closeout reviews of projects against certain criteria (which will also shortly be introduced in South Africa).

**RESEARCH INFRASTRUCTURE**

Turning now to the role of the research infrastructure in supporting national initiatives.

It stands to reason that research infrastructure, including national laboratories and
academic institutions, can play a strong role in supporting the development of the construction industry and any national programmes. However, the relationship between the research infrastructure and national initiatives has not always been clear—often due to conflicting priorities and conflicting departmental reporting lines.

**FUNDING STREAMS**
The past 20 years or so has seen significant changes in the landscape impacting on national laboratories due to changing funding streams, impacting in particular on national laboratories in Europe, the UK, Australia and South Africa.

For example, in the built environment, the Building Research Establishment (BRE) was privatised in the middle of the change programme in the UK, notwithstanding a very strong link between the UK reform initiative being driven from the then Department of the Environment, Tourism and the regions (DETR) and the DETR being a principle funding agency of the BRE.

Similarly, although not linked to any national change initiative (but certainly linked with supporting governmental departmental needs and priorities), changes in funding streams from the Department of Transport in South Africa had a major negative impact on transportation and road engineering at the CSIR. Similarly, changes in funding priorities from the Department of Science and Technology resulted in the closure of the structural, materials engineering and other laboratories at the building research facilities at the CSIR in South Africa— and in all probability these facilities have been lost to South Africa (and in fact Africa) for good.

Clearly, if the national research infrastructure is to support national change initiatives supporting the development of the construction industry, then it has to be funded appropriately.

**CONFLICTING PRIORITIES**
However, changes in funding streams have also been associated with changes in research priorities initiated by the research institutions themselves or by government departments.

For example, in South Africa, which probably mirrors some countries in the rest of the world, we have seen changes in which the national laboratories were originally set up in the 1950s as “agencies” of government departments to support the developmental objectives of the government departments. With the withdrawal of government funding for research in the 1980s and 1990s, these laboratories followed a more market orientated and commercial approach—resulting in research directions being dictated by commercial opportunities. More recently, in the 2000s, the national laboratories have adopted an increasing blend of “science for science sake” in order to rebuild a deteriorated research base in these laboratories.

All these changes in South Africa, and in many parts of the world, have largely been driven by a disjuncture between:

- research funding streams often determined by departments of science and technology; and
• developmental objectives determined by government service delivery departments and/or industry.

LINKING RESEARCH TO DEVELOPMENT NEEDS
I have always held the view there has to be a close link between research institutions and national or industry policies and priorities – but these have to be appropriately funded through the same mechanism.

In South Africa, the cidb (like some other countries in the world) have adopted an approach to consciously build capacity at selected departments within the research infrastructure in South Africa that is aligned with the objectives of construction industry development. It is our aim to build selected Centres of Excellence that can support the cidb – but in reality this vision will only be realised when the cidb is able to fund these Centres of Excellence on a sustainable basis. The cidb is in the process of developing such funding models and funding streams.

SUMMARY
Ladies and gentlemen, it is not for me to pass comment on whether there is a need or an opportunity for a focused initiative on the development of the construction industry in any country – that needs a deep understanding of context specific issues in that country.

But, in those countries that have adopted national initiatives to develop the construction industry, opportunities exist for the research infrastructure (the universities and research laboratories) to make a meaningful contribution to the development of the industry. In fact, it is likely that any national activity cannot succeed without the active support and participation of the research infrastructure.

And where it is not necessary to have a national focus on the development of the industry, or where such national focuses have not been initiated, opportunities still exist for the research infrastructure to make a meaningful contribution to the development of the industry – albeit that it may be more difficult to do so.

Research organisations, such as those represented here today, underpin the development of the construction industry in any country.

CONCLUDING COMMENTS
In conclusion, I am proud to have been a President of the International Council for Research and Innovation in Building and Construction (CIB). The membership of the CIB currently numbers over 400 members originating in some 70 countries.

CIB members include most of the major national laboratories and leading universities around the world in building and construction.

The CIB facilitates international cooperation and information exchange in building and construction research and innovation. Individually, and collectively, the members of CIB play an important role in many countries in supporting the development of the construction industry.

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INTERNAL STEEL BRACING OF RC FRAMES

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ABSTRACT
Steel bracing systems can be used effectively for seismic retrofitting of existing RC buildings as well as for seismic design of new buildings. Although adaptation of bracing to upgrade the lateral load capacity of existing RC frames has been the subject of a number of successful studies, guidelines for its use in newly constructed RC frames need to be further developed. This paper reports on some recent experimental and numerical work conducted by the author and his colleagues on internal bracing of RC frames using direct connections between the bracing system and the frame. The effects of X-bracing and knee bracing on enhancing the seismic capacity of the frames are investigated experimentally through pushover tests as well as cyclic tests. A compression release device has also been introduced and tested to enhance the seismic performance of the bracing system by avoiding the buckling of the compression member. An important consideration in the design of steel-braced RC frames is the level of interaction between the strength capacities of the RC frame and the bracing system. In this paper, results of experimental and numerical investigations aimed at evaluating the level of capacity interaction between the two systems are also discussed. It is found that the capacity interaction is due primarily to connections overstrength. Based on the numerical results the connection overstrength has been quantified and guidelines for the seismic design of the internally braced RC frames with direct connections are provided.

1. INTRODUCTION
Steel bracing is generally used to increase the lateral load resistance of steel structures. In recent years, the concept of steel bracing has also been applied to the retrofitting of reinforced concrete frames. Increased architectural flexibility, reduced weight of the structure, ease and speed of construction and the ability to choose more ductile systems can be considered as the main advantages of steel bracing in comparison with RC shear walls. Two bracing systems are generally used, external bracing and internal bracing. In external bracing, steel trusses or frames are attached either as a global external support to the building exterior or, more locally, to the face of individual building frames. A number of investigators have reported on the efficiency of external bracing in seismic retrofitting of existing RC buildings [1-4]. Architectural concerns and difficulties in providing appropriate connections between the bracing system and RC frames are two of the shortcomings of this method.

In internal bracing, steel bracing members are inserted in the empty space enclosed
by columns and beams of RC frames. As a result, each unit frame is individually braced from within. The bracing may be attached to the RC frame either indirectly or directly. In the indirect internal bracing, a braced steel frame is positioned inside the RC frame. As a result, the transfer of load between the steel bracing and the concrete frame is carried out indirectly through the steel frame. Successful retrofits of existing buildings by indirect internal bracing using different forms of X, V and K concentric and eccentric braces have been reported in the literature [5-8]. In some repair and retrofitting cases, provision of the steel frame may be necessary to reduce the strength demand on an already damaged and weakened RC frame; however, in other instances the steel frame acts only as a costly connecting mechanism with inhibiting technical difficulties in fixing the steel frame to the RC frame.

To overcome the shortcomings of the indirect internal bracing, Maheri and Sahebi [9] first recommended using direct connections between the brace elements and RC frame without the need for an intermediary steel frame. In an experimental work, they showed the ability of this bracing system to enhance the strength capacity of RC frames. Later experimental work on directly braced model frames by Tasnimi and Masoomi [10] also showed the applicability of this method. Recent analytical work carried out by Abou-Elfath and Ghobarah [11, 12] on both concentric and eccentric direct internal bracing in non-ductile RC buildings also showed an improvement in the seismic performance, particularly when using eccentric bracing. In continuation of their previous work, Maheri et-al [13] conducted experimental investigations on pushover response of scaled RC frames; braced with both diagonal bracing and knee bracing systems. In this study the effectiveness of the two bracing systems in increasing some seismic performance parameters was shown. Also, in a theoretical study, Maheri and Akbari presented the behaviour factor, $R$, for this class of dual systems [14].

Appropriate design of direct connections between the bracing members and the RC frame is important to achieve the required lateral load capacity. Maheri and Hadjipour [15] proposed a connection that minimizes the eccentricity of the brace member force. This allows transferring the brace force to the corner of the RC frame without producing local damage in concrete members. Using the results of an experimental program conducted on a number of full-scale connections, they also presented design guidelines for the brace-frame connections in new construction. Recent experimental works by Youssef et-al [16] and Ghaffarzadeh and Maheri [17-19] have shown further that different directly-connected internal bracing systems can be used effectively in retrofitting of existing concrete frames as well as shear resisting elements for construction of new RC structures.

2. SEISMIC RESPONSE PARAMETERS OF X-BRACED FRAMES

2.1. Pushover Tests

In an experimental study, details of which are presented in reference [13], pushover tests were conducted on scaled models of ductile unit frames, directly braced by X steel braces. The objective of the study was to compare some seismic response and design parameters, including; load capacity, stiffness, toughness, ductility and
performance factor of different unbraced and X-braced RC frames in order that suitability of the X bracing of RC frames could be investigated. Model unit frames constructed for experimental investigations were 1:3 scaled models of a typical 3mx3m unit ductile frame. For the purpose of comparison, RC beams and columns in all the model frames (braced or unbraced) had identical dimensions and reinforcement detail. The corresponding horizontal loads estimated for the ultimate capacity of the model frame and the bracing systems were, 33 kN for the unbraced frame and 79 kN for the X-bracing system alone. In total four model frames were constructed so that the repeatability of the tests could be verified. Two model frames (F1-P and F2-P) were identical unbraced frames and two models (FB1 and FB2) were identical X-braced frames. A detailed account of test set-up and observations are given in [13].

![Figure 1. The pushover test of frame F1-P](image)

**Load Capacity, Stiffness and Toughness**

Over 3.5 fold increase in the lateral load capacity was achieved for the X-braced model frames tested in that study. Test results show that the load capacity of an existing ductile frame can be increased to the desired level by directly adding a bracing system to the frame, without the need for prior strengthening of the existing frame. This point is further substantiated when it is noted that the load corresponding to the appearance of the first plastic hinge in the ductile RC beam (10kN for the unbraced frame) increased by 90 kN when X-bracing was employed. Cross-bracing also appears to increase the initial stiffness of the RC frame. The increased stiffness due to bracing remains true at higher loads up to failure. The increased stiffness, together with the increased capacity, substantially increase the toughness of the braced frame compared to the unbraced frame. Toughness of the test frames, determined as the area under the pushover force-displacement curves shows a five-fold increase for the X-braced frame. This indicates the ability of the X-braced frame to absorb large energies.

**Ductility, Overstrength and Performance Factor**

Ductility, overstrength and performance factor parameters were determined for the four test model frames and are given in Table 1. The parameters given in this Table are presented in Figure 2, in which $V_x$, $V_y$ and $V_e$ are forces corresponding to the
first yield, structural yield and the elastic response, respectively and \( R_\mu = \frac{V_e}{V_y} \), \( R_s = \frac{V_y}{V_s} \) and \( R = R_\mu R_s \). It should be noted that the performance factor parameters listed in this Table are specific to the model frames tested and do not represent those of the full size frames. The Table indicates that when a ductile frame is braced, in return for the increase in strength, stiffness and toughness, ductility, overstrength and the performance factors are reduced.

![Figure 2. Pushover curve parameters](image)

**Table 1: Seismic response parameters of the test frames**

<table>
<thead>
<tr>
<th>Frame</th>
<th>( \Delta_y ) (mm)</th>
<th>( \Delta_{max} ) (mm)</th>
<th>( \mu )</th>
<th>( V_s ) (kN)</th>
<th>( V_y ) (kN)</th>
<th>( V_e ) (kN)</th>
<th>( R_\mu )</th>
<th>( R_s )</th>
<th>( R )</th>
</tr>
</thead>
<tbody>
<tr>
<td>F1-P</td>
<td>2.78</td>
<td>10.0</td>
<td>3.0</td>
<td>9.5</td>
<td>22.0</td>
<td>79.1</td>
<td>3.60</td>
<td>2.31</td>
<td>8.3</td>
</tr>
<tr>
<td>F2-P</td>
<td>2.65</td>
<td>10.0</td>
<td>3.8</td>
<td>10.0</td>
<td>21.0</td>
<td>79.5</td>
<td>3.77</td>
<td>2.10</td>
<td>7.9</td>
</tr>
<tr>
<td>FB1</td>
<td>6.41</td>
<td>10.0</td>
<td>1.6</td>
<td>60.0</td>
<td>74.0</td>
<td>115.4</td>
<td>1.56</td>
<td>1.23</td>
<td>1.9</td>
</tr>
<tr>
<td>FB2</td>
<td>6.45</td>
<td>10.0</td>
<td>1.5</td>
<td>59.5</td>
<td>75.0</td>
<td>116.2</td>
<td>1.55</td>
<td>1.26</td>
<td>1.9</td>
</tr>
</tbody>
</table>

The test results lead us to conclude that X-bracing is more suitable for a strength-based design. However, the relatively small post-yield capacity and the somewhat brittle failure mode of the X-braced frame make this system unfavourable for a ductile design.

### 2.2. Cyclic Tests

In another experimental study, cyclic tests were carried out on half-scale RC unit frames braced with X-bracing. Details of the test set-up and results are reported elsewhere [18]. Unit frames where selected from the third floor of a three-bayed, four-storey frame of a residential building. Two lateral load resisting systems, namely; an RC moment frame and an X-braced RC frame, were considered for the study. The gravity and earthquake forces acting on these unit frames were determined in accordance with the Iranian seismic code [20] using the seismic force reduction factor for moment frames with moderate ductility. The size of the
test specimens was determined based on the available laboratory space and the equipment limits. A 2/5 scaled model, measuring 1.76 m by 1.36 m, was found to be satisfactory. The forces acting on the panels were also scaled down resulting in a lateral load of 22 kN and two vertical loads of 35 kN for the moment frame and the same lateral load of 22 kN and two vertical loads of 38.5 kN for the braced frames. One moment resisting RC frame model, namely F1 and two braced RC frame models, namely FX1 and FX2, were designed using the above gravity and lateral loads. The moment frame was designed according to ACI 318-02 [21] and its detailing was done in accordance with the ACI special provisions for seismic design. Reinforcement details for this frame are shown, on the left hand side, in Figure 3. AISC-LRFD [22] was used to design the brace members and their welded connections to the gusset plates. Reinforcement details for the braced frames are also shown on the right hand side of Figure 2. A double-angle brace cross-section, consisting of two 25×25×3.2 mm angles, giving a cross-sectional area of 300 mm², was chosen for the frame FX1 and a C 30×3.5 mm channel with a cross-sectional area of around 500 mm² was selected for the frame FX2 (Figure 3). The difference in the brace member cross-section, therefore, made the FX2 frame somewhat stronger than the FX1 frame.

Figure 3. Detailing of the moment RC frame (F1) and the braced RC frames (FX1 & FX2)
The model frames were subjected to gravity loads using two hydraulic jacks. For the cyclic test, the actuator was first pulled to a displacement, $d_1$, of 5 mm then pushed to the same displacement. The value of $d_1$ was increased in the following cycles by an increment of 5 mm. The behaviour of the test models was monitored by using electrical and mechanical instrumentations including: Load cells attached to the hydraulic jacks and the actuator to measure applied loads, Linear Voltage Differential Transformers (LVDTs) to measure the lateral deformations and electronic strain gauges to monitor local strains in the reinforcement bars as well as steel bracing elements.

**Hysteretic Response and Load Capacity**

Figure 4 shows details of crack patterns in frame F1. The hysteretic lateral load-drift curves for the three frames F1, FX1 and FX2 are also shown in Figure 5. For the moment frame F1, at a load of 37.5kN, yielding of the lower bars of the lower beam initiated the plastic response. Failure occurred by plastic hinging at the ends of the upper and lower beams at a load of 55kN. At a drift of 1.9%, corresponding to a lateral load of 105kN, yielding of the double-angle bracing member of the braced frame FX1 initiated the plastic response. A significant drop in the lateral load capacity was observed at a load of 140kN (drift of 4.0%). This was noted to be due to the buckling of brace members. Following this, the lateral load capacity was mainly provided by the RC frame, which failed when plastic hinges were formed at the ends of the lower and upper beams. In the frame FX2, the yielding occurred at a load of about 140kN. The lateral capacity of this frame was not however affected because the bracing members were still acting in the elastic range. Testing was continued to a load of 200kN, which was the loading capacity of the actuator and subsequently the test was terminated. A summary of the yield loads and the maximum sustainable loads and their corresponding displacement ratios for the three tested frames are presented in Table 2.
Figure 5. Lateral load-drift hysteresis of frames (a) F1, (b) FX1 and (c) FX2

Table 2: The yield and ultimate strength capacities and their corresponding displacements

<table>
<thead>
<tr>
<th>Frame</th>
<th>Yield strength (kN)</th>
<th>Yield displacement (%)</th>
<th>Ultimate strength (kN)</th>
<th>Displacement at ultimate strength (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>F1</td>
<td>37.5</td>
<td>1.5</td>
<td>55</td>
<td>4.6</td>
</tr>
<tr>
<td>FX1</td>
<td>105</td>
<td>1.9</td>
<td>140</td>
<td>4.0</td>
</tr>
<tr>
<td>FX2</td>
<td>140</td>
<td>2.8</td>
<td>200</td>
<td>3.9</td>
</tr>
</tbody>
</table>

**Stiffness Degradation**

The lateral stiffness was calculated as the slope of the line joining the peak of positive and negative loads at a given cycle. The lateral stiffness is an index of the response of the frame from one cycle to the following cycle. Figure 6 illustrates a plot of the lateral stiffness for the three tested frames. Before buckling of the compressive brace, the diagram shows that the lateral stiffness of the frame FX1 was more than double that of the frame F1 and that the rate of stiffness degradation for both systems was almost equal. However, after buckling of the compressive brace, the lateral stiffness of the frame FX1 dropped and became comparable to that of the moment frame (Figure 6). Also, the FX2 frame, having more robust bracing members compared to the frame FX1, shows higher hysteretic stiffness compared to the later. However, both frames show a similar rate of stiffness degradation.
Energy Dissipation Capacity (Toughness)
The energy dissipated by the three tested frames during the cyclic load testing was calculated as the area enclosed by each hysteretic loop. Figure 7 shows a plot of the energy dissipated during a load cycle versus the lateral drift. Also the energy dissipated by each test frame after a number of selected cycles is presented in Table 3. It is observed that at low drift levels, the energy dissipated by the frames FX1 and FX2 was comparable with that of the frame F1. At higher levels of drift, it is clear that the energy dissipated by the braced frames is much higher than that by the moment frame. This proves that the overall seismic performance of the braced frames regarding capacity, stiffness and toughness is expected to be superior to that of the moment frame. This was also deduced from the results of the pushover tests presented earlier.

![Figure 6. Degradation of the lateral stiffness of test frames](image)

![Figure 7. Variation of energy dissipation with the applied displacement](image)

Table 3: Energy dissipation capacity of the test frames

<table>
<thead>
<tr>
<th>Frame</th>
<th>Cumulative energy dissipated (kN.mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Cycle 5</td>
</tr>
<tr>
<td>F1</td>
<td>600</td>
</tr>
<tr>
<td>FX1</td>
<td>451</td>
</tr>
<tr>
<td>FX2</td>
<td>570</td>
</tr>
</tbody>
</table>

Ductility
In these tests, ductility is measured both as the ratio of the displacement pertaining to the maximum force $\Delta_{\text{max}}$, to the displacement at yield $\Delta_y$ and as the ratio of the maximum displacement $\Delta_{\text{available}}$ to the displacement at yield point $\Delta_y$ of the model frames. These are calculated and shown in Table 4. As it was expected, the addition of X-bracing system somewhat reduces the ductility of a ductile frame, but the reduction in ductility does not affect the energy dissipation capacity of the frames.
Table 4: The ductility of the test frames

<table>
<thead>
<tr>
<th>Frame</th>
<th>Yield displacement at ultimate strength ($\Delta_\text{y}$) (mm)</th>
<th>Maximum available displacement ($\Delta_\text{available}$) (mm)</th>
<th>Ductility corresponding to $\Delta_\text{max}$</th>
<th>Ductility corresponding to $\Delta_\text{available}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>F1</td>
<td>18.0</td>
<td>68.0</td>
<td>3.1</td>
<td>3.8</td>
</tr>
<tr>
<td>FX1</td>
<td>22.5</td>
<td>62.5</td>
<td>2.1</td>
<td>2.8</td>
</tr>
<tr>
<td>FX2</td>
<td>33.0</td>
<td>-</td>
<td>1.4</td>
<td>-</td>
</tr>
</tbody>
</table>

2.3. Seismic Behaviour Factor

In forced-based seismic design procedures, behaviour factor, $R$ is a force reduction factor used to reduce the linear elastic response spectra to the inelastic response spectra. The behaviour factor, $R$, therefore accounts for the inherent ductility and overstrength of a structure and the difference in the level of stresses considered in its design. In another study carried out by Maheri and Akbari [14], the seismic behaviour factor ($R$) was evaluated for steel X-braced RC buildings. The $R$ factor components including ductility reduction factor and overstrength factor were extracted from inelastic pushover analyses of brace-frame systems of different heights and configurations. In that study 4-storey, 8-storey and 12-storey frames were considered. These are typical numbers of storeys used by some other investigators to cover low-rise to medium-rise framed buildings. All frames were three-bay wide with the central bay braced in the braced dual systems. DRAIN-2DX program was utilized to carry out nonlinear pushover analysis of each system. Inelastic pushover analysis of the multi-storey systems under investigation was carried out at horizontal load steps equal to 2% of the design capacity. A constant gravity load equal to total dead load plus 20% live load was also applied to each frame.

The effects of some parameters influencing the value of $R$ factor, including the height of the frame, share of bracing system from the applied load and the type of bracing system were investigated. Of the three variable parameters investigated, the number of storeys appears to be the predominant variable. The other variables, including the type of bracing system and the share of bracing from the applied load, have more localized influences and therefore do not warrant a similar generalisation. The significant effect of the number of storeys on $R$ factor of steel-braced RC frames, stems from the fact that shorter braced frames exhibit larger ductility than taller frames, therefore they possess higher ductility ‘capacity’. It was therefore found to be prudent to calculate the $R$ factors for the frames under consideration using specific ductility ‘demands’ of $\mu = 2$, $\mu = 3$, $\mu = 4$ and $\mu = 5$.

Based on the results obtained, tentative $R$ values for steel-braced intermediate ductility, moment resisting RC frame dual systems were presented as shown in Table 5. The proposed $R$ factors are given for different ductility demands that constitute the generally accepted range of ‘intermediate ductility’ response.

Table 5: Tentative values of $R$ factor for steel-braced, RC frame dual systems

<table>
<thead>
<tr>
<th>Ductility Demand</th>
<th>$\mu = 2$</th>
<th>$\mu = 3$</th>
<th>$\mu = 4$</th>
<th>$\mu = 5$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$R$</td>
<td>5.0</td>
<td>7.0</td>
<td>9.0</td>
<td>12.0</td>
</tr>
</tbody>
</table>
3. CONNECTION OVERSTRENGTH
An important consideration in the design of internally-braced RC frames with direct brace-frame connections is the level of interaction between the strength capacities of the RC frame and the bracing system. In this paper, results of experimental and numerical investigations aimed at investigating the causes and evaluating the level of this interaction are also discussed. Results of the three half-scale RC frames representing a moment frame (F1) with moderate ductility, and two braced frames (FX1 and FX2) already tested under cyclic loading are used for this purpose. These results are also used as basis for developing and calibrating numerical models of full-scale frames. Using the numerical models, a parametric investigation is carried out to determine the role of the main variable parameters affecting the level of capacity interaction between the RC frame and the bracing system.

3.1. Experimental Brace-Frame Capacity Interaction
To investigate the level of interaction in the tested model frames, the corresponding forces in the bracing systems alone were evaluated by considering the relevant test displacements on the diagonals. A simple bilinear model for steel, which accounts for cyclic effects, was assumed and used to represent the force-deflection envelop curve of bracing system alone. The envelop curve of the calculated force-drift relationship for the FX1 bracing system alone (marked as No. 2 in the Figure) is plotted in Figure 8. Also plotted in this Figure, for comparison, are the experimental envelop of the force-drift relationship of the moment frame alone, F1, (marked as No. 1 in the Figure) and the experimental envelop of the force-drift curves of the FX1 braced frame. To be able to gain an insight into the level of capacity interaction between different elements, the envelop curves of the bracing system alone (2) and the moment RC frame (1) are added together to obtain the sum strength capacity of the two elements as also are presented in Figure 4 ((1) + (2)). By comparing the sum strength capacity of the two constituent elements with the actual strength capacity of the braced frame, it is evident that the actual braced frame exhibits a larger capacity than the sum of the capacities of the two elements. This means that by adding a bracing system to an RC frame, the capacity of the RC frame is increased beyond the capacity of the bracing system. The capacity interaction for the frame FX1 is measured, as the minimum of all the evaluated values, as 8.5 percent. It should be noted that the dimensions and reinforcement details and therefore the flexural capacities of the RC frames in F1 and FX1 models are the same. This enables us to make a viable capacity interaction comparison as discussed above. Considering the experimental results, it is evident that the capacity interaction is an overstrength which can be attributed mainly to the effects of brace-frame connections in reducing the effective lengths of the RC beams and columns, hence increasing the stiffness and strength of the frame.

3.2. Numerical Evaluation Of Overstrength
To investigate the level of connection overstrength in full-scale X-braced RC frames, nonlinear pushover numerical analyses of the moment frame, braced
frames and the bracing systems were carried out. The OpenSEES (Open System for Earthquake Engineering Simulation) program was utilised to numerically model the frames. Details of the numerical models and the numerical analyses are given elsewhere [19]. The numerical models were calibrated and their accuracy ascertained by comparing the results of the nonlinear cyclic analysis of the moment frame F1 and the braced frame FX1 with the results obtained from their respective cyclic tests.

After calibrating the numerical models, a series of nonlinear pushover analyses were conducted on full scale 2-D frames of different heights and widths with different bracing configurations. These included frames, 4, 8 and 12 storeys high and 3, 6 and 9 bays wide. The number of braced bays in each frame was also made a function of the number of bays such that the three, six and nine-bay frames had, respectively, one, two and three bays braced. All frames consisted of 3m high and 5m wide unit frames. Another variable parameter in this investigation is the apportioned share of bracing system from the applied loading. Load shares of 30%, 50%, 80% and 100% for bracing system are considered. As it was mentioned earlier, the main factor contributing to the interaction is the effect of connections on reducing the effective lengths of beams and columns. Therefore, considering the nature of this interaction, a representing parameter can be introduced as the ratio of the effective stiffness of the RC frame with brace-frame connections ($K_r$) to the stiffness of the RC frame without the brace-frame connections ($K_i$) and designated as $\rho$. Considering that the connections reduce the effective lengths of RC beams and columns, the effective stiffness of the frame with brace-frame connections corresponds to the stiffness of a reduced frame as shown in Figure 9. For simplicity and conservatively, the reduced frame is assumed to have beams and columns of lengths equal to the distances between the centroids of the four gusset plates as seen in Figure 9. Also, for practical purposes, the parameter $\rho$ is calculated as the ratio of the linear stiffness of the reduced RC frame of a central floor ($K_r$) and the linear stiffness of the initial RC frame of the same floor ($K_i$), also shown in Figure 9.
The stiffness ratio, $\rho$, as described above was calculated for all the frames analysed. The overstrength factors, $R$, previously determined for these frames with different problem variables were plotted against the stiffness ratio for different frame geometries considered. To condense the results of the 9 relations thus obtained, the linear relation for the 4-storey, 3-bay frame is considered as the base overstrength, $R_b$, and the effects of the two main variable parameters including the number of braced bays (number of bays in the frame) and the number of storeys are considered respectively as correction factors $\alpha$ and $\beta$. Therefore;

$$R = \alpha \beta R_b \text{ (\%)}$$  \hspace{1cm} (1) \\

where, \\

$$R_b = 32 \rho - 27$$

In order that quantitative relations can be drawn between the factors $\alpha$ and $\beta$ and the stiffness ratio $\rho$, the former parameters are plotted against the latter in Figure 10.a and Figure 10.b, respectively. Noting the near linear variation of $\alpha$ against $\rho$ the following relations can be presented for this correction factor;

$$\begin{align*}
\alpha &= 0.16m + 0.84, \text{ for } 0.0 < \rho \leq 1.0 \\
\alpha &= 0.09m + 0.91, \text{ for } 1.0 < \rho \leq 1.25 \\
\alpha &= 0.06m + 0.94, \text{ for } 1.25 < \rho \leq 1.40
\end{align*}$$  \hspace{1cm} (2)

Also, as the variation of $\beta$ with $\rho$ is small, this correction factor can be presented independent of the stiffness ratio in the following form;

$$\beta = 0.0425n + 0.84$$  \hspace{1cm} (3)

In equations (2) and (3), $m$ and $n$ are the number of braced bays and the number of storeys, respectively.
4. FORCE-RELEASE DEVICES

4.1. Knee Bracing

Knee bracing is used in steel construction to increase the ductility and to increase the seismic performance of the frames. Parallel to the work carried out on the pushover tests of model frames F1-P, F2-P, FB1 and FB2 described in section 2.1, two identical RC frames braced with knee-bracing system were also constructed. The RC frames of these models were identical to the unbraced and X-braced frames and the brace dimensions were also identical to the bracing system of the X-braced frames; the only difference being the four knee elements used at the ends of the diagonal bracing. Details of the bracing system and test set-up and the test results are given elsewhere [13]. The object of the tests was to investigate the role of knee bracing in increasing the ductility of the dual system while maintaining the strength and stiffness requirements. Tests similar to that described in 2.1 were conducted on these frames (Figure 11). The ultimate capacities of the knee-braced frames were found to be 2.5 times that of the unbraced frame. In Figure 12 a comparison is made between the three unbraced, X-braced and knee-braced frames regarding their stiffness and toughness. It is evident that the knee bracing has enabled the frame to possess considerable capacity and stiffness with good capacity to absorb energy. By extracting the ductility ratio from the pushover curves of knee-braced frames as around $\mu = 2.2$, it becomes evident that knee-bracing has also substantially increased the frames ductility compared to the ductility of the X-braced frames ($\mu = 1.5$).
4.2. Compression Release Tool

In this section another force release tool is presented and its performance is evaluated. This novel tool, named a 'compression release tool' (CRT) when installed in a brace member, releases its compressive force. The proposed CRT is shown in Figure 13. It is composed of two steel plates separated by a gap. The two plates are to be attached together with a maximum of four bars. A cylindrical steel pipe (cylinder) is attached to one of the plates. A steel rod (piston) is attached to the second plate. The cylinder is padded with rubber material. A typical brace member can be divided into two pieces; each is to be welded to one of the CRT steel plates. When this member is subjected to a compressive displacement, the piston will slide inside the cylinder and thus the member will not have any compressive stresses. When it is subjected to a tensile displacement, the bars will transfer the tensile force between the two brace pieces. The bars should be chosen such that the sum of their yield resistances is less than the yield resistance of the brace member. Following a strong earthquake, the brace member is expected to be easily retrofitted by replacing the bars.

Parallel to the experimental work carried out on the X-braced model frames, constructed for cyclic loading as described above, an experimental study was also conducted to evaluate the effectiveness of the CRT. Two, similar half scale RC frames were constructed and the CRT installed. The CRT can be installed anywhere along the brace member. For the tested specimens, it was decided to install the CRT at the location shown in Figure 13. The size of the steel plates in the CRT was chosen to be $120 \times 120 \times 10$ mm. The expected axial deformation in the brace members were calculated and based on that it was decided that a 135 mm gap between the steel plates of the CRT is required. To create this gap, the length of the cylinder and the piston were chosen to be 135 mm. The inner diameter and wall thickness of the cylinder were chosen to be 40 mm and 5 mm, respectively. The piston was chosen to be 35 mm steel rod. The bars connecting the steel plates were different in specimen FXS1 than those in specimen FXS2. They were two-12.7 mm
and two-16mm steel bars in specimens FXS1 and FXS2, respectively. Tensile load tests on the steel rods revealed that their yield stress is 350 MPa. A photo of an installed CRT is shown in Figure 13. Details of the test specimens and test set-up and results are given by Ghaffarzadeh and Maheri [17].

The frames with CRT (FXS1 and FXS2) were tested under cyclic loading the same way as the moment frames (F1 and F2) and the X-braced frames (FX1 and FX2). The seismic parameters evaluated from the test results include; stiffness degradation, energy dissipation capacity (toughness) and ductility. A discussion of the test results is given as follows:

The lateral load-deformation response for specimen FXS1 indicates the formation of first plastic hinge at a drift level of 1.2%. This was due to the yielding of the two-12.7mm steel bars joining the steel plates of the CRT. This happened at the lateral load of 75kN. The frame failed at the drift of 4.8% corresponding to lateral load of 182kN due to tensile failure of the two-12.7 mm bars. The behaviour of specimen FXS2 was similar to that of specimen FXS1. Yielding of two-16 mm steel bars in the CRT occurred at a drift of 2.5% (lateral load of 140kN). By increasing drift, cracks became visible. Strains in the top reinforcement of the top beam indicate that steel yielded at a drift of 3.4%. The test was terminated because of localized concrete failure in the vicinity of the supports.

**Stiffness Degradation**

The initial stiffness of specimens FX1 and FX2 was higher than that of the specimens FXS1 and FXS2. This is a direct result from the lower elastic stiffness of bracing members equipped with CRT. The steeper degradation in the lateral stiffness observed in specimens FX1 and FX2 however indicates that using the CRT minimized the cracking in the RC frame and kept the lateral stiffness of the frame almost constant.

**Energy Dissipation Capacity (Toughness)**

The cumulative energy dissipated by the frames after 5, 10, 15, 20 and 25 cycles were also calculated. It was noted that, at lower displacements, the energy dissipated by the braced frames with the CRT (specimens FXS1 and FXS2) is somewhat less than that of braced frames without the CRT (specimens FX1 and FX2). With increasing displacements and as the bars in CRT yield, the energy dissipated by the frames with CRT is increased to levels higher than those of the frames without the CRT. This indicates that the installation of the CRT did not, by and large, affect the energy dissipation capacity of the braced frames.

**Ductility**

The available ductility of the four specimens is given in Table 7. It can be observed in this Table that the overall behaviour of the specimen with CRT (specimen FXS1) is more ductile in comparison with specimen FX1 without CRT. The sudden drop in load-drift response curve of specimen FX1 after buckling of compression brace indicates a brittle behaviour. However, in specimen FXS1, in which buckling is inhibited and failure happens by yielding of steel bars of CRT,
the behaviour is evidently more ductile (almost two folds). This shows the effectiveness of the CRT in increasing the ductility of the braced frame. By comparing the results of the stronger braced frames without CRT (specimen FX2) and with CRT (specimen FXS2), the favourable effect of the CRT on the ductility of the frame can also be noted.

<table>
<thead>
<tr>
<th>Test Specimen</th>
<th>$\Delta_h$ (mm)</th>
<th>$\Delta_{max}$ (mm)</th>
<th>$\mu$</th>
<th>$V_i$ (kN)</th>
<th>$V_y$ (kN)</th>
<th>$V_e$ (kN)</th>
<th>$R_{\mu}$</th>
<th>$R_s$</th>
<th>$R$</th>
</tr>
</thead>
<tbody>
<tr>
<td>FX1</td>
<td>22.5</td>
<td>47.5</td>
<td>2.11</td>
<td>105</td>
<td>112</td>
<td>284</td>
<td>2.53</td>
<td>1.06</td>
<td>2.68</td>
</tr>
<tr>
<td>FX2</td>
<td>33.0</td>
<td>---</td>
<td>---</td>
<td>150</td>
<td>168</td>
<td>352</td>
<td>2.09</td>
<td>1.12</td>
<td>2.34</td>
</tr>
<tr>
<td>FXS1</td>
<td>17.5</td>
<td>71.5</td>
<td>4.08</td>
<td>75</td>
<td>118</td>
<td>296</td>
<td>2.51</td>
<td>1.57</td>
<td>3.94</td>
</tr>
<tr>
<td>FXS2</td>
<td>35.0</td>
<td>---</td>
<td>---</td>
<td>134</td>
<td>160</td>
<td>324</td>
<td>2.03</td>
<td>1.19</td>
<td>2.42</td>
</tr>
</tbody>
</table>

5. CONCLUSIONS
The results of the experimental and numerical investigations presented in this paper lead us to the following conclusions;
1. Internal bracing of RC frames with direct brace-frame connections is not only suitable for seismic retrofitting of existing building but it can also be used as a viable alternative to shear walls as shear resisting elements for the newly constructed buildings.
2. X-bracing is more suitable for a strength-based design. However, the relatively small post-yield capacity and the somewhat brittle failure mode of the X-braced frame make this system less favourable for a ductile design.
3. The proposed CRT can be effectively used in steel bracing systems to eliminate buckling failure. Its use will also result in an adequate energy dissipation capacity for the brace-frame system.
4. The inclusion of CRT can also greatly enhance the ductility of the braced frame. The desired level of ductility can be achieved by appropriate design of the CRT bars.
5. To increase the ductility and maintain the strength and stiffness capacities of the braced frames, Knee bracing of the frame or using CRT on the brace members is recommended. Such systems can be successfully utilised to design for both the damage-level and collapse-level earthquakes for which the damage level may be considered as the yield capacity of the knee elements.
6. The overstrength in a braced RC frame is due to the stiffening effects of connections. This overstrength is termed the capacity interaction or connection overstrength. It is significant and needs to be considered in design.
7. Presentation of the connection overstrength in the form of a frame stiffness ratio, $\rho$, enable us to use the results and formulations presented here for other types of concentric and eccentric bracing systems.

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18. Ghaffarzadeh, H. Maheri, MR, "Cyclic tests on the internally braced RC
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SUSTAINABLE DEVELOPMENT IN CEMENT AND CONCRETE

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Concrete Technology and Durability Research Center of Amirkabir University

ABSTRACT
Cement is the key ingredient in making concrete. Concrete is the second consumed materials after water in the world. When a material becomes as integral to the structure as concrete, it is important to analyze its environmental impacts to conclude if the material is as sustainable as it is prevalent.
In this paper the role of cement and concrete in sustainable development is discussed. The use of bio-fuels and alternative raw materials can reduce the CO$_2$ emission in cement production. Supplementary cementing materials are new widely used for making durable concretes and reducing the CO$_2$ emission. Exploiting the thermal mass of concrete to create energy-optimized solutions for heating and cooling residential and office buildings is discussed. Finally the production of recycled aggregate concrete from old concrete structures can have a major environmental impact in the future programs for sustainable development.

Keywords: sustainable development, concrete, cement, durability, CO$_2$ emission

1. INTRODUCTION
It is impossible to walk through cities without seeing concrete in some form. Whether it is in the latest high rise being constructed, new sidewalks being cured, in roads connecting the city, in dams, bridges, marine structures, industrial plants, etc. concrete is inescapable. When a material becomes as integral to the structure as concrete, it is important to analyze its environmental impacts to conclude if the material is as sustainable as it is prevalent. If the material does not satisfy the credential of sustainability it should be further developed, especially in present society when environmentally detrimental processes are currently subject to scrutiny.
It is often debated whether concrete can or should be considered to be a sustainable option due to its particular properties and characteristics. Concrete is made of several different elements. In its simplest form this includes cement, water, and aggregate. Cement requires substantial amounts of energy to produce and releases large amounts of carbon dioxide. However, it can also be replaced in part by supplementary cementing materials.
Concrete has a long service life; buildings made of concrete can usually be expected to last hundreds of years with proper maintenance. Since the structural lifetime is so long, potential waste if another type of building material was used is reduced. As well, once a building requires demolition, its material can be used for subsequent buildings. Truly, when a structure reaches the end of its useful life, its
concrete component can be completely recycled into the aggregates to be used in other concrete mixtures. In spite of the amount of initial energy required to produce concrete, the material has the potential to be efficient over its estimated life expectancy. A list of credentials that should addressed when examining the sustainability of a material is provided:

- Energy required to produce the material
- CO₂ emissions resulting from the material’s manufacture
- Toxicity of the material
- Transportation of the material during its manufacturing and delivery
- Degree of pollution resulting from the material at the end of its useful life
- Maintenance required and the materials required for maintenance
- Lifetime of the material and its potential for reuse if the building is demolished

It is important to address all of these factors in deciding if a material can be viewed wholly as a sustainable material. These factors can be separated into two main components, embodied energy is low, but the operational energy is high then a material cannot be deemed sustainable. An ideal solution would be a material with a low embodied energy which results in high operational energy savings. If these two factors can be satisfied then a material, when used with sustainability in mind, is well on its way to reducing its environmental impact.

2. EMBODIEND ENERGY

Embodied energy is the amount of energy required to produce a material. This includes the energy required for the raw material extraction; the energy required to process and manufacture the material; and transportation for all stages of production. It is impossible to assign a value to the embodied energy of concrete on a whole because mix designs vary widely which subsequently changing the embodied energy. Table 1 provides a summary of different elements found in concrete and the approximated values of the relative embodied energy.

<table>
<thead>
<tr>
<th>Source Material</th>
<th>Embodied Energy (GJ/tonne)</th>
</tr>
</thead>
<tbody>
<tr>
<td>&quot;The New Ecological Home&quot; Poured on site concrete</td>
<td>1.0–1.6</td>
</tr>
<tr>
<td>Cement</td>
<td>7–8</td>
</tr>
<tr>
<td>&quot;Ecohouse 2: A Design Guide&quot; Cement Natural aggregates</td>
<td>4.3–7.8</td>
</tr>
<tr>
<td>Cement Association of Canada Clinker</td>
<td>3.0–6.0</td>
</tr>
</tbody>
</table>

Although there is variation in the values a general idea about the embodied energy of cement can be established. From the table it can be estimated that 7 GJ of energy is required to produce one tonne of cement. The mix used to produce concrete can evidently have a large effect on the amount of embodied energy required. In general, however, the magnitude of this parameter can be understood as well as the
importance of attempting to minimize it. The process required to manufacture cement is fairly consistent regardless of the plant type. Conversely, the transportation component and the individual plant part efficiencies can differ and thus have room for improvement. Therefore, the embodied energy can be assumed to be a realistic estimation of the actual embodied energy. Accordingly, the result of a large embodied energy quantity is a significant production of carbon dioxide. Both of these components of concrete are of concern because they are unsustainable. It must be concluded then that the manufacturing process of cement must be developed further to increase the sustainability of cement and thereby increase the sustainability of the concrete itself. There are two obvious ways in which to reduce the impact that concrete has on the environment. The first is to increase the efficiency during production, and the second is to reduce the amount of cement in the concrete mix. In combination, these two ideas could become a powerful means to creating a versatile material that is practical, beneficial and sustainable in our society.

3. CONCRETE PRODUCTION
To thoroughly assess sustainability and the implications of embodied energy, it is important to have a sufficient comprehension of the processes involved. Possessing this understanding gives depth to the concept of embodied energy, which aids in finding opportunities for improvement, as well as a concept of the environmental impacts. Cement has been marked as a possible threat to the sustainability of concrete. By delving into the nature of cement, this threat can be realized and addressed. The missions resulting from the calcination process cannot be reduced, and thus the CO\textsubscript{2} emissions must be reduced by other means. Improvements to kiln efficiencies; alternate fuel methods; and reduction in the cement quality requirement though intelligent mix design, can all work to aid the reduction of embodied energy and CO\textsubscript{2} emissions.

4. REDUCING ENERGY & EMISSIONS
To reduce the energy-intensity and pollution levels, change must occur on all levels of the concrete process. In Germany, effort was made to decrease the negative environmental effects of the industry by improving the manufacturing process. This was achieved by installing new plants for the purpose of ensuring that the kiln production operation was smoother and energy requirements were decreased. Besides, the existing kilns were optimized in a way that required minimum fan power. A waste heat recovery process was devised to utilize remaining energy losses of the new cement kilns. Finally direct electrical power requirements were decreased by improving the efficiency of the grinding system. Once the manufacturing process was optimized a greater amount of waste fuels was substituted for fossil fuels and alternatives to the cement composition via inclusion of blended cements were derived. Finally, it was admitted that greater research and development was required in the field of “process technologies, use of secondary materials, properties and application of blended cements”.

Table 2 shows the estimated global cement consumption and CO\textsubscript{2} emission in the
year 2020.

Table 2: Estimates of global cement consumption and CO₂ emission attributable to clinker production in the year 2020, million tonnes.

<table>
<thead>
<tr>
<th>Options</th>
<th>No.1 Business as usual</th>
<th>No.2 Challenging option</th>
<th>No.3 Formidable option</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement Consumption/ production</td>
<td>3,500</td>
<td>2,800</td>
<td>2,100</td>
</tr>
<tr>
<td>Complementary Cementing Materials</td>
<td>700</td>
<td>840</td>
<td>1,050</td>
</tr>
<tr>
<td>Portland Clinker Requirement</td>
<td>2,800</td>
<td>1,960</td>
<td>1,050</td>
</tr>
<tr>
<td>CO₂ Emissions (0.9 T/T clinker)</td>
<td>2,520</td>
<td>1,760</td>
<td>945</td>
</tr>
<tr>
<td>%CO₂ increased from the 1990 level</td>
<td>270</td>
<td>190</td>
<td>0</td>
</tr>
</tbody>
</table>

- Cement consumption goes up by about 50% of the 2005 level and the use of CCM increased to 20% of the total cementing material.
- Cement consumption goes down by 20% of the BAU level, and CCM increased to 30% of the total cementing material.
- Cement consumption goes down by 40% of the B.A.U. level, and the use of CCM is increased to 50% of the total cementing material (see Table 3).

Table 3: Projects for CO₂ emission in cement industry

<table>
<thead>
<tr>
<th>Joint Projects</th>
<th>Individual Companies</th>
</tr>
</thead>
<tbody>
<tr>
<td>The cement sustainability initiative intends to create joint projects to:</td>
<td>As part of commitment to good practice and innovation in sustainable development, companies agree to:</td>
</tr>
<tr>
<td><strong>Climate Protection</strong></td>
<td></td>
</tr>
<tr>
<td>Develop a Carbon Dioxide (CO₂) protocol for the cement industry. (project already delivered)</td>
<td>Use the tools set out in the CO₂ protocol to define and make public their baseline emissions.</td>
</tr>
<tr>
<td>Work with WBCSD/World resources Institute (WRI) and other organization to investigate public policy and market mechanisms for reducing CO₂ emissions.</td>
<td>Develop a climate change mitigation strategy, and publish targets and progress by 2006.</td>
</tr>
<tr>
<td></td>
<td>Report annually on CO₂ emissions in line with the protocol.</td>
</tr>
<tr>
<td><strong>Fuels and Raw Materials</strong></td>
<td>Apply the guidelines developed for fuel and raw material use.</td>
</tr>
<tr>
<td>Develop a set of guidelines for the responsible use of conventional and alternative fuels and raw materials in cement kilns.</td>
<td></td>
</tr>
<tr>
<td><strong>Emissions Reduction</strong></td>
<td>Set emissions targets on relevant materials and report publicly on progress.</td>
</tr>
<tr>
<td>Develop an industry protocol for measurement, monitoring and reporting of emissions, and find solutions to more readily assess emissions of substance such as dioxins and volatile organic compounds.</td>
<td>Apply the protocol for measurement, monitoring and reporting of emissions.</td>
</tr>
<tr>
<td>Make emissions data publicly available and accessible to stakeholders by 2006.</td>
<td></td>
</tr>
<tr>
<td>Set emissions targets on relevant materials and report publicly on progress.</td>
<td></td>
</tr>
</tbody>
</table>
5. SUPPLEMENTARY CEMENTING MATERIALS & SUSTAINABILITY

When concrete is the material of choice one can look forward to having endless options and opportunities in its composition. There are many types of cement, admixtures, aggregate, and supplementary cementing materials that can be incorporated in different quantities. By incorporating a higher quantity of supplementary cementing materials the amount of cement can be reduced, lowering the emissions and energy with a mix.

Supplementary cementing materials have proven to be economical environmental alternatives to typical concrete mixes. Fly ash and silica fume in particular have allowed for significant reductions in CO₂ emissions in cement production. This was possible because of the decreased amount of clinker used to produce the cement and concretes as well as decreasing the requirement of fuel for clinker burning. CO₂ emissions could also be decreased with the increase in the efficiency of clinker in concrete strength development. This can be done by including mineral admixtures to allow for a decrease in the water requirement with fly ash use and strength development by using silica fume. By incorporating 7.5 percent of silica fume to cement during the grinding process, an increase in efficiency of 25% resulted in the clinker.

Another factor that can be manipulated using mix design is the durability. Increased durability decreases maintenance as well as increases lifetime. Longevity can be increased with a reduction in the heat of hydration; reduced porosity. Most widely perhaps, fly ash has been accepted as a great supplement to 100 percent cement use. It improves the most critical cement characteristics such as workability, impermeability and durability. However, regardless of fly ash’s positive affect on cement production there have been barriers associated with its
use. However, new technologies have been developed as well as newer cement formulations which overcome these previous barriers. Other alternatives have been used such as superplasticizers for water reduction, and silica fume or rice husk ash for porosity minimization and lengthening.

Rice husk ash (RHA) is a byproduct of burning rice husks. In 1982 about 406.6 million tonnes of paddy was produced on a global basis, which represents about 81 million tonnes of husk or 16 million tonnes of ash if an ash content of 20% is assumed. The available ash for cement is approx 4.5 million tonnes. The husk are incinerated and an ash that is predominantly silica is residual. RHA is highly pozzolanic due to its extremely high surface area (50,000 to 100,000 m²/kg). When 5% to 15% RHA is incorporated by mass higher compressive strengths, decreased permeability, resistance to sulfate and acid attack, and resistance to chloride penetration can be expected. By adding in RHA the durability and recycled content on the mix can be increased.

Natural pozzolans are also viewed as effective substitutes for cement. However, it has been observed that the substitution rate, a less workable mortar was produced. Thus, a decrease in compressive strength was witnessed. Yet, as the level of fineness is increased, the comparable strength increases as well. This is most apparent when natural pozzolan substitutions are high. Therefore, using natural pozzolans proves to be an optimal substitute for cement as it provides equivalent strength and reduces the per unit emissions of greenhouse gases.

6. DURABILITY

The durability of a product can have one of the most significant influences on the environment. The less durable a product, the shorter the service life is. This results in new purchases or repairs which is a wasteful practice especially when the industry is as energy intense as cement production. Therefore the longer the service life is the better earth’s natural resources are conserved. The challenge then is to produce concrete that is highly durable, and a high-performance building material for future structure.

The major causes of reinforced concrete deterioration in structures are “corrosion of the reinforcing steel, exposure to cycles of freezing and thawing, alkali-silica reaction and sulfate attack”. The mechanism of concrete expansion and cracking is highly dependent on a high degree of water saturation. Therefore it is evident that water-tightness of concrete is a crucial step to ensure minimal damage. Thus this suggests that the soundness of concrete is closely related to its durability.

Early cracking in concrete is usually a function of either thermal contraction and/or drying shrinkage. The construction industry is driven by economics and thus the mentality of the faster it is done the more profit earned encourages cements with high-early strength, or rather, concrete with high levels of Portland cement to be used. These concretes have low cracking resistances as a result of an increase in “the shrinkage, and elastic modulus on one hand, and a reduction in the creep coefficient on the other hand”. As such, this explains the high level of vulnerability to cracking of high-early-strength concretes to that of moderate or low-strength concrete mixtures.
Early cracking is commonly minimized by incorporation excessive steel reinforcement. However, this solution simply replaces a small number of wide cracks with a numerous number of invisible and often unmeasurable micro-cracks. This does not in any sense constitute as a sufficient solution for durability.

If concrete is properly consolidated and cured, it will remain watertight unless the pores and cracks within it form interconnected pathways to the surface surrendering the concrete to further deterioration. Evidently, however, the drive of economics does not allow for this. As such, when thermal cracking and durability are of primary concern, it has been shown that supplementary cementing materials (SCM) should be incorporated into the mix design as it will prove to be most cost-effective. The reasoning for this is the property of concrete mixes that contain SCM to have stronger transition zones and are thus less prone to microcracking. Ultimately, by incorporating byproducts such as fly ash or slag into mix design, the durability of concrete is augmented through prolonged watertightness.

The most readily available mineral additive for cement is limestone. In Europe, more limestone is used in Portland-based cements than all other mineral additions combined, notably in the European CEM II L class (24.6% of all European cement manufactured in 2003) and to a lesser extent in the M class, and as minor addition of up to 5% in almost all other Portland cements. It has been shown that much of the alumina from calcium carbo-aluminate hydrates, which can result in a significant decrease in porosity [16]. Both the European cement standard, EN 197-1, and ASTM C150 allow up to 5% limestone [18]. Limestone added in excess of this amount, although constituting essentially a "filler", can also act as an accelerator for alite hydration, so that, with suitable grinding techniques, cement strength up to 28 days are often not much reduced even at limestone contents as high as 20%. In addition to this, Limestone additions can improve concrete consistency by reducing cement water demand, and provided that a low w/c concrete mix design is used, high limestone replacement as some pure Portland cements [19].

7. RECYCLED AGGREGATE

When concrete structures reach the end of their useful lives, disposal is not the only available next step. Concrete can be crushed and used as recycled aggregate. Much research has gone into determining whether the properties of used aggregates are sufficient for reuse in concrete. It has been found that due to the suctioning behavior of recycled aggregates, water addition is a problem of major concern. The difficulty arises in determining the appropriate proportions of water to aggregate as it is required in higher quantities when using recycled aggregate to that of dense aggregate. Although the elastic modulus continued to increase for the first few days, it stabilized at approximately 7 days. In conclusion, processed building rubble is an adequate source for recycled aggregate that can produce concrete of sufficient strength and durability. The recycled material can be available at good quality without an unacceptable level of harmful impurities. Specifically, the quality should be assessed based on porosity which directly affects the performance of concrete.
In consideration of the environmental performance of materials, one needs to consider effects taking place during the entire life cycle of the material. It has been found that cement-based materials can permanently absorb CO₂ from the atmosphere. This process is termed carbonation and occurs during the normal service life of a concrete structure and also after demolition. On a geological time frame, the cement in hardened concrete will bind approximately the same amount of CO₂ as was originally liberated by the calcination of its raw materials (mainly limestone) in the cement kiln.

However, the impact that concrete carbonation has in the assessment of overall CO₂ emissions from cement manufacture is generally overlooked, due to the difficulty in estimating its rate. Depending on the concrete composition, the type of concrete structure, and the environment to which the concrete is exposed, total carbonation will take place over years therefore it is necessary to analyze the factors affecting the rate of carbonation. This is difficult to do in a precise manner, and the environmental benefit of this effect is still open to debate. However, a recent Nordic study points to that concrete recycling, in which the concrete is crushed, unexpectedly may lead to significant CO₂ uptake. The significance of these results is still controversial and is under discussion.

The Nordic study points to an opportunity to improve the environmental performance of concrete over its life cycle by enhancing carbonation when this has no negative durability effects. Most effectively, promoting concrete recycling and adapting recycling practices for optimal CO₂ uptake would have a positive environmental benefit.

8. OPERATIONAL ENERGY
As important as it is to reduce the embodied energy and emissions, it is just as important that when implementing a material that the energy requirements during its useful life are not increased as a byproduct of material selection. Concrete offers solutions to reduce the operational energy of structures such as buildings, dams, and roads.

BUILDINGS
Concrete can aid in lowering the operational costs of a building because it possesses thermal mass. Thermal mass is material property that stores and slowly releases energy. Materials that have significant thermal mass possess the following qualities:

- High specific heat
- High density
- Low (but not extremely low) thermal conductivity

Concrete is an example of material with high thermal mass. Thermal masses absorb and store energy when their temperatures are below air temperature. The stores energy is later slowly released when the air temperature drops below the temperature of the mass. The main advantages of thermal masses are:
1. There are fewer spikes in the heating and cooling requirements, since mass slows the response time and moderates indoor temperature fluctuations.
2. A massive building uses less energy than a similar low mass building due to the reduced heat transfer through the massive elements.

3. Thermal mass can shift energy demand to off-peak time periods when utility rates are lower. Since power plants are designed to provide power at peak loads, shifting the peak load can reduce the number of power plants required. Thermal masses can also be used to passively heat a building. This works especially well because solar gain increases during the winter, due to the low angle of inclination of the sun vertical walls are exposed to more solar energy. “Buildings with exterior concrete walls, also called mass walls, utilize less energy to heat and cool than similarly insulated buildings with wood or steel frame walls”. During the day, while the temperature is warmer and the sun is shining, the walls store solar energy. When the temperature drops the thermal energy radiates to warm the house and reduce heating loads. Concern may arise that this effect will occur as well in the summer and heat the house when heat is unwanted. Walls can be sheltered from solar radiation during the summer using overhangs. If the wall temperature can be kept below the interior temperature, the thermal mass will absorb energy from the air, cooling the space. Reducing the amount of addition energy required to heat/cool a building.

Concrete’s thermal mass also makes it possible to involve developments such as radiant heating. This development is a method of heating through radiant heat as opposed to convection heating. Where convective heating warms air and circulates the warm air through the building radiant heat warms materials, and the materials radiate the heat into the space. This method works well when using materials with significant thermal mass because they comfortably release the thermal energy into a room, as well as stores any excess. Radiant heating still uses energy to warm the water, but this energy is significantly less than the energy required, and wasted through convective heating. When heating a building through a typical HVAC the system, the warm air always rises to ceiling of the room, where it is lost, or wasted most people do not benefit from heat that is half a meter above their head. Radiant heating heats habitable space first and foremost, reducing the heating load.

Other developments that can aid in reducing the operational energy include implementing insulating concrete forms (ICFs). This product combines the form work and the insulation for a wall assembly into one, creating less waste and a more consistent air barrier with fewer thermal breaks. By creating a concrete wall sandwiched between insulation a thermally efficient wall is created. The resistance of insulation in addition to the thermal mass of concrete creates a wall where temperature changes are gradual due to the thermal mass, and they are small because of the insulation and building envelope continuity. By eliminating wood or metal studs thermal breaks are reduced, these points where thermal energy is generally lost are eliminated. The elimination of these materials reduces the strain on non-renewable source, like timber, and high energy materials such as steel. This further reduces the environmental impact of a new construction.

**DAMS**

Pozzolans are known scientifically to be both an environmental beneficial
substitute in cement production as well as an economically feasible solution. Since the 1980’s, roller-compacted concrete dams (RCC) have been known to be one of the most rapid and economical method for construction of medium-height dams. By 1992, ninety-six RCC dams were built in over seventeen countries. 85% of these dams included pozzolans in the mix design. In fact, high paste content RCC most commonly uses 250 kg/m$^3$ of concrete with 70-80% pozzolan addition. Of the 85% of dams which incorporated pozzolans in the mix design, fly ash was incorporated in 90%. This quantity hints at the capability pozzolans have in cement/concrete production and the effect it can have on the environment when waste products become commodities.

ROADS
The use of recycled concrete aggregate (RCA) in highway infrastructure has the potential to reduce wastes and costs while producing the type of durable new roads required. Recycled concrete aggregate is produced from “Portland cement concrete pavements, bridge structures and decks, sidewalks, curbs, and gutters that have been removed from serviced, had their steel removed and have been crushed to a desired gradation”. Various tests have proven that with the right conditions RCA has the potential to produce materials of significant strength and durability with a higher load carrying capacity. There are numerous resource conservation benefits that result with the implementation of RCA. Firstly, waste disposal quantities are reduced. In most cities where a lack of landfill space is a real problem, waste reduction is a large benefit. Similarly, the use of these waste materials diminishes the cost of energy typically required for hauling virgin aggregate from quarries. Similarly to fly ash, RCA has bared the stigma associated with waste materials being substandard material. It has been realized that for RCA use to become more extensively used the process control needs to be improved to prevent mix workability issues. This includes watering stockpiles and testing the moisture content of the aggregates regularly. RCA has been used as coarse aggregate in hot-mix asphalt and as dense-graded aggregate.

9. CONCLUSION
Concrete is taking leaps and bounds when it comes to sustainable development. The management of CO$_2$ emissions along with voiced concern regarding the negative environmental impact of cement production proves that the minds of the industry are in right place. Research involving supplementary cementing materials has continuously proven the benefits of incorporating what is often a waste product from industries into concrete mix design. This can be noted through the increase of durability and strength resulting particularly in greater sustainable practices but also economical ones. Developments in the cement production process suggest that the interest to make improvements is being realized. New innovative methods are also being created to reduce the quantity of cement in a mix which is proof of a new perspective on the role the concrete industry can play in sustainability. In addition to the developments occurring directly with the production of cement
and concrete the application of these materials is also being redefined. Many limitations once binding concrete from becoming sustainable are fading as its use is incorporated into newer areas. Buildings can be built to use concrete’s thermal mass to help reducing energy requirements. The construction of dams is being optimized to use concrete to save energy during construction, and the lifetime of concrete is expanding with its reuse in aggregate form in roads. In combination, concrete has become multipurpose. As a result, although the initial energy production level is high, concrete can become more efficient.

The environmental and economic benefits of development in the direction of sustainability are inescapable. The continual search for opportunities to make this material, which has become such an integral part of our cities all over the world, a more sustainable option, proves that the minds of the masses are in the right place.

REFERENCES


CD01
Materials
THEORETICAL AND NUMERICAL STUDY OF THE BAR PLACED IN CONCRETE UNDER AXIAL TENSION LOAD AND DESIGN OF EXPERIMENTAL MODEL

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ABSTRACT

For considering the design considerations and development of retrofitting of structures, exact identification of materials' behavior and their interaction, is an important subject. In concrete structures that include a high percentage of existent and under construction structures, for calculation of anchorage length of bars and investigation of sufficiency of anchorage length of bars, the state of stress distribution in bar and joint surface with concrete and their effective parameters are important. One of the important parameters in modeling the joint surface between bar and concrete is equivalent spring constant between two materials. Most researchers suggest experimental test to obtain actual value of stiffness between two materials. In this study, for obtaining the exact values of stiffness, formulation on the base of equilibrium of elements and investigation of governing differential equations and experimental method were used. In experimental tests, slips and strains, pertinent to applying load were measured by strain gauges and displacement transducers that were installed between 4 stations over concrete and bar surface with an acceptable accuracy.

Keywords: concrete, bar, axial tension, friction, experimental model, displacement

1. INTRODUCTION

Interaction of bar and concrete is one of the important parts of reinforcement design, because limited length of bar has a main effect on the behavior of reinforcement. So, each code according to formulation and experience, presented some value. For better design of reinforcement element and controlling of codes value, interaction of bar and concrete should be attended. For determination of distribution of stress between bar and concrete, there are two methods: theory method and experimental method.

2. DEVELOPMENT OF THEORETICAL AND NUMERICAL METHOD

Axial stiffness between concrete and bar ($k_a$) is obtained from the following formulation and by using experimental results. Figure (1) shows internal force between concrete and bar. Equilibrium principle on element (1) shows the following result:
That $F_1$ and $\Delta F_1$ are axial forces in the bar section between element 1 and 2 and friction force of element (1), respectively. In other words, we have:

$$F_1 - F_{i+1} = \Delta F_1$$  \hspace{1cm} (2)

Also from the Equilibrium principle, we have:

$$F_0 = \Delta F_1 + \Delta F_2 + \Delta F_3$$  \hspace{1cm} (3)

![Figure 1. Internal force between concrete and bar](image)

The force of element (i) is always bigger than element (i+1), then: $F_i > F_{i+1}$

Relation between friction force and friction stress is shown in the following form:

$$\Delta F_1 = P \cdot L_i \cdot \tau_i$$  \hspace{1cm} (4)

That $P$ and $L_i$ are perimeter of bar and length of one small element of bar, respectively. It should be mentioned that variation of friction force between element (1) and (2) is supposed to be equal. Shear stress in element (1) is:

$$\tau_i = \frac{\Delta F_1}{P \cdot L_i} = \frac{F_0 - F_1}{P \cdot L_i}$$  \hspace{1cm} (5)

For element i we can write:
\[ \Delta F_i = F_{i-1} - F_i \]  
\[ \Delta F_i = P_i L_i \tau_i \]  
\[ \tau_i = \frac{F_{i-1} - F_i}{P_i L_i} \]  

On the other hand \( F_1 \) can be defined in the following form:

\[ F_1 = \sigma_1 A_p = E_p \varepsilon_1 A_p \]  

Where \( \sigma_1, A_p \) and \( \varepsilon_1 \) are normal stress of bar, cross section of bar and normal strain of bar, respectively. If strain is equal along the length of the element (1), we can define it according to displacements of two points.

\[ \varepsilon_1 = \frac{u_2 - u_1}{L_1} \]  

Where \( u_1 \) and \( u_2 \) are measured displacements of points in the laboratory. Then \( F_1 \) is equal to:

\[ F_1 = E_p A_p \left( \frac{u_2 - u_1}{L_1} \right) \]  

Friction force of different elements can be stated as follows:

\[ \Delta F_1 = F_0 - E_p A_p \left( \frac{u_2 - u_1}{L_1} \right) \]  
\[ \Delta F_2 = F_1 - F_2 = \frac{E_p A_p}{L_1} \left( (U_1 - U_2) - (U_2 - U_3) \right) = \frac{E_p A_p}{L_1} (u_1 - 2u_2 + u_3) \]  
\[ \Delta F_3 = \frac{E_p A_p}{L_1} (u_2 - 2u_3 + u_4) \]  

Due to earthquake, force is dynamic and since behavior of this case is different from that of the static case, dynamic case should be taken into consideration. Dynamic stiffness between concrete and bar, damping and part of concrete mass are dynamic parameters that are obtained by dynamic equilibriums and experimental results. In time domain, we have:
In one degree freedom and in frequency form, dynamic stiffness is obtained from the following form:

\[ K_a(\omega) = -\omega^2 M + K + i\omega C \]  \hspace{1cm} (14)

If \( M \) is included mass of bar (\( M_p \)) and added mass of concrete (\( M_{\text{add}} \)), we have:

\[ K_a(\omega) = -\omega^2 (M_p + M_{\text{add}}) + K_{\alpha} + i\omega C \]  \hspace{1cm} (15)

In the right section of Eq.(15), \( M_{\text{add}} \) and in the left section, dynamic stiffness in frequency form are unknown. Dynamic stiffness in frequency form is defined according to the following form:

\[ k_a(\omega) = \frac{F(\omega)}{X(\omega)} \]  \hspace{1cm} (16)

It should be mentioned here that excitation of both the system and displacement function are harmonic (sinuous form), but the displacement function has a different phase. Excitation of system function is given in the following form:

\[ f(t) = F_0 \sin \omega t \]  \hspace{1cm} (17)

If force (\( f(t) \)) is supposed in dynamic equation of one degree freedom, we have:

\[
\begin{align*}
\ddot{x} + \frac{c}{m} \dot{x} + \frac{k}{m} x &= F_0 \sin \omega t \\
\dot{x} + \frac{c}{m} \dot{x} + \frac{k}{m} x &= \frac{F_0}{m} \sin \omega t \\
\ddot{x} + 2\zeta \omega_n \dot{x} + \omega_n^2 x &= \frac{F_0}{m} \sin \omega t
\end{align*}
\]  \hspace{1cm} (18)

As a result:

\[ x(t) = e^{-\frac{c}{2m} t} \left[ C \cos \omega_n t + D \sin \omega_n t \right] + \frac{F_0}{K} \left[ \frac{1}{\left[ 1 - \left( \frac{\omega}{\omega_n} \right)^2 \right]^2} \right] \left[ \frac{1 - \left( \frac{\omega}{\omega_n} \right)^2}{\left( 1 - \left( \frac{\omega}{\omega_n} \right)^2 \right)^{3/2}} \right] \left[ \frac{\omega}{\omega_n} \right] \sin \omega t - 2\zeta \frac{\omega}{\omega_n} \cos \omega t \]  \hspace{1cm} (19)
After some time the displacement function of points will become constant and it has been shown in the form given below:

\[ u(t) = u_0 \sin(\omega t - \phi) \]  

(20)

That has a different phase \( \phi \) than the excitation force wave.

In the laboratory, \( F_0 \) can be measured. Maximum amplitude of displacement in axial direction can be recorded by sensors. System excitation frequency and displacement vibration frequency are equal, but have a different phase that is due to system damping.

In order to obtain dynamic stiffness in frequency form, we should calculate \( F(\omega) \) and \( x(\omega) \), that are Fourier conversion of dynamic force and dynamic displacement function, respectively.

\[
F(\omega) = \frac{1}{2\pi} \int_{-\infty}^{\infty} f(t) e^{-i\omega t} dt = \frac{1}{2\pi(1 - e^{-i\omega T})} \int_{0}^{T} f(t) e^{-i\omega t} dt
\]

(21)

\[
X(\omega) = \frac{1}{2\pi} \int_{-\infty}^{\infty} u(t) e^{-i\omega t} dt = \frac{1}{2\pi(1 - e^{-i\omega T})} \int_{0}^{T} u(t) e^{-i\omega t} dt
\]

(22)

Value of \( K_d(\omega) \) is equal to:

\[
K_d(\omega) = \frac{\int_{0}^{T} f(t) e^{-i\omega t} dt = R_a}{\int_{0}^{T} u(t) e^{-i\omega t} dt = R_b}
\]

(23)

As a result:

\[
K_d(\omega) = \frac{F_0}{u_0} (\cos \phi + i \sin \phi)
\]

(24)

Where:

\[
\phi = \tan^{-1} \left( \frac{2\rho \frac{\omega}{\sigma_n}}{1 - \left( \frac{\omega}{\sigma_n} \right)^2} \right)
\]

(25)

3. DESIGN OF EXPERIMENTAL MODEL

The actual form of the bond stress-slip model can be assessed by evaluating the
results of pullout test specimens. $250 \times 450 \times 800 \text{ mm}$. The pullout specimen was therefore cast to investigate the validity of the mechanics-based relationships derived in the previous section. Fig (1) shows the schematic form of specimens. Section A-A in Figure 1 shows the built-up reinforcing steel bar used in the pullout specimen.

The concrete had a target concrete compressive strength of 25 Mpa to represent the range of strengths commonly encountered in structures. Type 2 Portland cement was used, without admixtures, to obtain a water-cement ratio ($w/c$) of 0.64. The maximum aggregate size was 9.5 mm. The pullout specimen would be tested when the concrete was 44 days old and had a strength of 24 Mpa. The static yield strength ($f_{ys}$) of bars was 350 Mpa and the ultimate strength $f_u$ was 400 Mpa. The same reinforcing bar was used for both bars. For loading, the Hydraulic pump has been used. Since the front surface of concrete should be unloaded, force entered into two plates and put spacers at the edges of plates. Because the pump is in the center of the plates, the loads of bars are the same (Figure 2).
Specimens were cast with the reinforcing bar secured horizontally by the left and right forms. Blocks on the top form bore against the outside of the side forms to ensure proper bar alignment. Lateral view of the experimental model is shown in Figure (3). Figure (4) shows top view of spacers men.

There are four transducers for measuring of bar displacement at four points of each of the bars. For stability of transducers, two plates are put on the surface of concrete and the transducers' magnet is activated. Figure(5) shows one transducer on the spacers men. Transducer number 1 was installed closest to the loaded end of the bar whereas strain gauge number 4 was closest to the unloaded end. A digital data logger and a personal computer where used to record the 8 slip values and the relative tension load (P). The loading rate for loads less than $P_{\text{max}}$ ranged from approximately 0.66 to 0.70 Kn /second (150 to 160 lb/second) and, after $P_{\text{max}}$ occurred, from $-1.90$ to $-8.69$ Kn/second ($-430$ to $-1950$ lb/second). The slip rate during the loading stage was calculated by examining the very small end slips that occur at the maximum load level. After those models of various cases are prepared, results are presented in a feature paper.
4. CONCLUSION
Interaction between bar and concrete is a main point of focus in the concrete design. We can use theory relation for specifying stiffness between bar and concrete. This stiffness has static and dynamic share. Properties of dynamic stiffness can be obtained by using the formulation and experimental results. Also these relations can be compared by the experimental results. For this purpose, the experimental model should be realistic.

REFERENCES
EFFECTS OF WOOD-PULP FIBERS ON THE MECHANICAL PROPERTIES OF CEMENT COMPOSITES

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ABSTRACT
The application of pulp fiber in cement paste has been under consideration to improve the bearing capacities of the cement composites. Cement composites made by various types of fibers have distinct properties regarding the stability and resistance in dissimilar environmental conditions and applied loads. This behavior depends on four main factors: a) fiber type, b) mixture percentage c) fabrication manner, and d) additives. In this work, to distinguish the flexural behavior of fabricated composites with wood-pulp cellules fibers, the experimental samples designed and tested. The samples made with these fibers were compared with the no-fibrous control samples for their flexural strength and modulus of elasticity. Moreover, to characterize the microstructure properties SEM micrographs were analyzed. The results showed that the application of fibers had suitable effects on the improvement of the flexural strength related to the amount of used fibers.

Keywords: cement board, Pulp fiber, flexural strength, Modulus of elasticity, SEM micrograph

1. INTRODUCTION
All over the world, the production of the cement boards, are based on eckhatch procedure. The history for this method, back to about 100 years ago, in which was derived from paper production technology. Following this procedure, water, fibers and cement should be mixed at first and then, using a special process, this matrix converted to cement composite boards (CCB). To fabricate CCB, the cement matrix positioned on the driving belt, water-drained, and after placing the layers on each others, CCB will be formed. In original procedure, asbestos fibers were used which had good consistency with cement paste, physical and chemical properties, durability and mass-productivity specifications. The growth in production of CCB leads to increase in use and application of Asbestos material, in which, in 1985 the outmost production rate was recorded. Unfortunately clinical researches show harmful effects of this material on the humanity health [1]. Consequently, application and fabrication of the asbestos-based products were inhibited in the majority parts of construction industries. Despite this, the need for CCB motivates the researchers to find an effective solution. This solution should cover the
The efforts were initiated in 1980's [10-2]. The solution was the mixed application of various types of cellulose, polymer, and suitable additives. The world master producers of CCB were the leaders of these research efforts. Also, several countries were looking for an appropriate and suitable materials and production techniques for their local applications. By the way, cellules and polymer based fibers (in particular Poly Vinyl Alcohol (PVA) fibers and other new products) were globally accepted as an effective material to be used. It is obvious that what is the researchers are seeking for is the economically optimized mixing proportion for production of the standard CCB. In this regard, the local fibers and domestic material and methods are recommended. For that reason, some factories in Thailand, Turkey, and Belgium succeeded to produce CCB with in access and localized materials, however, some other countries are preferred to import materials for this means. In Iran, there are many economical problems in technology transfer and on the other hand because of the dependency of the Iranian factories to PVA, these factories continue to use the asbestos in their products, despite the restricting regulations. In the recent years, with some growths in demands, 40 million tons of CCB and about 4500 km sewage and water pipes were produced with asbestos materials. Beside, the lack of intense restricting rules and regulations encouraged the continuation of the asbestos products in Iran. The first regulation concerned this issue was back to 2001 when the superior council for protecting of the living environment, puts some restrictions on the application of the asbestos materials. This states that after July 2001, the newly established factories are forbidden to use the asbestos in their products and the factories that were previously using the asbestos as a raw material have been ordered to modify their production procedure to replace the asbestos with other allowable material to completely eliminate the applications in the next 7 years.

The current study was started in early 2007 after this rules encouraged the researches seeking appropriate fibers to be replaced with asbestos materials. These fibers should be met all of the advantageous of asbestos and on the other hand these fibers should not affect the human heath. The pulp fiber produced in paper production factories were considered here with some surface treatments. These fibers were used in reinforcing the cement composite boards. After surface treatment process, their mechanical and physical properties regarding the flexural strength and young modulus were investigated. Samples were made with various fiber contents and then using SEM micrographs the micro-structural properties were studied.

2. TESTS AND METHODS

Cement: Type 2 cement supplied by Tehran Cement Factory was used in this study. Standard laboratorial tests (based on Iranian National Standard No. 398) were executed to determine the properties of this cement, which passed the requirements.

Fibers: The major part of the fibers of this study was prepared from the
agricultural wastes. These fibers are usually used for paper production in which this paper it was named as wood-pulp. The used dimensions were wide-spread of length and thickness where would be discussed later.

Water: Tap water was used to sample to make.

3. MIX DESIGN AND SAMPLE PREPARATION
The amount of wood-pulp was the major parameter of this study. These fibers were used in the range of 0 to 14% of cement weight.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Cement (g)</th>
<th>Water (g)</th>
<th>Fiber (gr)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Control</td>
<td>150</td>
<td>450</td>
<td>--</td>
<td>Non-fiber sample</td>
</tr>
<tr>
<td>P2</td>
<td>150</td>
<td>450</td>
<td>3.0</td>
<td>2% Cellulous-fiber</td>
</tr>
<tr>
<td>P4</td>
<td>150</td>
<td>450</td>
<td>6</td>
<td>4% Cellulous-fiber</td>
</tr>
<tr>
<td>P6</td>
<td>150</td>
<td>450</td>
<td>9.0</td>
<td>6% Cellulous-fiber</td>
</tr>
<tr>
<td>P8</td>
<td>150</td>
<td>450</td>
<td>12.0</td>
<td>8% Cellulous-fiber</td>
</tr>
<tr>
<td>P10</td>
<td>150</td>
<td>450</td>
<td>15.0</td>
<td>10% Cellulous-fiber</td>
</tr>
<tr>
<td>P12</td>
<td>150</td>
<td>450</td>
<td>18.0</td>
<td>12% Cellulous-fiber</td>
</tr>
<tr>
<td>P14</td>
<td>150</td>
<td>450</td>
<td>21.0</td>
<td>14% Cellulous-fiber</td>
</tr>
</tbody>
</table>

Composite cements were designed and made with a w/c ratio=3. At first fibers were mixed in rotary mixer with 15 mm horizontal blades for 5 min to be separated. This initial preparation was for untwisting the fibers to be well-dispersed in the cement mortar. Cement, water and fibers were mixed for another 5 minutes.

After preparing the materials and mixing process, the prepared mixtures were poured into 8×18×15 cm molds. Excess water was drained with a 0.9 bar suction pump (Figure 1) while applying a 10 kg weight on the samples. Then the samples were dried for 1 hour and cured in a steam cabinet with 100% RH for 14 days. After curing, the samples de-molded and dried for 6 hours in 75°C to prepare for mechanical tests.

Figure 1. Set up for preparing the samples
4. TESTS
4.1. Tests for fibers
4.1.1. Freeness test
One of the important characteristics of the fiber in cement matrix is the Canadian Standard Freeness (CSF) that was designed for measuring the drainage properties of the wood-paste. The results of CSF test depend on many variables such as: the amount of fine particles and small pieces of available wood, fibrillation degree, flexibility of fibers, and the finesse modulus. The procedure for this test is as follows:
1. Specific volume of wood-paste poured into the cylinders to be drained. Accompanying liquid was brought in the conical case with two orifices one in the bottom and the other located on the side surface of the case.
2. Drained volume of liquid was measured and reported as degree of freeness after some modifications on the values of temperature coefficient and paste density.
3. In this research, the cured fibers were examined for freeness test according CSF.
4. Average measured value for CSF was 500 which were very close to results of other researches.

4.1.2. Morphological tests for fibers
Prepared fibers were poured into the test tubes and de-fibered. After fully separation, length and diameter of fibers as well the diameters of cellulose pores were measured with projectina optical microscope with 30 tries.

Table 2: Morphological characteristics of fibers

<table>
<thead>
<tr>
<th>Morphological characteristics</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length</td>
<td>mm</td>
</tr>
<tr>
<td>Demeter</td>
<td>30.853 micron</td>
</tr>
<tr>
<td>Inside cellulose wall</td>
<td>4.102 micron</td>
</tr>
<tr>
<td>Pore cellules wall</td>
<td>22.648 micron</td>
</tr>
</tbody>
</table>

5. EXPERIMENTAL CURVES
In this research, the strength of samples was tested in flexural loads. The flexural samples were flat rectangular and tested with a 3-point load system according to the EN12467:2004.

Figure 2 shows the load-deflection curve for CCB with and without fibers. As it can be seen in Figure 2, the application of fibers in CCB increased the flexural bearing capacity (FBC). The maximum observed value in control sample of FBC was 54.42 N, while this enhancement for 4, 6, 8% fiber added samples were 109.75, 238.35, and 289.62, respectively. These values showed that the addition of fibers in cement paste notably increased the FBC. The effect of thickness was not evaluated in this curve, therefore, could not properly reflect the capabilities. Because the more fibers added, the more thickness of CCB appeared and so, the increase of thickness could affect the ultimate FBC. To eliminate this deficiency, the following relations were utilized in evaluation of the FBC of CCB:
\[ \sigma = M / W \]  
(Eq. 1)

Figure 2. Load-deflection curve of CCB reinforced with cellulosic fibers in comparison to the control samples

Where \( \sigma \) is available stress (MPa), M is flexural moment, W is the flexural capacity of the section.

\[ M = PL / 4 \]  
(Eq. 2)

\( P \) is applied concentrated load (N) and \( L \) is the length of sample.

\[ W = BH^2 / 6 \]  
(Eq. 3)

Where \( B \) is width (mm) and \( H \) is height (mm) of sample.

\[ E = \sigma / \varepsilon \]  
(Eq. 4)

Where \( E \) is modulus of elasticity (MPa) and \( \varepsilon \) is strain.

By replacing the values of \( M \) and \( W \) in Eq. (1), the stress will be attained as follows:

\[ \sigma = \frac{3PL}{2BH^2} \]  
(Eq. 5)

Deflection can be calculated by assuming the linear region as:

\[ \delta = \frac{PL^3}{48EI} \]  
(Eq. 6)

Where \( I \) is the moment of inertia and \( \delta \) is the deflection (mm).
By increasing the load $P$, the value of $\delta$ could be measured in real time. If the linear region was assumed, it would be computed as:

$$\delta = \frac{3P}{4EI} \quad \text{(Eq. 7)}$$

Where

$$I = \frac{1}{12}BH^3 \quad \text{(Eq. 8)}$$

Using the Eq. 7, the value of $\varepsilon$ can be gained:

$$E = \frac{P_L^3}{48\delta I} \quad \text{(Eq. 9)}$$

Based on these relations, flexural strength of CCB samples were evaluated as shown in Figure 3.

For more analysis, the stress-strain curves are plotted in Figure 3. As it can be seen, application of fiber upgrades the maximum yielding stress. Moreover, by increasing the fibers, the area under the stress-strain curve has been increased that is related to the energy absorption properties of boards.

6. FLEXURAL STRENGTH

Figure 4 demonstrates the results of flexural strength of samples. The results proved that the addition of fibers resulted in improvement of flexural strength of CCB. Obviously, the rate of enhancement depends on the type and amount of added fibers and various percentages of fiber replacements affect the mechanism of failure in CCB. Assessment of stress-strain curves of CCB guided us to classify the CCB based on the type and amount of fiber replacements into three groups:
Group 1: Samples with 0-6% fiber replacement
Group 2: Samples with 8-10% fiber replacement
Group 3: Samples with 12-14% fiber replacement

In group 1, as depicted in Figure 4, the flexural strength enhanced slightly with an increase in the fiber amount, whereas the minor enhancement, the failure mechanism of the samples was totally different which varied from brittle to ductile.

The samples were placed in group 2, have the most flexural capacities in contrast to the reference samples. In some samples in this group, the enhancement reached up to two folds. Moreover, the flexible failure mode was observed for all of the samples in this class. In this situation, well-dispersion of fibers with good bond formation between cement paste led to good development of flexural strength of these samples.

In group 3, with increasing the fibers, the more decrease in flexural strength was observed that it is expected to continue up to 14% fiber addition. Assessment of failure mechanism showed that the ductility of samples in this group is higher than other groups. It should be noted that the higher amount of fiber in this group lead to an unfavorable appearance due to high concentration of fibers at the outer surface of CCB. Moreover, after breaking the samples under load, the balled-shape fibers are visible in some parts of matrix that leads to missed-dispersion of fibers inside the matrix eventually leads to decreasing the flexural strength of this group.

The samples were analyzed and showed in Figures. 5 and 6. The microstructure of paper-pulp fibers represents the rough surface with good fibrillation. Well ionfibrilizat of fibers forms the numerous fibrils around the outer surface that could help the friction bond strength with cement matrix. On the other hand, the high aspect ratio (10.23) and smaller diameter (30.85 μm) assist in friction strength. The matter will be more important at the interfacial zone of fiber-cement paste and increasing the effective
bond between them. In these pictures very tiny particles associated with the fibers are gibleinegl. Therefore at the sites of cement paste that these materials stanceexi can be interpreted as defects that decrease the strength. Consequently, the existence of these particles in lower amounts could restrict the weak regions inside the cement paste. But other important factor influencing the cement composite with fibers is the fibers orientation in the cement paste. Figure 6 shows the distribution and performance of fiber in cement paste. It is considerable that the fibers are dispersed uniformly throughout the samples. Moreover, the existence of cement particles around the fibers demonstrates the establishment of well interaction between fibers and cement paste. This shows a good bond development between fiber and cement paste.

Figure 5. SEM Micrograph of cement paste with Kraft fiber

Figure 6. SEM Micrograph of Kraft fiber
7. MODULUS OF ELASTICITY

The modulus of elasticity (ME) is computed for ascending branch of stress-strain curves and the values are depicted in Figure 7. As it can be seen, the modulus of elasticity in all of the samples is lower than reference sample and with increasing the amount of fiber, it will be reduced. Many factors are involved in this property. If the cement composite is considered as a two-phase material (fiber and cement) then ME of the fibers can be effective in overall modulus of elasticity of cement composite so that the more or lesser ME of fibers the more or lesser ME of cement composites will be gained.

It should be noted that the amount of fiber in reinforcing the cement composite is very determinative. To verify this phenomena, the following relation that proposed by Allen [11] which was used to compare with the results of experimental modulus of elasticity of cement composites.

\[ E_c = E_m(1-V_f) + E_f V_f \]  
(Eq. 10)

![Figure 7. Modulus of elasticity of cement composite boards with and without fibers](image)

Where \( E_c, E_m, E_f \) are the composite, cement matrix and fibers modulus of elasticity respectively and \( V_f \) is the volumetric percentage of fiber in the composite.

Experimental results of modulus of elasticity are:
- Cement modulus of elasticity = 8.788 GPa
- Average modulus of elasticity of pulp = 85.36 GPa

Using the composite component's ME yields great values for in calculated ME for composite material while the experimental observation do not agree with these findings. So it is supposed that the Allen formula is proper when the composite is considered in an ideal condition. Researchers showed [11,5-6] that with application of fiber in cement paste resulted in increase of voids in cement matrix and
consequently the micro defects extended. Then generally the modulus of elasticity of cement composite with fiber would we more less that the samples without fibers. Allen [11] proposed a formula regarding this concern as follows:

\[ E_m = E_{mo}(1-p) \]  
(Eq. 11)

Where \( E_{mo} \) is cement matrix ME without fiber, \( p \) is the amount of voids in the composite in percent. Moreover, Allen [11] showed that \( p \) is as a part of fiber in percent and could be calculated as follows:

\[ p = 0.0522 + 3.7407 V_f \]  
(Eq. 12)

It seems that the Eqs.11 and 12 is very reliable and could reflect the experimental observations.

On the other hand, in the mixing procedure of cellules fiber and cement paste with water, some bubbles and spumes would appear on the surface of mortar which could be lead to increase the thickness of the samples with fibers. Thus the results obtained from laboratorial study could be comparable and reliable. The reason for producing the bubbles or spumes is the application of alkaline stuffs in chemical process for production of the pulp. When these chemicals components are contacted with oil used to lubrication of the molds, these bubbles or spumes are formed. As a result, though ME of fiber is greater than the cement paste, but because of these process (bubbles or spumes formation) resulted in increment of porosity of composite leads to decrease in reduction of composite ME. Increase in fiber amount would cause to extending the porosity and finally expansion of samples based on Eq.12 then, in all samples with fiber ME would be reduced by increase in fiber amount.

8. CONCLUSION
From the results obtained in the effect of wood-pulp fibers in mechanical properties of cement composites, the following conclusions can be drawn:
1- Cellulose-fibers extracted from the brief-preparation of pulp has good consistency with cement paste and could be dispersed inside the cement matrix and finally have well bonding with cement paste.
2- By increasing the fiber amount up to 8% of cement, the flexural strength of cement boards would be increase and in the range of 8 to 10% this is constant or has very low decreases. With addition of fiber more than 12%, the flexural strength development would have descendent slop.
3- The main reason for decrease of strength of cement composites, are: a) thickening of the samples because of porosity and b) miss dispersion or non-uniform distribution of fiber in cement paste in which fibers want to be like clew and twisting or miss dispersion inside the matrix.
4- Existence of fibrils could aid in binding and uniformity of fibers with cement paste and eventually leads to enhancing the flexural strength.
5- Modulus of elasticity of composites got more affection from the bonding and continuity of the fibers with cement paste than its components like cement or fibers; does the more bonding strength the more modulus of elasticity would be exist.

REFERENCES
APPLICATION OF SMA IN CONCRETE STRUCTURES

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ABSTRACT
Shape memory alloys (SMAs) are increasingly becoming a topic of research in the area of ‘smart materials’. SMAs are a novel functional material, which can exhibit large strains under loading–unloading process without residual deformation. They have the ability to remember a predetermined shape even after severe deformation. This article first presents an overview of the characteristics of SMAs associated with the temperature-induced and stress-induced reversible hysteretic phase transformation between austenite and martensite. The recent experimental studies and numerical simulations, which have been led to demonstrate the powerful role played by SMAs, are also presented in this article. Currently, research efforts have been extended to using SMA as sensors, actuators, passive energy dissipaters and dampers for shape control and vibration control of civil structures. This article then presents a review of applications of the SMA materials for passive and active controls of concrete structures.

Keywords: shape memory alloys, shape memory effect, superelastic effect, structural control, smart material

1. INTRODUCTION
Smart systems for civil structures are described as systems that can automatically adjust structural characteristics in response to external disturbances and/or unexpected severe loading toward structural safety, extension of the structure’s lifetime, and serviceability [1].

In 1965, shape memory alloys (Nitinol) as a smart material derived from Nickel and Titanium were first patented by Buehler and Wiley [2] in Naval Ordnance Laboratory. Since then, tremendous effort has been infused to the utilization and study of this smart material. In recent years, the two major properties of SMAs have attracted the attention of many researches for application to smart structural systems. One is the Superelasticity or pseudoelastic effect (PE), which is the ability of a shape memory alloy to accommodate large strains due to stress-induced phase change at a constant, sufficiently high temperature and to recover its initial shape upon unloading. The other is the superthermal effect or shape memory effect (SME), which is the ability to deform an initially austenitic SMA by cooling under constant stress and then to recover the austenitic shape by heating. The magnitude of the
temperature-induced strains depends on the applied stress. Although SMAs have been known for decades, they have not been used in the concrete structures until rather recently. SMAs can be passive or active components in civil structures to reduce damage caused by environmental impacts or earthquakes.

This paper is divided into three main parts. The first part focuses on the basic characteristics of SMA (Section 2). The second part contains recent research on the damping properties of SMAs (Section 3). The third part presents the Application of SMAs for passive and active concrete structure control (Section 4).

2. BASICS ABOUT SHAPE MEMORY ALLOYS

SMAs are found in two main phases: the high temperature phase, which is called austenite, and the low temperature phase, which is called martensite. SMAs could be transformed from austenite to martensite either by reducing the temperature or by applying a mechanical stress. On the other hand, martensite transforms into austenite by either increasing the alloy’s temperature or removing the applied stress.

SMAs have four transformation temperatures: (a) the austenite start temperature \( T_{As} \), where the austenite starts to develop in the alloy; (b) the austenite finish temperature \( T_{Af} \), where the development of austenite in the alloy is 100% complete; (c) the martensite start temperature \( T_{Ms} \), where the development of martensite starts; and (d) the martensite finish temperature \( T_{Mf} \), where the development of martensite is 100% complete.

There are three groups of shape memory effects [3]. All of them have one common speciality, namely at least one shape (macroscopic state) of the material is recoverable. In the case of one-way effect the material gets a permanent deformation by applying mechanical load in a relative cool temperature \( T < T_{Af} \). However, this deformation can disappear by heating above \( T_{Af} \) and it remains unchanged during the cooling to the start temperature (Figure 1.a).

When the start temperature is above \( T_{Af} \), mechanical load can cause deformation, but it disappears during unload. It seems like an elastic behavior, but the deformation can be unusually great. This effect is the PE, which does not concern only shape memory properties (Figure 1.b).

The third effect is the two-way effect that requires only thermal load to change between two stable shapes. One of the shapes is stable above \( T_{Af} \) and the other one is stable below a different temperature \( T_{Mf} < T_{Af} \). It has to be mentioned that this effect can be produced only after a special treatment (Figure 1.c).

![Figure 1. Shape memory phenomena: one-way effect (a), pseudoelasticity (PE) (b), and two-way effect (c) [4]](image-url)
Figure 2. Shape memory phenomena in stress-strain-temperature space: one-way effect (a), pseudoelasticity (PE) (b), and two-way effect (c) [4]

Behind these effects, there is a crystallographic transformation, namely the martensitic phase transition. As it can be seen from the phenomena, the phase transitions can be induced by mechanical and thermal load. Figure 2 shows the effects in a stress-strain-temperature space. The forward (austenite to martensite, $A \rightarrow M$) and backward (martensite to austenite, $M \rightarrow A$) transitions and their temperatures are also illustrated.

3. RESEARCH ON THE DAMPING PROPERTIES OF SMAS

The high damping capacity is known as one of the important functional properties of shape memory alloys. Damping, in a technical context, stands for the conversion of mechanical energy to thermal energy and therefore for the ability to reduce movements or vibrations of a structure.

Using SMAs for passive structure control relies on the SMA’s damping capacity, which represents its ability to dissipate vibration energy of structures subject to dynamic loading. As reviewed in the last section, the damping capacity comes from two mechanisms: martensite variations reorientation which exhibit the SME, and stress-induced martensitic transformation of the austenite phase which exhibit the PE.

The energy dissipation of the widely-used Nitinol superelastic SMA wires was investigated [5-8]. Dolce and Cardone [9] investigated the superelastic Nitinol wires subjected to tension loading. They observed the dependence of the damping capacity on temperature, loading frequency and the number of loading cycles. It is found that the mechanical behavior of the wires is stable within a useful range for seismic application. In addition, they suggested that the austenite wires should be pretensioned for larger effectiveness of energy dissipation.

A superelastic SMA wire demonstrates the damping capacity not only under tension loading, but also under cyclic bending. In 2000, Ip presented his effort to predict the energy dissipation in SMA wire under pure bending loading. His numerical results showed that the energy dissipated by the superelastic SMA wire is highly sensitive to its diameter; in detail, the thicker the SMA wire, the more energy was dissipated.

Recently, as large cross-section-area SMAs become available, studies on the properties of SMA bars or rods have attracted more attentions. As discovered by Liu et al [10], the damping capacity of a martensite Nitinol bar under tension-compression cycles increases with increasing strain amplitude, but decreases with loading cycles and then reaches a stable minimum value.
Dolce and Cardone [9] compared the martensite damping and austenite damping of Nitinol bars subjected to torsion. They found that the damping capacity of the martensite Nitinol bar is quite a bit larger than that of the austenite Nitinol bar, although the prior cannot remain at its highest value as the residual strain accumulates. They also noticed that the martensite bar’s mechanical behavior is independent of loading frequency and that of the austenite bar slightly depends on the frequency. This implies that both martensite and austenite Nitinol bars can work in a wide frequency range and have a good potential for seismic protection.

4. APPLICATION OF SMAS IN CONCRETE STRUCTURE CONTROL

The vibration suppression of concrete structures to external dynamic loading can be pursued by using active control and passive control. In the active control mode, an external power source controls actuators to apply forces to the object structures. For a passive control system, no external power source is required and the impact forces are developed in response to the motion of the structures.

4.1. Smas for Passive Structural Control

The passive structural control using SMAs takes advantage of the SMA’s damping property to reduce the response and consequent plastic deformation of the structures subjected to severe loadings. Indeed, martensite or austenite SMA elements as energy dissipation devices absorb vibration energy based on the hysteretic stress–strain relationship.

4.1.1. Sma Braces for Frame Structures

The SMA wire braces are installed diagonally in the frame structures. As the frame structures deform under excitation, SMA braces dissipate energy through stress-induced martensite transformation (in the superelastic SMA case) or martensite reorientation (in the martensite SMA case).

Several different scale prototypes of the devices were designed, implemented and tested. They showed that the proposed devices have characteristics of great versatility, simplicity of functioning mechanism, self-centering capability, high stiffness for small displacements and good energy dissipation capability. The combined steel–SMA type braces were also adopted by Tamai and Kitagawa [11] in their seismic resistance devices as shown in Figure 3. Cardone et al [12] proposed a design for bracings of multi-storey reinforced concrete (RC) frames with a martensitic Ni-Ti adapter as the damping element.

4.1.2. Sma Restrainers for Bridges

Bridge restrainers are elements that are commonly used to connect two adjacent bridge spans or frames and prevent them from experiencing large relative displacements during earthquakes. Superelastic SMAs can be used as damper elements or potential seismic restrainers for bridges.

As shown in Figure 4, DesRoches and Delemont [13] reported their full-scale tests of 25.4 mm diameter superelastic SMA restrainer bars used for seismic retrofit of simply support bridges and their simulation analysis on a multi-span simply
supported bridge. The results have shown that the SMA restrainer more effectively reduced relative hinge displacement at the abutment and it provided a large elastic deformation range in comparison with conventional steel restrainer cables. In addition, the SMA restrainer extremely limits the response of bridge decks to near-field ground motion. The increased stiffness of the SMA restrainers at large strains provides additional restraint to limit the relative openings in a bridge.

Moreover, Rita Johnson et al [14] conducted a large scale testing program to determine the effects of SMA restrainer cables on the seismic performance of in-span hinges of a representative multiple-frame concrete box girder bridge subjected to earthquake excitations. The SMA cable restrainer which was used in this study is shown in Figure 5. The results of the experimental testing have revealed that the SMA restrainers not only served as effective bridge retrofits, but also result in superior performance relative to equivalent traditional steel restrainer systems. Additionally, results of utilizing the analytical model revealed that using SMA restrainer cables reduced the peak hinge openings by nearly 50% for some cases.
4.1.3. Sma Connectors
Connectors or connections in various structures are prone to damage during an earthquake event. SMA connectors have been designed to provide damping and tolerate relatively large deformations. Tamai and Kitagawa [11] proposed an exposed type column base with SMA anchorage for seismic resistance. The SMA anchorages are made of the Nitinol SMA rods in 20–30 mm diameter and steel bars, as shown in Figure 6.

![Figure 5. SMA cable restrainer with the effective length of 1.16 m used as a retrofitting device](image)

![Figure 6. Schematic of SMA bar anchorage for a column](image)

The results obtained from the pulsating tension loading tests and numerical simulation of the SMA rods, have shown that the SMA wires were very effective in dissipating energy and reducing the building’s vibration under severe seismic ground motion. Furthermore, the SMA anchorages can recover their original shape after cyclic loadings and therefore their resisting performance remains the same to prevent plastic deformation and damage in the structural columns. Additionally, it is possible to design a column base with SMA anchorage that does not require repair after a severe earthquake, when the maximum rotation responses of the base plate are less than 0.025 rad [15].
4.1.4. Shape Restoration Using Superelastic Smas

In the literature, there is a specific type of application of superelastic SMA wires for structural control purpose different from the aforementioned examples. This application uses the shape restoration property of superelastic SMA wires. For example, Song and Otero [16] developed a more efficient way to use superelastic SMA wires to achieve a larger restoration force in the form of a stranded cable. Figure 7 shows a concrete beam (24 in. × 4 in. × 6 in.) reinforced with fourteen 1/8 in.-diameter superelastic stranded cables via the method of post-tensioning to achieve a 2% pre-strain. Each cable has seven strands and each strand has seven superelastic wires. Special clamps were made to hold the superelastic strands/cables without slippage. After a load of 11,000 lbs and the appearance of a large crack (Figure 7.a), the crack on this beam was closed (Figure 7.b) under the elastic restoration force of the superelastic SMA cables upon removing the load. Two quarter-scale RC column with SMA longitudinal reinforcement in the plastic hinge area were tested on the shaketable by Saiidi and Wang [17]. The exploratory study showed that the residual displacements in the SMA-reinforced columns were very small. Furthermore, Khaloo and Eshghi [18] studied numerically the response of RC columns using smart rebars under static lateral loading. It is found that by using SMA rebars in RC columns, these materials tend to return to the previous state (zero strain), so they reduce the permanent deformations and also in turn create forces known as recovery forces in the structure which lead into closing of concrete cracks in tensile zone and reduction of the eccentricity created in the concrete columns which is the result of permanent deformations.

![Figure 7. A large crack during a loading test (a), and the crack closes after the loading test (b)](image)

4.2. Smas for Active Structural Control

The SMA has the capability of recovering a previously formed shape via heat energy, which is referred to as an active property tuning when incorporated into a structural system.

4.2.1. Sma Wires in Concrete

The behavior of a simple concrete beam driven by heated SMA wires using electrical currents was investigated by Li et al [19]. Figure 8 shows the loading
apparatus. Prior to the test, a certain pre-tension was imposed on the SMA wires, which were fixed firmly at the two ends of the specimen by the special clamps. Specimens were first loaded at the midspan to a certain deformation until the concrete was cracked. Subsequently, the SMA wires were heated using a constant electrical current of 14 A in order to drive the concrete beam. The test results indicate that recovery forces of the SMA wires can reduce mid-span deformations and compressive strains of the specimens effectively. Furthermore, the heated SMA wires can make cracks close and perform the task of emergency damage repair in civil structures. The load capacity after actuating of the SMA wires increases although the concrete is already cracked. Moreover, the specimen embedded with more SMA wires proves much better than the specimen does with fewer SMA wires.

![Figure 8. Set-up of the loading apparatus (a), and instrumentation sketch (b) for concrete beam with embedded SMA wires](image)

![Figure 9. Active Confinement of a concrete column with a prestressed shape memory alloy wrapping for retrofitting purposes [20]](image)

### 4.2.2. Active Confinement of Concrete Members with SMA

Another application of using SMAs in concrete structures is in the confinement of reinforced concrete members. The increase in load bearing capacity and also ductility by wrapping columns with bands or sheets of steel or FRP is well known. In addition,
it is well known that the strength of confined concrete is a function of the load. Utilizing the shape memory effect for tensioning the wrapping can enhance the effect of confinement. Krstulovic-Opara et al [21] carried out tests on SMA confined concrete members. They performed compression tests of the model scale with confined concrete cylinders. The specimens were jacketed with thin continuous Ni-Ti wires. Stressing of the jacket was done by putting the whole specimen in an oven. The comparison between several variants of stressed and unstressed jacketing showed that the use of SMA spirals alone effectively introduced high levels of active confinement. Concrete columns could be easily helically wrapped by continuous SMA bands. The pitch of the helix can be fitted to the aimed confinement. Figure 9 shows the setup for the tensioning by resistance heating. Obviously, this technology is suitable in particular for retrofitting, in cases where there is only limited space for mounting, e.g. in cellars of buildings or in case of double columns. The strength values under confinement can theoretical enable very high loads at very high strains. However, only compression strains of several percent are acceptable in columns in order to prevent damage to the concrete or disadvantages to the whole structural system of a building. The load bearing capacity for small strains was hence of interest for the performed calculation. The calculations showed a lower axial strain for the active SMA confined column compared to the steel or CFRP confined column at the same load. On the other hand, a higher axial load can be applied at a given ultimate strain.

5. CONCLUSIONS
This paper presents a review of the basic properties of Nitinol SMA and their applications in passive and active control of concrete structures. The SME enables martensite Nitinol materials to be used as actuators and also enables their applications in active controls of concrete structures. Structural self-rehabilitation using reinforced martensite SMAs is an example of active structural control. Both martensite and superelastic SMAs show strong hysteretic effects in their stress–strain curves for loading–unloading cycles and dissipate energy during these cycles. This provides the basis for developing passive structural damping devices using both martensite and superelastic SMAs. We have seen a trend to combine the advantages of martensite and austenite SMAs to achieve optimal performance in structural control.

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EFFECT OF SILICA FUME ON HYDRATION HEAT AND STRENGTH DEVELOPMENT OF HIGH STRENGTH CONCRETE

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ABSTRACT
Recently, silica fume became a vital ingredient for producing concrete in aggressive hot climate of Iran. In mass structures heat generation due to hydration of cement and strength properties are two important parameters which affect service life of structures. In the early age the temperature in the body of mass concrete is high and tensile strength is low. This special condition may lead to occurrence of thermal cracks. In the present study, concrete specimens with water cement ratio 0.3, were made and 0, 10 and 15 percent Portland cement replaced with silica fume. Temperature rise of the specimens was monitored just after casting in a semi adiabatic box. Temperature rise was recorded for 7 days. Furthermore, compressive strength of the cubic specimens from 1 day after casting to 91 days was measured. The results declared that hydration heat regime is affected by silica fume percent. The mixes with both 10 and 15% silica fume, had peak temperature about 6ºC lower than the specimens without silica fume. Furthermore, peak temperature of the specimens without silica fume occurred 23 hours after casting, while it became about 31 hours for silica fume specimens. The slope of cooling zone of hydration regime in the specimens with 10% silica fume is very mild compare with the others. On the other hand, 10% silica fume enhanced compressive strength more effectively. These results demonstrated that hydration heat and strength development of mass concrete are affected by silica fume content and higher replacement of this pozzolan material may adversely affect service life of structures.

Keywords: silica fume, hydration heat, high strength concrete, strength development

1. INTRODUCTIONS
Strength development of high strength concrete in the body of large mass concrete structures is effectively influenced by temperature rising due to hydration heat. Normally, cement content of high strength concrete is high, therefore temperature in the body of mass structures increase more rapidly and may adversely affect properties of concrete at early or later ages [1]. In addition, the characteristics of heat development such as peak temperature, the time at which peak occur, slope gradient in heating or cooling zone and the remaining time at peak can be responsible for concrete properties in body of large mass concrete[2]. However,
nowadays the consulting have no especial considering to a fact that reality of concrete properties in the center of structure is completely different with that named as standard specimens [3].

2. MATERIALS AND TESTING PROGRAM
Crushed stone, with 19 mm maximum nominal size, in two ranges of 5-10 and 10-19 with relative density at saturated surface dry of 2.61 were used. Fineness modulus of sand and relative density were 3.24 and of 2.56 respectively. Absorption value is 3.09 and 2 for fine and coarse aggregate. The cement used was Portland cement Type 2, with a specific gravity of 3.11 and 3750 cm²/gr surface area. A commercial carboxylic type plasticizer, (Gelenium 110M), was used to maintain workability of fresh concrete. Silica fume, made by Semnan Ferro Alley factory, was used at 0%, 5% and 10% (by weight) as partial replacement of cement. The characteristics of silica fume are given in Table 1. Mix proportions of the concrete are given in Table 2. Water-cementitious material (w/cm) ratio is 0.3. A pan mixer was used and the mixing procedures are as follows. First, sand and cement were placed and 50% mixing water and half admixture were added and mixed for 1 minute. The remaining water and admixture and coarse aggregate were together added and mixed 2 minutes.

<p>| Table 1. Chemical composition of silica fume |</p>
<table>
<thead>
<tr>
<th>Composition</th>
<th>Percent</th>
</tr>
</thead>
<tbody>
<tr>
<td>Al₂O₃</td>
<td>0.5-1.7</td>
</tr>
<tr>
<td>SiO₂</td>
<td>85-95</td>
</tr>
<tr>
<td>Fe₂O₃</td>
<td>0.4-2</td>
</tr>
<tr>
<td>C</td>
<td>0.6-1.5</td>
</tr>
<tr>
<td>CaO</td>
<td>2-2.3</td>
</tr>
<tr>
<td>MgO</td>
<td>0.1-0.9</td>
</tr>
</tbody>
</table>

<p>| Table 2. Mix proportions of the Concrete |</p>
<table>
<thead>
<tr>
<th>Material</th>
<th>SF0.D</th>
<th>SF10.D</th>
<th>SF15.D</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>500</td>
<td>450</td>
<td>425</td>
</tr>
<tr>
<td>Water</td>
<td>150</td>
<td>150</td>
<td>150</td>
</tr>
<tr>
<td>Fine agg.</td>
<td>845</td>
<td>845</td>
<td>845</td>
</tr>
<tr>
<td>Coarse agg. (5-10 mm)</td>
<td>387</td>
<td>387</td>
<td>387</td>
</tr>
<tr>
<td>Coarse agg. (10-19mm)</td>
<td>528</td>
<td>528</td>
<td>528</td>
</tr>
<tr>
<td>Micro silica</td>
<td>0</td>
<td>50</td>
<td>75</td>
</tr>
<tr>
<td>Super plasticizer</td>
<td>7.5</td>
<td>7.5</td>
<td>7.5</td>
</tr>
<tr>
<td>Water/Cementitious material</td>
<td>0.3</td>
<td>0.3</td>
<td>0.3</td>
</tr>
<tr>
<td>Slump</td>
<td>150</td>
<td>150</td>
<td>150</td>
</tr>
</tbody>
</table>

*Aggregate in saturated surface dry condition
3. EXPERIMENT RESULTS

3.1. Compressive Strength

Strength development of the concrete mixtures versus age is shown in Figure 1. As it is shown from the early to later ages, compressive strength of the specimens with 10% silica fume is highest compare with two other specimens. However, as silica fume content increased to 15% rate of strength development in the early age was slow but, in the later ages of 28 and 90 days compressive strength of SF15 became higher than SF0. This may be attributed to the fact that in the early age, production of cement hydration, \( \text{Ca(OH)}_2 \), is not enough for activation of 15% silica fume. However, 10% silica fume may be an optimum value for cement replacement in the predetermined water cement ratio.

![Figure 1. Strength development of the concrete mixtures](image)

3.2. Hydration Heat Development

Temperature rising of the specimens after casting was monitored and is demonstrated in Figure 2. It is shown that regime of hydration heat in silica fume specimens is different compare with the specimens without silica fume. Temperature rising curve during early age hydration can be divided in 4 following pattern:

1: slope of heating zone
2: peak temperature value
3: time of peak temperature
4: cooling zone slope

For the silica fume specimens SF10 and SF15, slope of heat zone decreased compare with SF0. Furthermore peak temperature diminished about 8 to 10°C for 15% and 10% cement replacement. Occurrence of peak temperature postponed for silica fume specimens. An interesting result is that a mild slope is seen in cooling zone for silica fume specimens (Tables 3 and 4). This mild slope is desirable for lower thermal gradient.
Effect of Silica Fume on Hydration Heat and Thermal Expansion

Figure 2. Temperature rising during hydration process

Table 3. Peak temperature, heating and cooling slope of the hydration heat curves

<table>
<thead>
<tr>
<th>Mixture</th>
<th>Peak temperature. (°C)</th>
<th>95% peak temperature. (°C)</th>
<th>angle of tangent in temperature increasing (degree)</th>
<th>angle of tangent in temperature decreasing (degree)</th>
<th>slope of tangent (G₁)</th>
<th>slope of tangent (G₂)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SF0.D</td>
<td>49.3</td>
<td>46.80</td>
<td>68</td>
<td>38</td>
<td>2.48</td>
<td>0.78</td>
</tr>
<tr>
<td>SF10.D</td>
<td>40.0</td>
<td>38.00</td>
<td>58</td>
<td>23</td>
<td>1.60</td>
<td>0.42</td>
</tr>
<tr>
<td>SF15.D</td>
<td>40.5</td>
<td>38.50</td>
<td>48</td>
<td>29</td>
<td>1.11</td>
<td>0.55</td>
</tr>
</tbody>
</table>

Table 4: Net peak temperature and time of peak temperature for the mixtures

<table>
<thead>
<tr>
<th>Mixture</th>
<th>Concrete Temp. (°C)</th>
<th>Ambient Temp. (°C)</th>
<th>Concrete Temp. (°C)</th>
<th>Ambient Temp. (°C)</th>
<th>Time after casting (min)</th>
<th>Time after casting (hr)</th>
<th>Maximum net temperature rise. (°C)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SF0.D</td>
<td>24.2</td>
<td>21.1</td>
<td>49.3</td>
<td>20.5</td>
<td>1374</td>
<td>23</td>
<td>25.1</td>
</tr>
<tr>
<td>SF10.D</td>
<td>20.7</td>
<td>19.6</td>
<td>40.0</td>
<td>20.7</td>
<td>1873</td>
<td>31</td>
<td>19.3</td>
</tr>
<tr>
<td>SF15.D</td>
<td>19.8</td>
<td>21.0</td>
<td>40.5</td>
<td>19.5</td>
<td>1930</td>
<td>32</td>
<td>20.7</td>
</tr>
</tbody>
</table>

4. CONCLUSIONS
From the present study the following conclusions can be drawn:
- 10\% silica fume enhanced compressive strength of the concrete with 0.3 water cement ratio from early to later ages.
- Increasing silica fume content in the specimens did not lead to higher strength.
- Curve of temperature rising versus age was changed as cement was replaced with silica fume as follows:
  - Peak temperature decreased about 5\(^\circ\)C,
  - peak temperature postponed,
  - Slope of heating and cooling zone became mild; this may lead to a desirable low thermal gradient in mass concrete.

REFERENCES
ABSTRACT
Nowadays, many buildings and infrastructures are made of concrete and therefore, strength of concrete structural members ensures longer useful life and serviceability of the structure. Beams are one of the most important structural members that must have sufficient load bearing and ductility. Since brittle behavior and low ductility are two noticeable drawbacks of concrete members, they should be reinforced by materials with higher ductility and tensile strength such as steel fibers which can significantly improve these two characteristics of bending members. With this respect, use of industrial steel fiber deposits, percentage of fibers and optimized amount of fibers are critical factors which affect the flexural strength of beams and cost-effectiveness of the projects.

In this paper, experimental results of 3-point bending tests carried out on fiber reinforced concrete beams with different percentage of fibers are presented and compared and also its cost-effectiveness has been considered.

Keywords: concrete, steel fiber deposits, beam, ductility, flexural strength

1. INTRODUCTION
Applications of fiber reinforced concrete have been explained in different literature among which Keivani has mentioned the following items:
Floors cover:
It has been proved that using fibers can reduce the thickness of slabs and covers up to 1.2 times. With this respect, McCarran airport in Las Vegas in 1976 for airplane parking lot surface as wide as 52700m² was covered by steel fiber reinforced concrete of 15 cm thickness, while the required thickness was estimated as 37.5cm.
Rehabilitation of dams and hydraulic facilities:
In this application, the concrete resistance against impact and abrasion and cavitation is considered. According to ICOLD report, in order to prevent erosion and deterioration of stilling basins of Mayfield and Alder dams and also spillway of Little Goosc dam, fiber reinforced concrete as 38 to 45cm thick was used for cavitation prevention.
Instable slopes and trenches covering with sprayed concrete:
A kind of this application was used in a refinery in Sweden in an area of 4500m².
Explosion and impact resistant structures:
In such cases, the foundations of heavy machinery are of interest to be made of fiber reinforced concrete. With this regard, impact and crack resistance properties of concrete are outstanding.
Concerning flexural strength, studies have shown that fiber reinforced concrete behavior is significantly affected by shape, length to thickness ratio and material of fibers. The fibers can also be prepared from factories or from two resources such as industrial deposits each of which has significances and drawbacks. The common result is that in both cases, the flexural strength of concrete would be improved effectively. In this study, since the economic aspect of the projects was to be taken in to account as an important factor, effect of steel fibers from industrial deposits was investigated in concrete.

2. FLEXURAL TEST (PRISMATIC BEAM WITH THREE-POINT LOADING)
Since based on the studies done before, the most significant effect of steel fibers in concrete has been turned out to be ductility and energy absorption improvement especially in flexural behavior, this part of the research was of great importance. Because the previous studies were based on different standards (Keivani, based on Japan standard and most of other literature based on ASTM), referring to the standards was inevitable.
In this study, in order to maintain the best comparability and recurrence of the tests, direct reference to ASTM was necessary. The results of the tests performed will be presented in the following. All of the tests carried out were corresponding to ASTM C 1018 94b.

2.1. Specimens Preparation
The moulds used were of dimensions 10×10×50cm. at the first step, the moulds were cleaned and greased. The concrete ingredients was prepared according to determined mix design and mixed in the mixer and in order to make specimens with different percent of fibers, in each step, specified percentage of fibers was added to the mixture. after placing the concrete in the mould, vibration was applied to the moulds for a while so that enough compaction of concrete is reached. As a result of presence of fibers in the concrete, fiber reinforced specimens need more vibration time compared to plain concrete specimen to cause the air come out of the concrete. On the other hand, more vibration time causes segregation and bleeding. So, desirable duration of vibration was gradually obtained by some vibration trials.
The moulds were opened after 48 hours and specimens were kept in water basin (Figure 1) for 28 days in the laboratory conditions so that full curing was maintained. During this period, the temperature of about 20$^{\circ}$C and curing conditions were tried to be kept constant.
2.2. Test Apparatus
The conditions of test apparatus should be prepared according to ASTM C 1018. The apparatus used is illustrated in Figure 2. It is a kind of STRASSEN TEST, model 205. Its final load capacity is 100kN.

In order that the apparatus thoroughly meet the ASTM C 1018 provisions, some tools were installed. As shown in Figure 3, adjustable supports for load applicator of 30cm distance from the bottom were adjusted.

The load was applied by upper jaw in the middle of the span. The specimens were placed on the supports in order that 10cm of two ends of each specimen locate out of the supports edges. (Standard suggests span of at least 350mm but, loading span should be three times as the cross section dimensions i.e. 300mm).

Displacements were measured and recorded at three points (two support points and at the middle of span). The displacement was calculated by subtracting the average displacement measured at two supports from displacement in the middle of the span. After some tests, it was observed that the displacements in the supports after the first contact were nearly negligible and could be taken equal to zero. Loading was applied at the minimum rate (about 0.04 mm/min).
2.3. Tests
After preparation of the specimens for test implementation as mentioned above, three-point loading was applied by the apparatus with instrumentations shown in Figure 3. The measurements were repeated at appropriate time increments and for displacement of 0.02mm. During the test, regular reading of displacement and force gauges and also minimum rate of loading were maintained. In most of the specimens, large displacements and failure occurred rapidly after formation of the first crack (Figure 4). It would make the measurement a little difficult especially at the time near to the critical point.

Before implementation of the tests, the weights of the specimens were determined and dimensions were carefully measured. The average of parallel dimensions was selected as the dimension of prismatic beam base. The lengths of the specimens were also measured. After start of loading, measurements were recorded at each 0.02mm displacement. Failure was finally completed by splitting of the specimen (Figure 4).

At the failure time of the specimens containing 0 to 3% of fibers, it was observed that two parts of the specimen were completely separated. But concerning the specimens containing 5% of fibers, the connection and bonding between two parts of the specimen was still maintained. Visual inspection of specimens after failure (Figure 5) showed that regarding the fibers, most of them did not fail due to lack of tensile strength but as a result of low bonding between concrete and fibers, they were pulled out of the concrete.

From one hand according to ASTM, the ratio of fiber length to the smallest dimension of the mould is limited and On the other hand, the ratio of length to thickness of fibers is of great importance. With this respect, the more the length of fibers, the better bonding and involvement between fibers and concrete, and tensile failure of fibers is more probable. But long fibers can not be mixed easily in the concrete and especially their distribution in the mould would not be uniform. It can affect the results of flexural tests.
According to this limitation, length of 2.5-3 cm was chosen for fibers. Considering the cross section dimensions of the mould (10*10), this length could cause a little nonuniformity especially in low percentage of fibers. But in higher percentages, visual inspection of cross section revealed uniform distribution of fibers in the
specimen cross section (Figure 6).

Figure 7 shows the specimens with 5% of fibers after failure. It is illustrative that despite of full failure for the three specimens, the bonding between two parts of the specimens has yet been maintained via the fibers. If distribution of fibers in the failure location is considered, it will seem desirably uniform. Figures 8 and 9 show illustrative views of fibers distribution in failure surface of the specimens. In short, these observations show that the fibers have been pulled out of concrete and did not split due to excessive tensile stress. Also appropriate distribution of fibers can obviously be seen in the Figures. At the end corner of specimen (where bonding is yet present), it can be considered that the connection and bonding is conserved via just a few number of fibers and in spite of complete failure of concrete, the strength and bonding of the remained fibers is sufficient to carry the weight of specimen (Figure 7). These observations may be actually efficient in selection of length, thickness and particularly shape of fibers.

Figure 8. horizontal view of the failed specimen with 5% of fibers
2.4. Results and Discussion
After implementation of each test, the displacements in the middle of the span (mm) with respect to the corresponding force (kN) prepared in a table. Table 1 is an example of such tables. In the first column of the table, the values recorded at the time of test are presented. Displacement values for each 0.02mm are presented for which the corresponding forces are also presented in the next column. Such tables were prepared for all of the tests.

<table>
<thead>
<tr>
<th>Deflection (mm)</th>
<th>Primary values</th>
<th>Final values for curve plotting</th>
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</tr>
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</tr>
<tr>
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<tr>
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<tr>
<td>0.38</td>
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</tr>
<tr>
<td>0.82</td>
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</table>
In all of the tests, displacements increased and recorded as nearly small amounts and approximately proportional to the forces before the maximum strength point. It should be noticed that each reading step included three displacement readings and one force reading. Displacements were measured at the supports and at the middle of the span and absolute amount of displacements at the middle of the span were calculated relative to the supports displacement. In practice, it was observed that displacement values at the supports were stopped after start of the tests. Also the displacement values need to be corrected at the beginning of the test as follows. At the beginning of the test, loading jaw cannot thoroughly be attached on the specimen surface. This is due to the fact that if it is attached, some bending may be created in the specimen before the displacement gauge is adjusted to zero. However, it should be noticed that specimens without smooth surfaces produce even more errors in measurements which can be removed by ignoring the earlier part of the displacements (strains).

**Figure 9.** Vertical view of the failed specimen with 5% of fibers

**Figure 10.** Load-deflection curves of three specimens with 1% of fibers
Table 2: characteristics of prismatic specimens

<table>
<thead>
<tr>
<th>specimen characteristics</th>
<th>fiber content</th>
<th>length</th>
<th>cross section</th>
<th>density</th>
<th>Apply. ultimate strength</th>
<th>Ultimate strength</th>
<th>Ultimate deflection</th>
<th>first crack strength</th>
<th>max. first crack strength</th>
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correcting the primary readings, were used to plot the final load-deflection curve. Load-deflection curve of three specimens with equal percentage of fibers were plotted together for comparison. For example Figure 10 shows load-deflection curves of three specimens with 1% of fibers. From the tables and curves prepared and corrected, all of the critical information has been obtained and presented in Table 2. Table 2 presents all of the specimens characteristics in different tests. Specimens number and fibers percentage are presented in the first and second columns, respectively. Cross section area and weight of the specimens are given in next three columns. It can be concluded from the table that the more the fibers in the concrete, the lower the slump would be.

3. CONCLUSION
Considering the curves obtained from various tests, the following results can be concluded:
- The results of tests implemented on specimens with equal percent of fibers are in good agreement so as the average of three tests can be taken as the representative of that fiber percentage.
- Although the first correction may cause a little difference in the tests results but, first contact error and support corrections are much more effective than the remained error.
- Deformation is very slow until near point to the failure but after appearance of the first crack, it increases and with a sudden large deformation, specimen fails and the force decreases.
- Final flexural strength of steel fiber reinforced concrete in the beam test with three-point loading increases significantly. For example in 3% of fibers, the strength increase up to 70%
- Amount of maximum deflection increases by increase of fibers percent and in 3% of fibers, it approaches 95%.

REFERENCES
2. Concrete International; “state of art report on fiber reinforced concrete” reported by ACI committee 544, ACI Journal nov.1973- copyright by concrete international / may 1982.
A STUDY ON BONDING STRENGTH OF POLYMERIC FIBERS TO CEMENTITIOUS MATRIX

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ABSTRACT

Cementitious materials are brittle in nature. Due to this behavior, short, randomly and distributed fibers are mostly being used to reinforce cementitious materials. Added fibers enhance tensile strength and flexural toughness and reduce crack creation and propagation in cement matrix. The major effect of fibers is to act as bridging at crack tips to resist crack propagation. Fiber bonding to cement paste is an important factor that affects performance of the fiber reinforced cementitious composite (FRCC). Bonding energy (adhesion) between these materials is composed of interfacial interactions (chemical bonding) and mechanical interactions (interlocking). The adhesion of fiber to cementitious materials can be evaluated by pull-out test. This test is the newest method and one of the most commonly used practical methods to evaluate performance of fibers in FRCCs. This article presents the bonding strength results of commercial polymeric fibers to cement matrix. To investigate adhesion of polypropylene (PP), nylon66 (N66) and acrylic (PAN) fibers to cement matrix a single fiber pull-out test setup is designed and fabricated. The specimens were prepared at the water-cement ratio of 0.4 and they were tested at 7, 14 and 28 days of curing. Fiber's surface after pull-out test was also studied by microscopic analysis. Some interesting results were obtained from the pull-out test of different fibers. On the basis of results, it was found that fiber should be selected for FRCC reinforcement due to their mechanical interactions and physical/chemical/mechanical behavior in cement matrix instead of their chemical interactions.

Keywords: adhesion, pull-out test, interfacial interactions, polymeric fibers

1. INTRODUCTION

The application of fibers to reinforce cementitious materials is an ancient subject. At first, asbestos fibers were used in industrial process to produce fiber reinforced cement sheets. Because of their great fiber strength and durability, high physical and chemical resistance, none-combustibility and resistance to weathering attack and cost effectiveness, they were used as building material during the last century with various forms and styles to suit different needs. Despite of these properties,
they can cause a major health hazard to human's safety [1]. Hereafter, various types of synthetic fibers were produced and used as asbestos substitutes. The performance of FRCC depends on many factors, such as fiber material properties (fiber strength, stiffness, and Poisson’s ratio), fiber geometry (fiber surface and cross section), fiber volume content, matrix properties (matrix strength, stiffness, Poisson’s ratio), and interface properties (adhesion, frictional and mechanical bond) [2].

Bonding depends on the structure of the fiber-matrix interface. Fiber bonding to the cementitious matrix is an important and effective parameter on fiber reinforced cement composites. Also, the performance of fiber reinforced composites is strongly related to the debonding/pull-out behavior of the fibers. For this purpose, the relationship between the pull-out load and the displacement of a fiber, when it is pulled out from the cement matrix, serves as an important parameter in the design of cement composite materials.

Many researchers have been done on the evaluation of bonding between fibers and cement matrix [3-5, and 6]. Some methods and equipments were adapted for pull-out test to evaluate fiber/cement bond strength in the present work. Fiber pull-out behavior contributes to the energy absorption ability of fibers in FRCCs. Fiber to cement bonding allows stress transfer between them. Regarding to the importance of this behavior in composite materials, fiber/cement interface has been studied in this research.

The aim of the present work is to characterize the bonding mechanisms of polymeric fibers to cement matrices. To determine the bond strengths of polymeric fibers to cement matrix, pull-out test was employed. The test setup was basically similar to the numerous techniques that have been developed by previous researchers. Besides the testing setup, it is also important to understand the way that pull-out specimens are prepared. A new technique for preparing specimens for pull-out test was suggested in this work. Fiber pull-out specimens were prepared with single filaments of PP, N66 and PAN fibers. The surface of the used fibers after pull-out test was evaluated by optical microscope (OM). The effect of fiber types on the pull-out results of fiber/cement matrix at different ages of curing was also studied.

2. MATERIALS AND EXPERIMENTS

Cement used in this study was ordinary Portland cement type II. The type of used synthetic fibers and their properties are given in Table 1. Figures. 1-3 show the optical microscopic images of the longitudinal and cross-sectional surface of the fibers.

<table>
<thead>
<tr>
<th>Fiber type</th>
<th>Diameter (µm)</th>
<th>Density (gram/cm³)</th>
<th>Tensile strength (MPa)</th>
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</thead>
<tbody>
<tr>
<td>PP</td>
<td>25</td>
<td>0.91</td>
<td>326</td>
</tr>
<tr>
<td>N66</td>
<td>26</td>
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<td>PAN</td>
<td>40</td>
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<td>344</td>
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</table>
2.1. Specimen Preparation
Specimens for pull-out test were prepared by the equipment that has been designed and made for this research, as shown in Figure 4. The specimens were prepared
with a matrix made by 0.5 of water to cement ratio. After demolding, specimens were subjected to cure in the condition of 23 ± 2°C and 100 ± 5% of relative humidity. Pull-out tests were carried out on specimens after 7, 14 and 28 days of curing. The embedded length for all series was 10mm long. Figure 5 shows the pull-out specimen before test.

2.2. Pull-Out Test
To investigate the bonding characteristics, single fiber pull-out test was performed. The pull-out tests were carried out by an Instron testing machine (Tinius olsen) at the crosshead rate of 0.02 mm/s as shown if Figure 6. The schematic representation of the test set-up can be seen in Figure 7. The free length of single fiber was 10mm. Load–displacement data of pull-out process were obtained and plotted by computer.
3. RESULTS AND DISCUSSION

3.1. Pull-Out Test Results
The pull-out behaviors of all series are illustrated by the load-extension curves in Figures, 8-10. In all series, it was observed that pull-out force is increased by increasing in fiber displacement to a maximum force. Thereafter, it decreases to zero level because of fiber slippage, pulling out or failure.

The analysis of N66 fibers load-displacement curves shows that there is no significant difference between pull-out load at 7 and 14 days. It can be said that the cement microstructure is not significantly changed during curing period from 7 to 14 days. The bonding strength in 28 days is remarkably higher than 7 and 14 days. After complete debonding of specimens, N66 fiber begins to slip-out, so pull-out force is decreased.

Increasing the cement curing age of specimens containing PP fiber from 7 to 28 has a positive effect on pull-out load, as shown in Figure 9. Pull-out curves for PP specimens demonstrate that there is no significant difference between 7 and 14 days specimens. Pull-out curve at 28 days indicates that mechanical bonding between PP fiber and cement matrix is maximum. The pull-out curves show that the fiber/matrix bond strength gets close to fiber tensile strength.

In case of PAN fibers, increasing in curing period from 7 to 14 days has no significant effect on pull-out load. At 28 days of curing time, fiber failure happens during pull-out process because of higher bonding strength to cement matrix, as shown in Figure 10.

In all series, increasing curing ages from 7 to 28 days, improves bonding strength. In general, improvement in cement hydration results in decreasing of the porosity of hardened paste. The cement maturity has direct effects on the fiber/matrix bond properties.

In the transition zone more nucleation sites and open space are available in the around of the fiber surface. Based on this microstructure, CH layer in contact with
the fibers grows much faster than the cement bulk. As reported by Chan [7], the transition zone is considerably weaker than the cement bulk due to large CH crystals and higher porosity. In other word, longer cement age and consequently the increase of hydration degree results in a decrease in the porosity and finally stronger fiber/cement interface.

Figure 8. Pull-out behavior of N66 fiber at different cement ages

Figure 9. Pull-out behavior of PP fiber at different cement ages
Figure 10. Pull-out behavior of PAN fiber at different cement ages

Figure 11. Pull-out curves of tested fiber at 28 days cement curing

Figure 11 shows pull-out curves of different fibers at 28 days of curing. It can be seen that PAN fibers showed higher pull-out strength in comparison to PP and N66 fibers. Figure 12 shows longitudinal image of pulled-out PAN fiber. It is evident that some cement particles are present on fiber surface. Due to the non-round shape of these fibers, during pull-out process, mechanical bonding can be performed because of interlocking effect to cement matrix. The special shape of cross-section indicates higher specific surface than round shape fibers. In other word, PAN fibers have much contacting surface to cement matrix which leads to
increasing frictional resistance during pull-out. In the case of other fibers (PP and N66), the smooth surface and round shape of fibers causes less friction. However, in specimens containing PP, due to hydrophobic properties of PP and bleeding of cement paste, water is collected on the surface of the fiber. Therefore, calcium hydroxide (CH) coarse crystals are produced at the PP/cement interface. These crystals are enough big and coarse to deform PP surface. So, during pull-out, PP fibers interlock to these crystals.

3.2. Microscopic Analysis
The surfaces of pulled-out fibers were analyzed using optical microscopy. Figure 12 shows the chemical adhesion between cement bulk and PAN fibers. Due to the affinity between PAN fiber and cement paste which are both hydrophilic, chemical adhesion can be produced. These observations and the image of pulled-out PAN fibers indicate that PAN fibers have both chemical and mechanical bonding to cement paste.

Study on the pulled-out PP fibers from the cement matrix with optical microscope (OM) reveals the mechanical bonding of PP fiber to cement paste due to fiber deformation and elongation (Figure 13). Deformed points at fiber surface resist to
fiber pull-out and thus, pull-out load are increased. Generally, the force and energy of fiber pull-out are increased with the presence of interlocking points between fibers and cement matrix, but this increase is limited by fiber tensile strength. In general, the friction bonding is changed with fiber deformation at fiber embedded length.

As shown in Figure 14, the evaluation of N66 pulled-out fibers shows that cement particles attach to fiber surface. Based on this observation, it's found that N66 fibers have chemical bonding to cement matrix. Microscopic analysis also demonstrates that surface of N66 fibers have not been deformed.

4. DISCUSSION

Regarding to the pull-out behavior of fibers, it is resulted that N66 fiber has lower bonding strength to cement matrix compared to PP fibers. Microscopic analysis demonstrates that PP fiber has no chemical bonding to cement matrix while the presence of cement hydrates particles on the surface of N66 fibers is observed. Based on the observation, it is found that mechanical bonding is more effective than chemical bonding in fiber/cement matrixes.

PAN fibers have both mechanical and chemical bonding to cement matrix, due to their hydrophilic nature and cross section shape. Thus, the bonding strength for this fiber is higher than other studied fibers (Figure 11).

It should be noted that mechanical bonding in fiber/cement interface has an important role to enhance the mechanical performance of cement composite materials.

5. CONCLUSION

- The new pull-out sample preparation method was introduced in this research on the basis of single filament pull-out test.
- Increasing cement curing period from 7 to 28 improved bonding strength for all fibers. In general, the increase in the degree of hydration resulted in the decrease of hardened cement paste porosity.
- The imaging of all fibers surface showed that N66 and PAN fibers had chemical adhesion to cement matrix. The observation of propylene fiber surface confirmed its deformations.
- PAN fibers showed to have better bonding behavior to cement matrix because of mechanical and chemical adhesion. N66 fibers had weaker bonding action with cement paste in comparison to PAN and PP fibers.
- In spite of the lack of chemical adhesion between cement paste and PP fibers, high pull-out force was registered due to the mechanical interlocking.

REFERENCES


PREVENTION OF DELETERIOUS ASR BY ASSESSING AGGREGATES AND SPECIFIC CONCRETE MIXTURES

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ABSTRACT
Alkali-silica reaction (ASR) is a major world-wide durability problem and concretes exposed to external alkalis are particularly endangered. In Germany, concrete pavements have a long tradition but the experiences over the last years showed, that the currently standardized test methods are not able to consider externally supplied alkalis from deicers sufficiently. Especially alkali acetate and formate based deicers as used for airfields turned out to be extremely deleterious. For this reason, an ASR performance-test that was developed at the Finger-Institute (FIB) is used for some years now to assess specific concrete job mixtures regarding their ASR potential under consideration of externally supplied alkalis. The lab-field correlation of this performance-test was assessed by testing two reproduced pavement concretes that showed ASR-distress in the field after 8-12 years in service. It has been shown that the performance-test was able to assess the tested concretes correctly. Contrary, the mortar-bar test could not predict the field performance of the aggregates correctly in all cases.

Keywords: alkali-silica reaction, performance-test, deicer, pavement concrete

1. INTRODUCTION
According to the European standard EN 206-1, sufficient concrete and unit durability is guaranteed when the recommended requirements are met. For the most durability problems ample long-term experience exists and appropriate recommendations are provided. However, ASR occurred with increasing frequency to pavement concretes for highways and airfields in Germany [1, 2, 3] but also in the USA [4, 5] in the past few years despite of following standards and additional recommendations. This shows that the currently standardized ways and test procedures to characterize the reactivity of aggregates and to evaluate the durability of concrete were insufficient to avoid ASR-damage. Especially the deleterious influence of alkali-containing deicers is not well integrated in present ASR-test procedures.
Pavement concretes are one of the most stressed concrete types known. Placing must be done under the given weather conditions, curing is difficult and in central Europe, pavements are exposed to changing temperature and moisture conditions, freeze-thaw cycles as well as deicers. Moreover, permanent dynamic traffic loads create stresses and pre-damages (e.g. micro cracks) that support the progress of
deterioration processes by the ingress of water and deicer. Especially the heavy vehicle traffic plays an important role and increases year by year. All these factors support a possible ASR more or less, depending on how well the concrete mixture is optimized from the very beginning regarding ASR. A typical ASR-distress on a pavement concrete containing slow/late reacting aggregates occurs after 10-12 years in service. Repair often means complete replacement of the damaged concrete, long before the designed service life of about 30 years is reached. The current situation with increasing numbers of ASR-damages on pavements shows clearly the necessity of an ASR performance-test for pavement concretes that will be exposed to external alkalis.

2. MATERIALS AND METHODS

2.1. Materials

Two damaged German highway pavement concretes were examined by thin section analysis and it was found, that ASR was the main reason for the distress (Figure 1, Figure 2). In concrete 1, the gravel and rhyolite aggregates were affected by ASR. The concrete contained 360 kg/m³ of ordinary portland cement with an Na₂Oeq of about 0.95 wt.-% and a w/c of 0.44. The concrete was air-entrained with 4.5-5.0 % air. The damage occurred after 12 years in service. In concrete 2, the used granodiorite aggregates were clearly affected by ASR and first damage occurred after 8 years in service.

![Figure 1. Highway pavement concrete 1, 14 years old, cracks running through aggregate grains, partially filled with ASR gel (Image: Ernst Freyburg, Doreen Erfurt)](image-url)

The objective was to examine, if the FIB cyclic climate storage as ASR performance-test assesses the ASR potential correctly for these specific concrete mixtures compared to the field experience. Therefore, concrete 1 was reproduced in the laboratory exactly as possible, by using all the coarse and fine aggregates from the original deposits and a comparable portland cement with a Na₂Oeq of 0.90 wt.-%. For concrete 2, the granodiorite aggregates from the original deposit were used in a typical mixture that meets all the current requirements for pavement
concretes in Germany [6, 7]. It must be noted of course, that the geological situation in the deposits may have changed more or less over the time, so that the reactivity of the new batches might deviate from the material used 15 years ago. To evaluate this influence, the coarse rhyolite aggregates from the cores of concrete 1 were extracted for a mortar-bar test by means of shock-wave crushing [8]. For concrete 2, the available material was not enough to perform the mortar-bar test.

2.2. Mortar-Bar Test
Prior to the performance-test, new batches from the deposits as well as the extracted coarse rhyolite aggregates from the core of concrete 1 were tested with a mortar-bar test according to the German alkali-guideline [Error! Bookmark not defined.]. The aggregate is crushed and sieved to obtain 450 g of the grain size fraction 0.5-1 mm and 1-2 mm respectively. Both grain size fractions are mixed and 450 g of innocuous quartz sand (0.1-0.5 mm) is added. The mortar bars (4×4×16 cm) are prepared according to DIN EN 196-1 using a high-alkali portland cement (Na₂O_eq = 1.3±0.1 wt.-%) and a w/c ratio of 0.50. NaOH is added to the mixing water to gain a total Na₂O_eq of 2.5 wt.-% in order to boost alkalis and pH. After 1 day curing at 20°C and > 95 % RH, the mortar bars were demolded and initial length and mass were measured. Afterwards, the bars were stored at 70°C above water until the 28th day. The expansion limit for this mortar-bar test is 1.5 mm/m after 28 days.

According to the German alkali-guideline, the tested aggregates are considered as suitable for pavement concretes without any further testing, if they pass the mortar-bar test. If the aggregates fail the test, further testing (e.g. concrete prism test) can be done.

2.3. ASR Performance-Test
Since 2001, an alternating climate test method (cyclic climate storage) is used at
the FIB for accelerated simulation of Central European climatic conditions, in order to assess the durability of specific concretes for outdoor structures [9-12].

Both concrete mixtures were prepared without added alkalis (unboosted) and were air-entrained with 4.5-5.0 % air. Concrete prisms (100×100×400 mm) were cast from each mixture with embedded stainless steel studs for the expansion measurements. After 24 hours, the prisms were demolded, wrapped airtight in polyethylene foil and stored for 5 days at 20°C. Subsequently, a flexible foamed rubber tape was glued around the upper edges of the prisms to form a railing that will keep the NaCl solution. At the 7th day after casting, the cyclic climate storage was started. Three prisms of every mixture were applied with the NaCl solution (0.6 mol/l) and three more prisms with distilled water for control (Figure 3). In a special walk-in climate simulation chamber (Feutron, Type 3705/04, Figure 3) the concrete prisms were stored under defined cyclic alternating temperature and moisture conditions. One cycle lasts 21 days and consists of 4 days drying at 60°C (< 10 % RH), 14 days fog at 45°C (100 % RH) and 3 days of freeze-thaw-cycling between +20 and –20°C (Figure 4).

Figure 3. Climate simulation chamber and concrete prisms with NaCl solution (0.6 mol/l)

Figure 4. Scheme for one cycle of the cyclic climate storage
At the end of the first drying phase, initial length and weight are measured and 400 g of test solution (deicer or water respectively) is applied on every prism for the first time and remains on the prisms until the end of the cycle. After the cycle is completed, the test solution is removed to measure length change and weight of the prisms and is placed back again when the readings were taken. All measurements are done at 20°C. During the second drying phase, the test solution evaporates completely, leaving behind minor solid residues from the deicer as well as leached substances from the concrete, e.g. alka lis. At the end of the second drying phase new test solution is applied. In this way the cyclic climate storage continued until 9 cycles (7 month) were completed. For pavement concretes exposed to deicers it was found that 8 cycles (6 month) are usually sufficient to assess the potential regarding a deleterious ASR for a typical service life of 20-30 years. The expansion limits after 8 cycles were defined with 0.5 mm/m for application of deicer solutions (higher moisture impact) and with 0.4 mm/m for application of water only.

3. RESULTS

The mortar-bar test results show that the new batches of the rhyolite as well as the extracted rhyolite from the core are clearly reactive. The new batches (8-16 mm, 16-22 mm) show a slightly higher expansion than the original material from the cores. Gravel 2-8 mm and sand 0-2 mm are from the same deposit and are clearly reactive. All the granodiorite aggregates (2-8 mm, 8-16 mm, 16-22 mm) stayed below the limit and are non-reactive (Figure 5).

For the reproduced concrete 1 with the gravel and the rhyolite, the cyclic climate storage shows that the expansion exceeds the limit of 0.5 mm/m after 7 cycles if exposed to NaCl solution but not if exposed to water only (Figure 6).
Concrete 2, the typical pavement mixture with the granodiorite aggregates, shows deleterious expansion after 8 cycles if exposed to NaCl solution, but no critical expansion occurred if exposed to water (Figure 7). A subsequent thin section analysis provided clear evidence for an ASR (cracks, ASR-gel), triggered by the granodiorite (Figure 8, Figure 9).

If non-reactive aggregates are being used, no deleterious expansion occurs, no matter if alkali-containing deicer solutions or water is applied. As an example, Figure 10 shows the expansion of a pavement concrete with a non-reactive andesite...
exposed to different deicer solutions.

Figure 8. Concrete 2 after the cyclic climate storage, granodiorite grain (G) with micro cracks (Image: Ernst Freyburg, Doreen Erfurt)

Figure 9. Concrete 2 after the cyclic climate storage, pore with ASR gel next to a granodiorite (G) grain (Image: Ernst Freyburg, Doreen Erfurt)

Figure 10. Cyclic climate storage for a pavement concrete with a non-reactive andesite
4. DISCUSSION
Since 2004, more than 130 concretes, mostly job mixtures, with different cements and aggregates were tested with the cyclic climate storage. The objective in this study was to verify, if the ASR potential of two concrete job mixtures can be assessed correctly with the cyclic climate storage compared to field performance, where both concretes showed ASR-distress after 8-12 years in service.

For concrete 1, already the mortar-bar test results showed that the gravel and the rhyolite aggregates are reactive and that the new batches and the material in the cores are comparable. The cyclic climate storage showed correspondingly that after 7 cycles and exposed to NaCl deicer solution a deleterious ASR occurred. This result corresponds well to the field performance, where ASR-distress with that specific concrete mixture occurred after 12 years in service.

For concrete 2, the granodiorite is non-reactive according to the mortar-bar test and a portland cement with Na$_2$O$_{eq}$ $\leq$ 0.80 wt.-% was used. Finally, concrete 2 meets all the requirements of the current regulations for pavement concretes [12, 13], but deleterious expansion occurred in the cyclic climate storage after 8 cycles exposed to the NaCl deicer solution. Concrete 2 was assessed correctly with the cyclic climate storage compared to field performance, because in the field ASR-distress occurred after 8 years in service. For the original highway concrete with the granodiorite, a portland cement with Na$_2$O$_{eq}$ of 0.9-1.0 wt.-% was used according to the former regulations (Na$_2$O$_{eq}$ $\leq$ 1.0 wt.-%). But also by following the new regulations for pavement concretes (passed mortar-bar test and Na$_2$O$_{eq}$ $\leq$ 0.80wt.-%), ASR-distress must be expected when using that specific granodiorite. Hence, the mortar-bar test was not able to assess the granodiorite correctly. The mortar-bar test has been available in Germany since 2005 and was introduced in the German alkali-guideline in 2007. Since the mortar-bar test is used for assessing the aggregates for pavements, the risk of ASR should have become lower for pavements build after 2005. But the risk is not eliminated completely, because the mortar-bar test does not asses every aggregate correctly.

It must be noted generally, that the mortar-bar test provides in many cases an acceptable correlation with the performance-test for highway pavement concretes, i.e. under exposure of NaCl deicer solution. But the presented results demonstrate that there are exceptions, as also reported in other studies [13]. The situation is much more serious for airfield pavement concretes, where deicers based on alkali acetates and formates are used instead of NaCl. Concrete 2 (Figure 7) and a concrete with non-reactive andesite aggregates (Figure 10) were also tested with such airfield deicer solutions. Compared to NaCl, this resulted in a much faster and higher expansion for concrete 2 with the supposed non-reactive granodiorite according to the mortar-bar test. Meanwhile, a modified ASTM C 1260 mortar-bar test is recommended from the FAA [14], which might provide better predictions for airfield concretes as was also found for the tested granodiorite and andesite (Figure 11), but further research is needed with this test.

It was found in a recent study that the solubility of portlandite increases in presence
of alkali acetates and formates due to the formation of strong calcium acetate and calcium formate complexes respectively (Figure 12). Thus, more and more OH$^-$ ions will be released gradually which results in an increase of the pH and consequently in an accelerated attack of reactive aggregates [15]. This also means that the mortar-bar test alone is not sufficient to assess the suitability of aggregates for use in airfield concrete pavements, because the mechanism of ASR in presence of alkali acetates and formates is considerably different from the mechanism in presence of NaOH or NaCl. The risk of an underestimation of the aggregate reactivity based on a mortar-bar test is much higher in this case, why a performance-test is highly recommended. Even low-alkali cements are not a reliable countermeasure in this specific case to prevent a deleterious ASR permanently (Figure 13).

Figure 11. Modified FAA-test, mortar-bars submerged in K-formate deicer solution at 80°C

Figure 12. Species distribution in a saturated solution of Ca(OH)$_2$ with addition of Ca$^{2+}$, Ca(CH$_3$COO)$^-\cdot$H$_2$O, Ca$^{2+}$, CaOH$^-$, Ca$^{2+}$, Ca(CH$_3$COO)$^-$.
5. CONCLUSIONS

In concrete with reactive aggregates ASR is initiated and accelerated highly, if exposed to alkali-containing deicers. The FIB cyclic climate storage is used as a performance-test, considering the influence of external alkalis on ASR. In this study, the cyclic climate storage was compared to field performance. It has been shown that two reproduced highway pavement concretes were assessed correctly regarding their ASR potential. Both concretes and aggregates respectively showed ASR-distress in the field after 8-12 years in service. After 7-8 cycles (5-6 month), the concretes failed the cyclic climate storage if exposed to NaCl deicer solution, in one case despite of following all the current requirements for pavement concretes.

Mortar-bar tests are suitable for a first short-term assessment of the reactivity of aggregates, but some aggregates may be classified incorrectly, especially for the use in concrete pavements. Airfield concrete pavements, exposed to alkali acetate and formate based deicers are particularly endangered. Because of the different mechanism, mortar-bar tests will not be reliable enough in that specific case. Even low-alkali cements are not a reliable method to avoid deleterious ASR if reactive aggregates are used and the concrete will be exposed to alkali-containing deicers, especially based on acetates and formates.

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PERFORMANCE OF NOVEL COVENTRY BINDER AS CEMENT REPLACEMENT

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ABSTRACT
Global warming due to emission of green house gasses is one of the main challenges in the twenty first century. The industrial activities have a major impact on global warming due to emission of a large quantity of green house gasses by industries particularly cement manufactures. Production of one ton of Portland cement produce approximately one ton of carbon dioxide. In addition a large quantity of good quality natural limestone is used in production of cement which leads to significant reduction in natural resources. A novel cementitious material (Coventry Binder) was developed at Department of Civil Engineering, Coventry University using 100% industrial wastes (i.e. Basic Oxygen Slag, Plasterboard gypsum waste and cement by pass dust). This paper presents the result of investigation on performance of the novel binder as cement replacement. Paste, mortar and concrete samples were prepared with various proportions of Coventry binder, Portland cement and run of station ash. The compressive strength and density of samples were measure at 3, 7, 28 and 90 days. It was found that paste mixes containing 5% and 10% of Coventry Binder in binary system Coventry Binder-OPC gained higher strength at 28 days than OPC samples. Increasing Coventry Binder content in paste mixes results in a considerable decrease of compressive strength.

Keywords: pozzolanic materials, plasterboard gypsum waste; sustainability, basic oxygen slag, Coventry binder, run of station ash, compressive strength

1. INTRODUCTION
In the 21st century one of the most significant and important problems is Global Warming. It can be observed that rapid changes in weather is happening around the world i.e. hurricanes, typhoons, floods, and droughts which cause several damage such as forests fire and agricultural problems. These weather phenomena are the effect of increases in carbon dioxide in the atmosphere, referred to as “Greenhouse gasses”. In the Earth’s atmosphere there are many chemical components, some of which have a natural origin e.g. water vapour, carbon dioxide, methane and nitrous dioxide. Some of these components are artificial i.e., man-made gases used in aerosols [1]. The air pollution started with the Industrial Revolution in the late 18th century. Then the whole world started developing. The manual labour began to be replaced by machinery and all these machines were working using steam power.
which was getting by coal combustion. This was the main source of CO₂ emission which was released into the atmosphere [2]. Nowadays this problem still exists and even arises as a result of the human convenient and comfortable lifestyle and also because the developing countries i.e. India, China, Eastern Europe are contributing. The Economic policy of each country is close related to Greenhouse gas emissions. Especially developing countries and big countries like the USA have a huge demand for production energy by industry, transportation and in construction industrial processes, such as the production of cement [3]. These economic fields cause the major pollution. It is a major problem to deal with because the impacts of pollutants emitted in one country have a direct impact on citizens from other countries. Industrial processes have a major influence on carbon dioxide emissions and global warming. It is therefore imperative in construction industries to find alternative materials with low environmental impact to help in reducing this negative and dangerous phenomenon [4]. By producing 1 ton of cement, one ton of CO₂ and other gasses are emitted. Concrete is the most popular construction material in the world [5]; however, concrete only exists with cement so novel and modern cementitious materials must be developed to replace the ordinary Portland cement. Most importantly, these new materials must be environmental friendly and also help to utilise various industrial wastes in order to minimise the consumption of the natural sources such as limestone [5]. Development of such a material will also help to reduce the landfills. The majority of waste materials from construction and demolition are currently landfilled [6]. Recycling and reusing waste materials can be an effective solution for escalating problem of landfills [4]. The Civil Engineering Department at Coventry University has been developing a novel binder which is entirely made from waste materials [7]. The developed novel binder was successfully used for construction of road-base, sub-bases and soil stabilisation. The aim of this research is to evaluate the performance of the novel binder which is referred to as ‘Coventry Binder’ as cement replacement. Comparison will also be made for strength of mixes containing the novel binder with mixes made with other pozzolanic materials including BOS and run of station ash (ROSA).

2. EXPERIMENTAL PROGRAMME

2.1. Materials

Plasterboard Gypsum (PG) used for this project was collected from demolition and reconstruction activities. Gypsum is major component of modern buildings so waste from construction contains up to 30% of gypsum drywall scraps by weight [7]. Plasterboard waste was crushed by grinders and sieved through a 600 micron sieve. The powder was then stored in a sealed bucket [8]. Basic Oxygen Slag (BOS) is a non-metallic by-product of steel production. The slag was grounded by using a laboratory ball mill and after that sieved through a 600 micron sieve. By Pass Dust (BPD) is obtained from kiln bypass in cement industry. By pass dust is the waste highly alkaline materials of Portland cement manufacture. It is generated during the calcining process in the kiln [9]. Run of station ash (ROSA) is an unclassified ash collected from chimney stack of power stations. This is pozzolanic
in nature and reacts with calcium hydroxide and alkalis to form calcium silicate/aluminate hydrates which are cementitious compounds. Coventry Binder is a blended mixture of 15 percent Plasterboard Gypsum (PG), 80 percent Basic Oxygen Slag (BOS) and 5 percent By Pass Dust (BPD). Coarse and fine aggregates used in mortar and concrete mixes were of natural source and complied with BS 812.

2.2. Mix Proportions
A large number of paste samples were made during this investigation. The word “paste” which will be using in this paper means a mixture of resembling cementitious powder and water without aggregate. The proportions of pastes used in investigation were design in order to optimize the mixture ingredients to achieve the highest compressive strength. The pastes mixes were prepared in five groups. The mix proportions of mixes are shown in Tables 1 to 4.

<table>
<thead>
<tr>
<th>Coventry Binder [%]</th>
<th>OPC [%]</th>
<th>W/B</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>95</td>
<td>0.3</td>
</tr>
<tr>
<td>10</td>
<td>90</td>
<td>0.3</td>
</tr>
<tr>
<td>20</td>
<td>80</td>
<td>0.3</td>
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<tr>
<td>40</td>
<td>60</td>
<td>0.3</td>
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<tr>
<td>60</td>
<td>40</td>
<td>0.3</td>
</tr>
<tr>
<td>80</td>
<td>20</td>
<td>0.3</td>
</tr>
</tbody>
</table>

Table 2: Mix proportions for PG-BOS-OPC paste mixtures (Group 2)

<table>
<thead>
<tr>
<th>PG [%]</th>
<th>BOS [%]</th>
<th>OPC [%]</th>
<th>W/B</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>85</td>
<td>5</td>
<td>0.3</td>
</tr>
<tr>
<td>15</td>
<td>80</td>
<td>5</td>
<td>0.3</td>
</tr>
<tr>
<td>20</td>
<td>75</td>
<td>5</td>
<td>0.3</td>
</tr>
<tr>
<td>30</td>
<td>65</td>
<td>5</td>
<td>0.3</td>
</tr>
<tr>
<td>50</td>
<td>45</td>
<td>5</td>
<td>0.3</td>
</tr>
</tbody>
</table>

Table 3: Mix proportions for Coventry Binder-ROSA-OPC paste mixtures (Group 3)

<table>
<thead>
<tr>
<th>Coventry Binder [%]</th>
<th>ROSA [%]</th>
<th>OPC [%]</th>
<th>W/B</th>
</tr>
</thead>
<tbody>
<tr>
<td>80</td>
<td>10</td>
<td>10</td>
<td>0.3</td>
</tr>
<tr>
<td>70</td>
<td>20</td>
<td>10</td>
<td>0.3</td>
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<tr>
<td>60</td>
<td>30</td>
<td>10</td>
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<tr>
<td>50</td>
<td>40</td>
<td>10</td>
<td>0.3</td>
</tr>
<tr>
<td>40</td>
<td>50</td>
<td>10</td>
<td>0.3</td>
</tr>
</tbody>
</table>
The mix in group 5 was made with 100 percent ordinary Portland cement (OPC) and 30 percent of water. This group was used as control mix to be compared with strength samples in groups 1 to 4. Mortar and concrete mixes were also made to investigate the performance and binding properties of Coventry binder with aggregates (Tables 5 and 6).

Table 4: Mix proportions for BOS-OPC paste mixtures (Group 4)

<table>
<thead>
<tr>
<th>BOS [%]</th>
<th>OPC [%]</th>
<th>W/B</th>
</tr>
</thead>
<tbody>
<tr>
<td>90</td>
<td>10</td>
<td>0.3</td>
</tr>
<tr>
<td>80</td>
<td>20</td>
<td>0.3</td>
</tr>
<tr>
<td>60</td>
<td>40</td>
<td>0.3</td>
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<td>40</td>
<td>60</td>
<td>0.3</td>
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<tr>
<td>20</td>
<td>80</td>
<td>0.3</td>
</tr>
</tbody>
</table>

2.3. Experimental Method
The mixing for pastes and mortar was carried out in a mechanical mixer having a 2 litre capacity. Mixing and casting procedure was as follows:

- Dry mixing of PG, BOS, BPD, ROSA and OPC (depending on which ingredients are applicable for each mix) for 1 minute.
- Half of the mixing water was added during next one minute of mixing.
- Mixing was continued for next minute at medium speed.
- The mixer was stopped and mixture was scraped off the sides of the bowl.
- The rest of the mixing water was added and mixing was carried out for a further one minute at medium speed.
- The mixture was poured in two layers in 50mm cube moulds.
- Each layer was fully compacted using a vibrating table.

Mortar and concrete mixes were also prepared using the similar procedure with only difference that aggregates were mixed with one third of required water prior to adding the dry mixed binder. 50 mm and 100 mm moulds were used for casting mortar and concrete samples respectively. All samples were demoulded after 24 hours and stored in containers in constant temperature 20±2 °C and 98 % RH humidity. The compressive strength of paste samples was measured at 3, 7, 28 and 90 days and compressive strength of mortar and concrete samples was measured at
3, 7 and 28 days. The Lloyd computerised crushing machine was used for compressive strength test of paste and mortar samples.

3. RESULTS AND DISCUSSION

3.1. OPC-Coventry Binder Paste Mixes (Group 1)
The results of compressive strength (Figure 1) showed that an increase in substitution of OPC with Coventry Binder had no significant beneficial effect on early and long term compressive strength. However, the mix incorporating 5 percent and 10 percent Coventry Binder achieved a higher compressive strength than paste mix made with 100 percent OPC at 28 days. This therefore indicates that the mix with 5% Coventry Binder and 95% Ordinary Portland cement is the optimum mixture in this combination. Pozzolanic reaction of part of slag present in Coventry Binder with calcium hydroxide of cement may be a reason for higher strength of the mix containing 5% Coventry Binder. In other mixes Coventry Binders appeared to act as filler and therefore the strength decreased due to less cement used in the mix.

![Figure 1. Compressive strength development of Coventry Binder-OPC paste mixes (W/B ratio 0.3)](image)

3.2. PG-BOS-OPC Paste Mixes (Group 2)
Figure 2 shows the compressive strength development of paste mixes made with 5% OPC and various BOS/PG contents. It was observed that in the ternary system increasing the Plasterboard Gypsum content in the mix resulted in decrease in long term compressive strength of pastes in this group. However, substitutions of PG with BOS led to higher strength gain. It can be also observed that all mixes achieved 90-day compressive strength of about 5.0 MPa. It was found that the compressive strength of PG-BOS-OPC mixes were the lowest compared to other mixes studied.
3.3. Coventry Binder-Rosa-OPC Paste Mixes (Group 3)

The strength development of paste mixes using various proportions of Coventry Binder and ROSA with constant 10% OPC and water to binder ratio 0.3 are shown in Figure 3. Results indicate that the mix containing 40% Coventry Binder and 50% ROSA achieved the highest strength at 7 and 28 days. Paste mixes containing 40, 50 and 60% Coventry Binder with subsequent amount of ROSA showed similar compressive strength 34.5 MPa at 90 days. This indicates that up to 60% of the binder can be replaced with novel Coventry Binder without significant effect on long term compressive strength of the mix.
Further investigation is needed to evaluate the strength of ternary mixes containing the lesser percentage of Coventry Binder with greater amount of ROSA.

3.4. BOS-OPC Paste Mixes (Group 4)
It was found that replacing of Ordinary Portland Cement with Basic Oxygen Slag does not have a beneficial effect on early and long term compressive strength (Figure 4). However, the mix incorporating 40 percent BOS achieved the highest compressive strength at 3 days and mix containing 20 percent BOS achieved the highest long term strength at 28 and 90 days.

Therefore it can be observed that the paste mix with 20% BOS and 80% OPC is the optimum mixture in this combination.
3.5. Coventry Binder-OPC Concrete Mix
Figure 5 shows the strength development of concrete mix containing 210 kg/m³ Coventry Binder and 90 kg/m³ OPC. It also contains coarse and fine aggregate. It was found that the long term strength at 28 days is equal 8.9 MPa. The result showed that it the strength was lower that the lowest class of concrete C12/15 with 20 MPa cube strength (Figure 5) [10]. This indicates that although the paste mixes containing Coventry binder achieved comparable strength with OPC samples, the weak binding of the novel binder with aggregates resulted in low compressive strength ion concrete samples.

3.6. Coventry Binder-OPC Mortar Mix
Mortar mix samples contained 266 kg/m³ Coventry Binder and 114 kg/m³ ordinary Portland cement. It was observed that after 3 and 7 days compressive strength was low, however after 28 days age compressive strength was 3.8 MPa (Figure 6). This showed that the long term strength for mortar containing Coventry Binder was comparable to standard class mortar M2.5 and M5 [11].

4. CONCLUSIONS
Base on results of this research the following conclusions can be drawn:
- Replacing ordinary Portland cement with Coventry Binder does not have a beneficial effect on early and long term compressive strengths.
- Paste mixes containing 5% and 10% of Coventry Binder in binary system Coventry Binder-OPC gained higher strength at 28 days than ordinary Portland cement paste mixes. Paste mixes containing 20% of Coventry Binder achieved nearly similar compressive strength at 28 days compared to mixes made with 100% OPC.
- In the ternary system PG-BOS-OPC, increasing the amount of Plasterboard Gypsum content in the paste mixes results in a reduction in long term compressive strength. 60% of Coventry Binder can be used as a cement
Replacing ordinary Portland cement with Basic Oxygen Slag does not have a beneficial effect on early and long term compressive strengths. The paste mixes in binary system BOS-OPC containing 20% BOS gained slightly lower compressive strength at 28 days than the OPC samples.

Paste mixes containing 20% Coventry Binder have a higher compressive strength at 3 days compared to paste mixes containing 20% of Basic Oxygen Slag. Paste mixes containing 40% Coventry Binder showed lower compressive strength than mixes made with 40% BOS at 3 and 7 days; however, the 28-day strength was similar.

Replacement of high level cement with Coventry Binder resulted in considerably lower strength than ordinary concrete mixes. This is due to relatively weaker binding of the novel binder to coarse aggregates particularly at early ages. However, the long term strength of mortar mixes containing Coventry Binder was comparable to standard mortar class M2.5 and M5.

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COMPERING THE EFECT OF USING THE COPPER BLAST FURNACE SLAG AND TAFTAN POZZOLAN ON CONCRETE PROPERTIES

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ABSTRACT
Today large number of researches are carried out on concrete properties due to it's widely use as an important construction material. The most part of concrete is cement which its cost decrease leads to producing of economical concrete. Using the industrial sweepings such as copper blast furnace slag which have pozzolan properties can make the cement economical. By using these materials not only production costs of cement will be reduced but also saving costs of sweepings will be eliminated and natural environment will be protected. In this research copper blast furnace slag milled to mesh325 which in this mesh maximum diameter of grains are 45 micron. 455 cubic specimens with size of 15×15×15 and cylindrical specimens with size of 30×15 have been made. Compressive and tensile experiments carried out and the results showed the positive effect of Taftan pozzolan and copper slag on concrete properties.

Keywords: copper blast furnace slag, compressive and tensile strength of concrete

1. INTRODUCTION
Concrete is a construction material composed of cement as well as sand and aggregate. Today the usage of pozzolans either natural or artificial has been increased. One of these artificial pozzolans is the slag of metal melt furnace such as iron, copper, etc. several researches on iron slag have been carried out in and out of Iran. Although acceptable studies have been conducted by Prof. Barzin Mobasher at Arizona State University in America; there are no earlier ones in Iran. Slag is a byproduct of metal smelting which float to the top during the smelting process because of its low density. It includes the compounds presenting in ores as well as the materials adding for lowering down the melting point of gangue. Copper ores usually include acid gangue which mainly have silica. These are the industrial waste materials which are removed from melting tank. Up to 300 thousand tons slags are produced each year during the production of copper. Slag was prepared from two kinds of copper furnace: Reverb -from Sarcheshmeh copper Complex- and flash furnace. There is about 1% copper in the slag of converter furnace which transferred to reverb one to obtain. Then it is exposed to the weather and cooled down after exiting the furnace. But in the Flash furnace at the copper factory of
Khatun abad in Rafsanjan, the slag is cooled down by water after exiting. Given such a high cooling rate makes the slag not to be crystallized and results in amorphous solid.

Therefore the substituting this kind of slag in the constituent of cement instead of reverb copper slag or pozzolan works very well. In the experiment conducted by Shargh Kan Micronize in Birjand both the slags were distinguished to be completely amorphous and enduring against the mill. Because the subject was comparing the effect of using copper slag and Taftan pozzolan on concrete properties so the grains diameter had to be similar to pozzolan in size. Therefore the slag was milled and the grains diameter decreased less than 45 micron. Because of being amorphous and high hardness (6 to 7 Mohs) this was a slow process. Unexpectedly, reverb kind was milled easier.

2. CHEMICAL ANALYSIS OF REVERB SLAG

As the slag will be a constituent of cement, its elements and components should be examined. Therefore chemical analysis was performed by Khash Cement Factory. The following table 1 demonstrates the chemical analysis of reverb slag:

<table>
<thead>
<tr>
<th>SiO₂</th>
<th>CaO</th>
<th>Al₂O₃</th>
<th>Fe₂O₃</th>
<th>SO₃</th>
<th>MgO</th>
<th>Cl</th>
<th>K₂O</th>
<th>Na₂O</th>
</tr>
</thead>
<tbody>
<tr>
<td>35.8</td>
<td>6</td>
<td>8.1</td>
<td>46.84</td>
<td>0.72</td>
<td>0.3</td>
<td>0.09</td>
<td>1.44</td>
<td>0.7</td>
</tr>
</tbody>
</table>

Activate module can be calculated by the following formula.

\[
\frac{\text{CaO} + \text{MgO} + \text{Al}_2\text{O}_3}{\text{SiO}_2} \geq 1
\]

\[
(0.3 + 6 + 8.1)/35.8 = 0.4 \leq 1
\]

Based on this formula the activation module is less than one and it is expected that it does not have appropriate properties. On the other hand according to ASTM C 618-92A the summation of this three oxides (Fe₂O₃, CaO, SiO₂) exceeds the percentile requirement of pozzolanic activity. This number compares various pozzolans for their degree of reactivity as compared to class F flyash. In this research for the study of compressive and tensile strength samples with different gravity percentages (5, 10, 15, 20, 25, 30) and ages (7, 28 days, 3 months, 6 months, and 1 year) were made and studied within 3 months. Study on 6 months and 1 year samples continues.
3. INITIAL AND FINAL SETTING TIMES OF THE CEMENT PASTE
In the present study, the cement of Qaen (type two), the slag of Khatoon abad and Sarcheshme Copper Complex and the pozzolan of Taftan were used. The time of the cement paste experiment only carried out on 20% of the pozzolan and the copper slag replacement of cement.

In this experiment the following results using ASTM C150-200 were obtained: (here the Vicat method has been used) (a cubic specimen 5×5×5 in dimension for compressive strength and a cylindrical specimen 30×15 in dimension for tensile strength). Based on this table, the flash copper slag and the pozzolan behave the same way and have the same final and initial paste, in comparison with the other samples, the reverb copper slag has a more initial set but its final set is closer to the cement one. In higher temperature of Sistan and Baluchestan using of this cement compound is advised because it reduces the volume variety to a minimum and will prevent the likely crack due to the volume changes.

4. STUDY OF THE SAMPLES’ COMPRESSIVE STRENGTH WITH THE CONTROL SPECIMEN
Analysis and study of the samples have a considerable importance and should be investigated. Samples with different gravity percentages (5 to 30%) used as cement replacement were made. Study on 6 months and 1 year ones continue. Control Specimen got 68% and 86% of the three months strength after 7 and 28 days respectively. At the end of three month rate of development became 16.32.

Analysis of the Flash Samples:
The 5% flash sample considered to be the most samples because its three months strength is 58% superior to the three months sample. And 28 days sample is approximately equal to the control specimen.

The 28 days strength of the 10% flash sample is about 37% superior to the control specimen and the three months sample exceeds in early strength (400) by as much as 428 which is the appropriate percentage.

The strength of the 15% flash sample is about 431 which corresponds to the three months control specimen and it is superior to the specific strength. The 28 days sample has the 99% of the control specimen strength. And the rate of compressive strength development during 3 months has increased in comparison to the 28 days sample. Therefore this sample has an appropriate compressive strength. The 3 months sample of the 20% flash has the 96% of the control specimen and compressive strength is superior to the specific one. Note that 28% of the control specimen strength took place during the 28 days period. The 20% flash sample is also appropriate one.

The compressive strength of the 25% flash sample is not appropriate because it has the 73% and the 76% of the control specimen strength during 28 day and 3 month period respectively. This sample has 83% of the specific strength during 3 months which is a very small amount.

The 30% flash sample has a better function than the 55% because it has 88% and 86% of the control specimen strength during 28 day and 3 month period respectively. Note that 69% of the control specimen strength took place during the 7 day period. The rate of development for 3 month sample is about 14%.

4.1. Analysis of Reverb Sample’s Compressive Strength

In this study two dosages of 5% and 15% samples considered to be ideal. At the 28th and 90th day, 5% sample exceeds in strength by as much as 4% and 7% respectively over the control specimen. The strength of 15% sample during the 28 day and 3 month period is respectively 2% and 1% superior to the control sample.
Rate of development in 3 month period for 5% sample is about 19.36, reflecting a high rate of development in comparison to the other samples during the 3 month period. The strength of the 3 months sample is superior to the control sample as much as 16%. The 20% reverb sample has 83% and 81% of the control specimen strength in 28 day and 3 month period respectively which is not an appropriate sample. But at the 3rd month, 25% reverb sample exceeds in strength by as much as 10% over the control sample and it has 94% of the control sample strength. At the end of the 3rd month, rate of development for the 15% and 20% samples is 14% but this number for the 25% dosage is about 20%.

Because the 30% sample has 75% and 78% of the control sample strength at 28 day and 3 month period respectively, it is not considered to be an ideal sample. Its rate rate of development is 21%.

4.2. Analysis of Pozzolan’s Compressive Strength
The best sample for pozzolans was 5% one but it had 92% of strength during the 3 month period. Rate of development for this sample was 14 to 33% while the 15% sample had a high rate of development. The reverb and flash copper slags behave well than the pozzolans. The 5% flash samples at ages of 7, 28, and 30 days were superior to the 5% pozzolan sample as much as 3-10%. In comparison to the pozzolan sample the compressive strength of
the 10% flash sample was superior as much as 11-19%.

The 15% flash sample exceeds in strength as much as 4.5-26% over the 15% pozzolan sample.

The strength of the 20% flash sample is superior to the pozzolan sample as much as 20%.

The 25% flash sample at age of 7 days was superior to the pozzolan one at age of 7 days over 5% but at the ages of 28, 30 days it was lower than the similar pozzolan sample as much as 3-5%.

The 5-25% reverb samples at the ages of 7, 28, and 30 days were superior to the 5-25% pozzolan samples as much as 4-34%.
4.3. The study of tensile VS. Bending Strength

For calculating tensile and flexural strength, cylindrical specimens (15×30 in dimension) were made and the Brazilian method was adopted for calculating the tensile strength but the coefficient made in this method was different from the experimental coefficient.

In experimental method, strength can be calculated from the following formula:

\[(N/mm^2)\]

(cf= compressive strength of cylindrical specimen)

Experiment on the cylindrical specimens for calculating tensile strength showed this results:

The coefficient for samples at ages of 7, 28 and 90 days became 1057, 1.5 and 1.375 respectively (Kg/Cm²).

For calculating flexural strength the following experimental formula is used:

Which in comparison to the tensile strength formula is 18% superior. According to the calculated coefficient, the flexural strength is higher than the tensile one.

The tensile strength showed increase with the addition of copper slag. The positive rate of development for three dosages of 5,10 and 20% of reverb samples stand in contrast with the 15, 25 and 30% samples i.e. the increase in ages of samples will decrease the tensile strength.

For the 10-25% flash samples, the tensile strength is positive as compared to the 5 and 30% samples reflecting negative one.
The increase in percentage and ages of pozzolan translates into an increase in the rate of development. For example, three dosages of 10, 15, and 15% pozzolan samples have a rate of development between 4-22% but 5 and 15% sample have a negative rate of development.

5. CONCLUSION
The studies carried out on the copper slag resulted in an improvement in the concrete properties such as tensile strength as well as compressive one. The flash samples had a high tensile and compressive strength than the reverb ones. However, the reverb samples were better than pozzolan ones. Therefore, the flash samples (for 5% slag) considered to be optimum and ideal.

The 10 -20% flash copper slag had a compressive strength as much as the specific one; so the following benefits can be derived from the usage of them in concrete:
1. Using the potential of the artificial pozzolans in development projects
2. Lowering the cost of the concrete production
3. Optimizing the concrete’s quality
4. Increasing the concrete efficiency and the quality of concrete productions
5. Increasing the age of concrete constructions against erosion factors
6. Eliminating the materials added in the concrete compound resulted in lowering the cost of the concrete
7. Making a change in development
8. Eliminating slag in copper smelting operation
9. Lowering the use of energy in the concrete production
10. Protecting the environment from the copper wastes
11. Using in the concrete constructions such as damming, silos, water reservoirs, etc.

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بررسی آزمایشگاهی تأثیر پوزولان خاش بر افزایش مقاومت بتن در سنین بالا

کچیده

در همه آخر، بنن غلتكی (Roller Compacted Concrete) به عنوان یک مصالح جدیدی با به عبارت پهپار روش اجرا می‌شود. دست‌اندرکاران ساخت و موسسات تحقیقاتی و دانشگاهی می‌باشند.

امروزه استفاده از مواد پوزولانی به عنوان مصالحی دارای خاصیت سیمانی و جایگزین شونده در قسمتی از سیمان پرتن مخلوط‌های بتنی مورد توجه زیادی می‌باشد. علت این امر به‌وجود آمدن اثرات خاص بتن و قیمت کمتر پوزولان در پوزولان سدازی می‌باشد.

در این مقاله با توجه به اینکه پوزولان طبیعی تلقین نزدیک‌ترین مجموعه به محل اجرای پوزولان سد زیردان و کارخانه سیمان خاش می‌باشد تاثیر این در افزایش مقاومت بتن مترکم غلتكی (RCC) در سنین بالا مورد بررسی قرار گرفته است.

نتایج این تحقیق تاکیدی بر عملکرد پوزولان خاش در افزایش مقاومت از 28 روز به 90 روز به این موضوع زمین کارکرده که به‌وجود آمده با تلاقی تکان‌دهی‌های سنین مقاومت مشخصه از مقدار سیمان کاسته و به انتظار مقاومت در مدت پوزولان بنشینیم.

کلیدواژه‌ها: پوزولان، بنن غلتكی، پوزولان خاش، سد بنن غلتكی

1- مقدمه

پوزولان ماده‌ای است که در مجاورت با آب اهم خواص سیمانی از خود نشان می‌دهد. پوزولان بر اساس استاندارد ASTM-C618 چنین تعریف می‌شود: "پوزولان ماده‌ای است سیلیسی با سیلیسی آلومیناتی که به خودی خود ازش چسبندگی ندارد، اما به شکل ذرات بسیار در مجاورت غلیظ‌بودن در درجه حرارت معمولی با هیدروکسید کلسیم واکنش شیمیایی دسته و ترکیباتی را به هدف مورد کلسیم سیمانی و چسبندگی می‌آورد." [7]. پوزولان ماده‌ای طبیعی با صفت عالی است که چاپ سیلیسی فعال می‌باشد. لازم است که ماده پوزولان به شکل پودر شده باشد، زیرا فقط دراین صورت سیلیسی می‌تواند در حضور آب با آهک (که به‌تمامی هیدراتاسیون سیمان پرتندازد) سیلیکات‌های کلسیم پایین را در خواص چسبندگی از طرف، تشکیل
دهد. همچنین سیلس ماده پوزولانی باید مکر و گردو دارای وکیس سپی سیلس فعال باشند.

استاد [8].

سیمان پوزولانی سیمانی است که از مخلوط سیمان و پوزولان در کارخانه تهیه می‌شود. اغلب مواد سیمان پوزولانی سیمانی است که جایگزین می‌شود آزمایشاتی که در تهیه روند افزایش حرارت کم نفته است. در بین حالت‌های امر اهمیت زیادی دارد و به هنین دلیل در این نوع بین‌ها بالاتر پوزولانی با یک‌بیانه‌ی سحابیزی پوزولانی و مصرف می‌شود. سیمان‌های پوزولانی در حجم حمله سولفات‌ها و به‌خصوص از عوامل مخرب مقاومت خویش از خود نشان می‌دهد. این امر به دلیل واکنش پوزولانی این که مقادیر کمتری از همکه‌یهای تا بهخارج راه یافته و زیو شنویدنی سطح را که راه‌های میدهند. لیکن مقاومت در بریار بخ‌زدن و آب‌سوزدن با سنین بالاتر که واکنش عمدی پوزولان تخلخل خوب سیمان را کاهش داده است [نیم‌نوادا: چابهاری 1387].

عوامل پوزولان در جلوگیری از انسانس مقر قلبی متفاوت از هوا نگذارید. پوزولان با تکامل زیایی که نرخ‌ات و گرد وردهای مختلفی را افزایش داده و نیز باعث افزایش درجه‌یی ساختمانی مترک می‌شود. این نوکسنه است که می‌باشد میانگین این مواد معمولاً در مخلوط بین می‌شود، نسبت انسانس مقر قلبی‌ها که در واکنش به نسبت زود همگام سیمان و پوزولان است که می‌باشد میانگین این مواد سیمان سیمان باقی و سر سیمان باقی نمی‌باشد. این مصلاحی این مستند که باعث افزایش مقاومت در باکس پوزولان می‌شود.[9].

اگرچه پوزولان خاص به عوامل یک پوزولان طبیعی در سال‌های اخیر مورد بررسی‌های تحقیقاتی و آزمایشگاهی قرار گرفته، اما این کشورهای مورد استفاده سنتی می‌دانند. سند و زیردان تقیبی اولین اصل روز ماده این پوزولان هستند. از اینکه میانگین سیالی سیالی و پوزولان به‌کمک اولین محصول کنونی است. این پوزولان از نوع RCC است که در روزهای سال‌های اخیر مورد استفاده قرار گرفته است. تاکنون که می‌باشد ماده که این پوزولان واقد بوده انتظار باید به دلیل حصار در بریار سد زیردان پوزولان خاص در حال استفاده است و با کنار چهار مت مکب در این جهت با استفاده از ان این اجماع می‌شود. در این مقاله سویه شده است که به نقش افزایش مقاومت در ایستاده شود در دی‌دروstad از پوزولان.

۲- روش تحقیق

- سد زیردان

به منظور تأمین آب شرب شهرهای چابهار و کنارد و آب موردیاز برای توسعه کشاورزی دشت‌های پیر‌سره‌های در.
بررسی ارزیابی‌کاری تاثیر پوزولان خاوش بر افزایش مقاومت بتن

کمینه و لاش از طریق مهار آب‌های نسبتی و همچنین بهره برداری از آب‌های زیرزمینی دست‌های سه‌گانه فوق الگر، نشان داد که برای شرکت سه‌گانه آب منطقه ای سیستان و بلوچستان در مهاره سال ۶۲ فشارهای مقاومت‌پذیری و همکاری مهندسین مشاور پوزولان - آمیپ - کاراکه معتقد گردید. پس از مطالعات اولیه طرح زیردان به عنوان یکی از طرح‌هایی به صورت دو منظوره تامین آب شرب و کشاورزی در اولویت قرار گرفت. معافیت از اواخر سال ۱۳۸۰ اجرای سن مخزی زیردان که از نوع بتنی و اکناری RCC است به پیمان‌کار و اکناری گردید.

سیمان

سیمان مورد استفاده در بتن غلکی و سایر اجزای بندی سد و سازه‌های وابسته، از نوع سیمان پرتلند پوزولانی نوع IP می‌باشد. مشخصات این سیمان مطابق استاندارد سیمان‌های آمیخته (ASTM C595) نوع IP و میزان پوزولان QP قدر خواهد بود با برآزش منحنی لگاریتی، معادله مقاومت – زمان را برای این نوع سیمان بسته اوریم (شکل ۱) بر اساس این معادله، مقاومت ۷، ۲۸ و ۱۸۰ روزه و به دنبال آن ضرایب رشد مقاومت قابل محاسبه است (جدول ۱ و ۲).

![شکل ۱ - برآزش منحنی لگاریتی بر تابع ۷، ۲۸ و ۳۸ روزه سیمان IP](image)

جدول ۱: مقاومت‌های محاسبه‌شده با برآزش منحنی لگاریتی بر تابع (۷، ۲۸ و ۳۸ روزه سیمان IP)

<table>
<thead>
<tr>
<th>سن (روز)</th>
<th>۷</th>
<th>۲۸</th>
<th>۹۰</th>
<th>۱۸۰</th>
</tr>
</thead>
<tbody>
<tr>
<td>(kg/cm²)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>مقاومت فشاری</td>
<td>188</td>
<td>261</td>
<td>323</td>
<td>360</td>
</tr>
</tbody>
</table>

جدول ۲: ضرایب رشد محاسبه‌شده با برآزش منحنی لگاریتی بر تابع (۷، ۲۸ و ۳۸ روزه سیمان IP)

<table>
<thead>
<tr>
<th>روز آزمایش</th>
<th>۷</th>
<th>۲۸</th>
<th>۹۰/۸۰</th>
<th>۱۸۰/۹۰</th>
</tr>
</thead>
<tbody>
<tr>
<td>کم شده</td>
<td>۱.۳۹</td>
<td>۱.۷۲</td>
<td>۱.۹۲</td>
<td>۱.۲۴</td>
</tr>
</tbody>
</table>

\[ f(t) = 53.057\ln(t) + 84.503 \]

\[ R^2 = 0.946 \]
بیوزولان

سطح‌های بیوزولانی در اکثر نقاط ایران باقی می‌مانند و در حاضر می‌توان به دیانیت‌های آذرشهر، پامیس سیلان، توف‌های انتخابی تراس جارود، زلولیت میانه، توف انتخابی بیشتری در بندر بندران و تخته نخستین سه ناحیه اصلی مíasه در محل اجرای پروژه کرد و کارخانه سیمان خاس می‌باشد در ادامه مشخصات این بیوزولان در جدول ۲ به تفصیل بیشتری ارائه شده است.

### جدول ۲: مشخصات بیوزولان از آزمایش‌های استاندارد ASTM C618

| ترکیبات شیمیایی | بیوزولان سیمان | پروژه
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>SiO2</td>
<td>۵۱-۵۸</td>
<td></td>
</tr>
<tr>
<td>Al2O3</td>
<td>۱۸-۱۹</td>
<td></td>
</tr>
<tr>
<td>Fe2O3</td>
<td>۰-۵</td>
<td></td>
</tr>
<tr>
<td>CaO</td>
<td>۰-۷</td>
<td></td>
</tr>
<tr>
<td>MgO</td>
<td>۰-۵</td>
<td></td>
</tr>
<tr>
<td>Na2O</td>
<td>۰-۷</td>
<td></td>
</tr>
<tr>
<td>K2O</td>
<td>۰-۵</td>
<td></td>
</tr>
<tr>
<td>L.O.I</td>
<td>۰-۵</td>
<td></td>
</tr>
<tr>
<td>حداکثر ۲۰ درصد</td>
<td>SiO2+Al2O3+Fe2O3</td>
<td></td>
</tr>
</tbody>
</table>

- تست‌های آزمایشگاهی

حالاتی از آزمایش‌های فیزیکی، در واقع بخش اول آزمایش‌ها، در اندازه‌گیری استفاده شده در این آزمایش‌ها همان مصالح محلی بیوزولان بوده و نیز از سیمان تیپ دو بیوزولان و بوزوولان بوده سیمان استفاده شده است. در این مطالعات برای هر طرح بین غلتکی سد بیوزولان تعداد ۶ نمونه استفاده یافته‌ای ۱۵۳۰۰ تهیه شده است که پس از گذشت ۲۸ و ۹۰ روز نکته‌های در شرایط استاندارد آزمایشگاهی به منظور تعیین مقاومت فشاری زیر جک، از روش‌دان شرکت قرار. میانگین مقاومت هر دو نمونه بعنوان تیه‌میود بررسی قرار گرفت. جدول ۳ مواد مشابه طرح‌های اختلاف و نتایج آزمایش را نشان می‌دهد، در این طرح‌ها از سیمان بیوزولان خاک (حاوی ۱۵% بیوزولان) با افزودن مقداری گرد بیوزولان استفاده شده است.

### مقاومت فشاری ملات سیمان

برای آگاهی از نرخ تأثیر بیوزولان خاک بر افزایش مقاومت، یک سری آزمایش مقاومت فشاری ملات برای سیمان دوچرخه ۲۰،۳۰ و ۴۰ درصد بیوزولان مطابق ASTM C349 انجام گرفت، تعدادی از نمونه‌های ساخته شده و همچنین روش آزمایش در شکل ۲ ارائه شده است، همچنین نتایج آزمایش وجود شده در شکل ۲ ارائه شده است.
جدول ۴: مواد مشکل و نتایج ارزیابی‌های طرح اختجاط بین افتکاری

<table>
<thead>
<tr>
<th>شماره طرح</th>
<th>سیمان پوزولان</th>
<th>وزن مخصوص (kg/m3)</th>
<th>مقاومت فشاری (kg/cm2)</th>
<th>زمان ۷ روزه</th>
<th>وزن ویبی (ناتیو)</th>
<th>مقاومت فشاری (kg/cm2)</th>
<th>زمان ۹۰ روزه</th>
</tr>
</thead>
<tbody>
<tr>
<td>۱</td>
<td></td>
<td>۲۳۵۰</td>
<td>۲۵.۳</td>
<td>۲۲.۰</td>
<td>۲۰</td>
<td>۱۲۴</td>
<td>۱۴۵</td>
</tr>
<tr>
<td>۲</td>
<td></td>
<td>۲۲۰۰</td>
<td>۳۰۵</td>
<td>۳۴.۳</td>
<td>۲۰</td>
<td>۱۴۰</td>
<td>۱۴۵</td>
</tr>
<tr>
<td>۳</td>
<td></td>
<td>۲۲۰۰</td>
<td>۱۴۳.۵</td>
<td>۲۴.۰</td>
<td>۲۰</td>
<td>۱۵۰</td>
<td>۱۵۵</td>
</tr>
<tr>
<td>۴</td>
<td></td>
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<td>۲۲.۰</td>
<td>۱۹۸</td>
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<td>۱۴۰</td>
<td>۱۴۰</td>
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<td>۱۴۰</td>
<td>۱۴۰</td>
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<td>۲۰</td>
<td>۱۴۰</td>
<td>۱۴۰</td>
</tr>
</tbody>
</table>

جدول ۵: ضرایب رشد طرح‌های اختجاط بین افتکاری

<table>
<thead>
<tr>
<th>شماره طرح</th>
<th>نسبت پوزولان به شماره</th>
<th>ضریب ضریب</th>
<th>کسب شده ۹۰ روزه/۲۸ روزه</th>
<th>کسب شده ۹۰/۲۸</th>
</tr>
</thead>
<tbody>
<tr>
<td>۱</td>
<td>۳۹.۲</td>
<td>۰.۱۷</td>
<td>۰.۱۹</td>
<td>۱.۰۰</td>
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<td>۲</td>
<td>۳۹.۲</td>
<td>۰.۲۳</td>
<td>۰.۶۲</td>
<td>۲.۲۵</td>
</tr>
<tr>
<td>۳</td>
<td>۳۹.۵</td>
<td>۰.۳۶</td>
<td>۰.۷۱</td>
<td>۳.۸۵</td>
</tr>
<tr>
<td>۴</td>
<td>۲۷.۸</td>
<td>۰.۶۲</td>
<td>۱.۱۱</td>
<td>۲.۸۰</td>
</tr>
<tr>
<td>۵</td>
<td>۲۷.۵</td>
<td>۰.۶۶</td>
<td>۰.۸۲</td>
<td>۲.۱۳</td>
</tr>
<tr>
<td>۶</td>
<td>۲۷.۵</td>
<td>۰.۳۷</td>
<td>۰.۶۷</td>
<td>۲.۴۹</td>
</tr>
<tr>
<td>۷</td>
<td>۳۹.۲</td>
<td>۰.۴۰</td>
<td>۰.۶۹</td>
<td>۲.۴۲</td>
</tr>
</tbody>
</table>

نمونه‌های منشوری برای تعیین مقاومت فشاری ملات سیمان

شکل ۲- نمونه‌های منشوری برای تعیین مقاومت فشاری ملات سیمان

شکل ۳- نتایج مقاومت فشاری نمونه‌های ملات سیمان حاوی مقادیر مختلف پوزولان
در شکل ۴ محاسبه شده با پوزولان ۱۳۸۸ و ۹۰۰ روزه

یک توجه به جدول ۵ ضرایب رشد مقاومت به شرح جدول ۶ بدست آورده می‌شود. میانگین خلو ضرایب رشد در سطهرایین جدول ۶ محاسبه شده است.

جدول ۵: ضرایب رشد محاسبه شده با پوزولان ۱۳۸۸ و ۹۰۰ روزه

<table>
<thead>
<tr>
<th>ضریب</th>
<th>۷</th>
<th>۲۸</th>
<th>۹۰</th>
<th>۱۸۰</th>
</tr>
</thead>
<tbody>
<tr>
<td>f(20%)P</td>
<td>۲۹۰.۰۳</td>
<td>۳۹۶.۰۳</td>
<td>۴۸۲.۰۳</td>
<td>۵۳۲.۰۳</td>
</tr>
<tr>
<td>f(30%)P</td>
<td>۲۷۰.۰۳</td>
<td>۳۸۰.۰۳</td>
<td>۴۷۲.۰۳</td>
<td>۵۲۷.۰۳</td>
</tr>
<tr>
<td>f(40%)P</td>
<td>۲۴۱.۰۳</td>
<td>۳۳۷.۰۳</td>
<td>۴۱۹.۰۳</td>
<td>۴۶۷.۰۳</td>
</tr>
<tr>
<td>f(50%)P</td>
<td>۲۲۰.۰۳</td>
<td>۳۰۳.۰۳</td>
<td>۳۷۳.۰۳</td>
<td>۴۱۵.۰۳</td>
</tr>
</tbody>
</table>

جدول ۶: ضرایب رشد محاسبه شده با پوزولان ۱۳۸۸ و ۹۰۰ روزه

<table>
<thead>
<tr>
<th>ضریب</th>
<th>۲۸/۷</th>
<th>۹۰/۷</th>
<th>۱۸۰/۷</th>
<th>۹۰/۲۸</th>
<th>۱۸۰/۹۰</th>
</tr>
</thead>
<tbody>
<tr>
<td>میزان پوزولان (٪)</td>
<td></td>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>۲۰</td>
<td>۱.۳۴۴</td>
<td>۱.۶۳۳</td>
<td>۱.۸۰۵</td>
<td>۱.۲۱۵</td>
<td>۱.۱۰۵</td>
</tr>
<tr>
<td>۳۰</td>
<td>۱.۴۰۵</td>
<td>۱.۷۴۷</td>
<td>۱.۹۵۰</td>
<td>۱.۲۴۳</td>
<td>۱.۱۱۶</td>
</tr>
<tr>
<td>۴۰</td>
<td>۱.۴۰۲</td>
<td>۱.۷۴۰</td>
<td>۱.۹۴۱</td>
<td>۱.۲۴۱</td>
<td>۱.۱۱۵</td>
</tr>
<tr>
<td>۵۰</td>
<td>۱.۳۷۷</td>
<td>۱.۶۹۵</td>
<td>۱.۸۸۴</td>
<td>۱.۲۳۱</td>
<td>۱.۱۱۱</td>
</tr>
</tbody>
</table>

اگر این ضرایب رشد را بر روی نمودار مانند شکل ۵، شامل محاسبه‌های درجه سه با ضریب رگرسیون ۱ پیدا شوند، مشاهده می‌شود که بیشترین ضرایب رشد در نسبت وزنی ۳۳.۸ درصد پوزولان حاصل می‌شود. بنی ۴۵۰۰

Compressive Strength vs. Time; Mortar Test

\[ f(20\%)P = 73.10 \ln(t) + 152.76 \]
\[ f(30\%)P = 79.01 \ln(t) + 116.40 \]
\[ f(40\%)P = 69.73 \ln(t) + 104.93 \]
\[ f(50\%)P = 59.94 \ln(t) + 103.63 \]
فرض داشتن مخلوط‌های با مقاومت ۷ روزه یکسان، مخلوط حاوی ۳۳.۸ درصد پوزولان بیشترین افزایش مقاومت را در سینه بالاتر خواهد داشت. بازی مقدار پوزولان پیشنهاد شرایط رشد ماکزیمم ۲۸ به ۷، ۹۰ به ۷ و ۱۸۰ به ۷ روزه به ترتیب عبارت خواهند بود از ۱.۷۵، ۱.۴۱ و ۱.۹۶.

ازمایش‌های مقدماتی طرح‌های اختلاف بین غلکی در این مرحله برای هر مخلوط علاوه بر نمونه‌های ۷ و ۲۸ روزه، نمونه‌های ۹۰ روزه نیز ساخته شد تا بر اساس آنها روند مقدار مقاومت دانسته شود. در شکل ۶ مقاومت‌های ۹۰ روزه در مقابل مقاومت‌های ۲۸ روزه به تفکیک برای مخلوط‌های حاوی ۲۵ و ۳۵ درصد پوزولان - رسم شده اند. همان‌گونه که می‌بینید منحنی‌های رشد مقاومت از هم‌سنتی خوبی پرخوردن. 

شکل ۵ - تعیین درصد پوزولان بهینه با پرداخت منحنی‌های درجه ۳ بر ضرایب رشد

شکل ۶ - رابطه بین مقاومت ۹۰ روزه مخلوط‌های بین غلکی با مقاومت فشاری ۲۸ روزه آنها

باینری بودن خط مربوط به پوزولان ۳۵ درصد نیست به پوزولان ۲۵ درصد در شکل ۶ به معنای آنست که برای دو مخلوط با مقاومت ۲۸ روزه یکسان، پوزولان ۳۵ درصد باعث افزایش مقاومت بیشتری در سن ۹۰ روزه نیست.
به پوزولان ۲۵ درصد خواهد شد: میان مقدار پوزولان فعل، عدید بیش از ۲۵ درصد می‌باشد. شکل ۶ روند کسب مقاومت ۲۸ تا ۹۰ روزه را به‌صورت تابعی از مقاومت ۲۸ روزه شانه می‌دهد.

- ازمانی‌اکی، طرح‌های اختلاف بین غلتكی در این مرحله مجموعاً ۱۶ طرح اختلاف بین غلتكی کار شده است. نتایج ۲۸، ۲۹ و ۳۰ روزه همه طرح‌ها در جدول ۷ آراگ ترکیب‌های است. مقایسه مجدده شده در سطح دو جدول ۷ جدول ۲، مقاومت ضرب زند ۲۵ تا ۳۰ درصد پوزولان برای ۱۱۰. است. در شکل ۷ ملی‌های آب به سیمان و مقاومت ضرب‌ال.czv ۲۸، ۲۹ و ۳۰ روزه به طرح‌های حاوی و ۲۰ درصد پوزولان رسم شده است. هم‌طور که دیده می‌شود در سنین بالاتر، ضرب در گرسنین شده و ۲۵ درصدی پوزولان در زمان مورد و جهان ضعف‌های اختلافی موجود در ریز ساختر بین غلتكی دارد.

جدول ۷ نتایج طرح‌های بین غلتكی و ضرایب رشد مربوطه

<table>
<thead>
<tr>
<th>طرح</th>
<th>نام طرح</th>
<th>وزن</th>
<th>زمان سیمان</th>
<th>مقاومت ضرب (kg/cm2)</th>
</tr>
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<tr>
<td></td>
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<td>۲۸</td>
<td>۹۰</td>
</tr>
<tr>
<td>R170-20-02</td>
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<td>۱۱۱.۸</td>
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<tr>
<td>R190-20-01</td>
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<tr>
<td>R190-20-02</td>
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<td>۲۸۶.۳</td>
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<tr>
<td>R150-20-01</td>
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<td>۲۰</td>
<td>۹۰.۷</td>
<td>۱۲۴.۵</td>
</tr>
<tr>
<td>R150-20-02</td>
<td>۰.۷۶</td>
<td>۲۶</td>
<td>۹۳.۱</td>
<td>۱۲۵.۱</td>
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<tr>
<td>R150-20-03</td>
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<tr>
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<td>۱۱۰.۶</td>
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<tr>
<td>R210-38-02</td>
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<td>۲۵</td>
<td>۱۱۰.۶</td>
<td>۲۰۸.۳</td>
</tr>
</tbody>
</table>

- بحث

این محقق فرض کردند بر عملکرد پوزولان خش در آزمایش مقاومت از ۲۸ روز به ۹۰ روز بوده و این موضوع زمانی کاری کرد که با هم‌اکتیون سیمان مقاومت مشخصه، از مقدار سیمان کاسته و به انتظار مقاومت روز پوزولان رقیم می‌گردد در سطح مقاومت شارژی و هرینه تولید یک‌داترا باید از محلی موضوع دارد که افزودن پوزولانشان به سیمان انجر به اقدام صد طرح و اثرات زیست محیطی در ارتباط با روش‌های تولید سیمان باید.[۱۸].

این بحث مخاطرات از آن‌ست که افزایش سیمان مقاومت مشخصه و آزمایش دوباره چرا که در صورت آزمایش سیسان مقاومت مشخصه در سنین فیزیکی به سیمان و این میان دگر جایی برای حس و گمان باید تخویه بقد.
همچنین، با تغییر مقادیر مشخصه پنتهای RCC قادر خواهیم بود مقدار سیمان مصرفی در هر متر مکعب را کاهش داده و از تیانسیل افزایش مقدار پوزولان در زمان دیده می‌شوید. در مقایسه با آن بدون پوزولان، می‌توانیم در شکل‌های 7-28، 90 و 180 غلکسی، نشان دهنده داشته نیز نشان دهنده رشد مقاومتی مناسبی در سطح مختلف جز در سطح 3 و 7 روزه از خود نشان داده است. (17) یکی از مفاهیم درحقیقت به عمل آمده با توجه بالا شده بود.

یکی از مهم‌ترین خصوصیات موادی که در بین پوزولان و افزایش مقاومت در زمان آن است. کاهش حرارت هیدراسیون می‌تواند احتمال پیدایش ترکیبات حضوری را کاهش دهد.

1. یکی از خرابی‌های عمده در سده‌ها و اکثر قبایل سنتگانه‌های بایزی و ایکت اگزکس که بر به کمکه درد. درصد شیب‌های بایزی با توجه به امکانات کارگاه و ملایم‌ها فیلی تغییر. بهینه‌تر به یکی از شرایط آب و هوایی مناطق جنوب ایران و رود ایرانی زیاد کارگاهی استفاده از شیب‌های بایز، باعث بالا شدن شیب‌های مورد استفاده (16) که در آزمایشات انجام گرفته نیز به‌همان و رشد خصوصیات بین غلکسی از جمله
تشکر و قدردانی

بیدوییلینه از مهندس محسن جعفریگلی و تکنیسیان‌های آزمایشگاه بین شرکت جهان کوت (پیمانکار سد زیردان) که در کلیه مرحله‌های مراحل ساخت و انجام آزمایش‌ها، عملکرد هماهنگ، فراوانی و میدویل داشته‌اند

سیاست‌گرایی و قدردانی می‌شود.

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CD02
Concrete Structures
Analysis and Design
PUSHOVER ANALYSIS OF ASYMMETRIC ORDINARY MOMENT R.C FRAMES DESIGNED ACCORDING TO THE IRANIAN CODES

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ABSTRACT
In this paper, the nonlinear seismic behavior of three ordinary moment-resisting concrete space frames with unsymmetrical plan in three, four and five stories are evaluated using pushover analysis. The three buildings were originally designed according to existing Iranian codes. Seismic loads are calculated and distributed over the height of the frame using both rectangular and triangular forms. It has been found that the obtained capacity curves have been affected greatly by the forms of loading. Results have been also produced in form of story drifts to establish the performance level of these buildings. The results show that all of the frames in both directions are within the life safety performance level.

Keywords: pushover, concrete frame, Seismic assessment, irregularity, design codes

1. INTRODUCTION
Experience shows that buildings with irregularities are prone to earthquake damage, as observed in many earthquakes in the past. Despite structural regularity is quite easy to obtain through a careful design; it is very common that, in the reality, different irregularities can occur, changing the seismic performance of the building. However, current codes fail to provide acceptable definition of an irregular structure. Moreover, most of the seismic codes fall short of providing sufficient specifications for designing irregular buildings. As an example, the ASCE/SEI 7-05 standard \([1]\) defines five types of horizontal structural irregularities. One type of irregularity is reentrant corner irregularity, and this irregularity is considered to exist where both plan projections of the structure beyond a reentrant corner are greater than 15% of the plan dimension of the structure in the given direction. In the ASCE/SEI 7-05 standard \([1]\), additional provisions are given to increase the design forces for connections of diaphragms to vertical elements and to collectors and for connections of collectors to the vertical elements. Although such provisions are not adequate to take care of all the stresses concentrated at different points of the buildings, the Iranian Code of Practice \([2]\) has not provided similar provisions. Furthermore, the Iranian Code of Practice for Seismic Resistant \([2]\) has given more relaxed regulations for such irregularity by specification projections of more than 25% of the plan dimension in the given direction. Accordingly, there is an apparent need to develop a more accurate
analysis procedure to provide information on the seismic response of irregular structures. In this paper, an introductory investigation on the effect of such irregularities on the total behavior of the buildings is carried out.

In this paper, and as shown in Figure 1, reinforced concrete frames having plan projections between 33 to 50% are examined. These buildings have been designed according to the current Iranian codes of practice [2, 3]. However, and to examine the validity of these practices and evaluate the real strength of these structures, a performance-based design needs to be carried out. To meet this requirement, static pushover analysis is used to investigate the effect of such irregularities on the total behavior of the buildings.

The recent advents of performance-based design show that an inelastic procedure commonly referred to as the pushover analysis is a viable method to assess damage vulnerability of buildings [4, 5]. This procedure is a static, nonlinear one in which the magnitude of the structural loading is incrementally increased in accordance with certain predefined patterns. With the increase in the magnitude of the loading, weak links and failure modes of the structure are found. The loading is monotonic with the effects of the cyclic behavior and load reversals being estimated by using a modified monotonic force-deformation criteria and with damping approximations. The present pushover analysis has been carried out using the "SAP2000" software [6]. The method used by this software is based on procedure C given in ATC-40 [7].

![Figure 1. Plan view of the building structure (a) a 3-story building (b) 4 and 5-story buildings](image)

In most studies, the method was applied to symmetrical structures. Assuming the floors act as rigid diaphragms, the state of damage of the building can be inferred from applying a two-dimensional pushover analysis on the building. The advantages and the limitations of this analysis for damage assessment are described by Lawson et al. [8]. Usually, the presence of an asymmetry in a given structure makes the pushover analysis rather complicated, since floor displacements of the building will consist of both translational and rotational components. The lateral load resisting elements located at different positions in plan will experience different deformations. Torsional effect can be particularly damaging to elements located at or near the flexible edge of the building where the translational and
rotational components of the floor displacement are additive.

In the last few years several proposals have been put forward to extend traditional pushover analysis to the assessment of three-dimensional models. Among the early ones is that of Moghadam and Tso [9, 10]. It has been based on the study of the non-linear static behavior of the critical frames only, identified by means of LDP analyses performed on three-dimensional models. Other attempts to extend and verify 3D pushover algorithms can be found in references [11-13]. Furthermore, Fajfar et al. [14-16] extended the N2 method to three-dimensional structures. On the other hand, Chopra and Goel [17] have presented an extension of the MPA (Modal Pushover Analysis) procedure for asymmetric-plan structures.

2. SAMPLE STRUCTURES

In this paper, the nonlinear seismic behavior of three ordinary moment-resisting concrete space frames with unsymmetrical plan in three, four and five stories are evaluated. The plan configurations of these space frames contain reentrant corners. To compare the nonlinear response of structures differently involved in the inelastic range of behavior, each building was designed according to the rules proposed by the Iranian Code of Practice [2], for low ductility structures. However in all these cases, the masses of the floors are less than 5% from the corresponding centers of rigidity of the floors in both perpendicular directions.

Using the Iranian Code of Practice for Seismic Resistant [2], the design was performed with reference to the importance category II, assuming peak ground acceleration equal to 0.25 g and parameters shaping a soil profile II spectrum. According to the code provisions the strength level of the structures was defined by assuming an R-factor equal to 5.00 for the low ductility (LD) buildings, and consistently with the supposed importance category, an importance factor equal to 1.0 was assumed.

The floors were considered to be subjected to dead loads equal to 620 kg/m² (due to self weight, finishes and permanent partitions) and to live loads equal to 200 kg/m². Moreover, claddings weighting 500 kg/m were considered to be present along the external perimeter of buildings and parapets weighting 200 kg/m were considered to be present along the external perimeter of the roof. Design lateral forces are given in Table-1.

<table>
<thead>
<tr>
<th>Buildings</th>
<th>Loads (ton)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3-story</td>
<td>52.1</td>
</tr>
<tr>
<td>4-story</td>
<td>51.7</td>
</tr>
<tr>
<td>5-story</td>
<td>65.1</td>
</tr>
</tbody>
</table>

The three gravity (vertical) loads used in this paper are as follows:

\[
\text{GR}_1 = 1.1 (Q_D + Q_L) \quad (1)
\]

\[
\text{GR}_2 = 0.9 Q_D \quad (2)
\]
In Equations (1-3), \( Q_d \) is the total dead loads and \( Q_l \) is the total live loads.

For lateral seismic loads, the analysis was performed by assuming two types of lateral loads distributions. First by assuming a triangular distributions similar to that obtained by the equivalent static analysis method, and second by assuming rectangular distributions proportional to the weight of the floor. Combining these loads with the three vertical loads defined in Equations 1 to 3, buildings were tested under the effect of twenty four different combinations. These are as follows:

(a) \( P_X1 \) and \(-P_X1\) triangular distributions of lateral forces + \( G_R1 \)
(b) \( P_X2 \) and \(-P_X2\) triangular distributions of lateral forces + \( G_R2 \)
(c) \( P_X3 \) and \(-P_X3\) triangular distributions of lateral forces + \( G_R3 \)
(d) \( P_Y1 \) and \(-P_Y1\) triangular distributions of lateral forces + \( G_R1 \)
(e) \( P_Y2 \) and \(-P_Y2\) triangular distributions of lateral forces + \( G_R2 \)
(f) \( P_Y3 \) and \(-P_Y3\) triangular distributions of lateral forces + \( G_R3 \)
(g) \( F_X1 \) and \(-F_X1\) rectangular distributions of lateral forces + \( G_R1 \)
(h) \( F_X2 \) and \(-F_X2\) rectangular distributions of lateral forces + \( G_R2 \)
(i) \( F_X3 \) and \(-F_X3\) rectangular distributions of lateral forces + \( G_R3 \)
(j) \( F_Y1 \) and \(-F_Y1\) rectangular distributions of lateral forces + \( G_R1 \)
(k) \( F_Y2 \) and \(-F_Y2\) rectangular distributions of lateral forces + \( G_R2 \)
(l) \( F_Y3 \) and \(-F_Y3\) rectangular distributions of lateral forces + \( G_R3 \)

Each of the Frames considered has uniform storey height of 3m. The strength of the beams and columns in the frame are allocated following the "strong column-weak beam" capacity design procedure. All members of the frame have been designed according to the Iranian Concrete Code of Practice [3]. All columns have square cross-section of 300*300 mm. at the upper two stories and 350*350 mm. at the third storey and 400*400 mm for the first and second stories. The required different strength levels in columns were also obtained by varying the amount of reinforcement. All the beams have rectangular cross-section of 200*300 mm. Table 2 contains detail of column and beam cross-sections and reinforcements. All members were detailed considering a normal weight concrete with compressive strength of 250 kg/cm² and steel having yield strength equal to 4000 kg/cm².

<table>
<thead>
<tr>
<th>Columns and beams cross-sections</th>
<th>3-storey</th>
<th>4-storey</th>
<th>5-storey</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Buildings</strong></td>
<td>35 cm</td>
<td>40 cm</td>
<td>40 cm</td>
</tr>
<tr>
<td><strong>First Floor</strong></td>
<td>8 cm</td>
<td>12 cm</td>
<td>16 cm</td>
</tr>
<tr>
<td><strong>Second Floor</strong></td>
<td>30 cm</td>
<td>35 cm</td>
<td>40 cm</td>
</tr>
<tr>
<td><strong>Third Floor</strong></td>
<td>30 cm</td>
<td>35 cm</td>
<td>30 cm</td>
</tr>
<tr>
<td><strong>Fourth Floor</strong></td>
<td>30 cm</td>
<td>30 cm</td>
<td>30 cm</td>
</tr>
<tr>
<td><strong>Fifth Floor</strong></td>
<td>30 cm</td>
<td>30 cm</td>
<td>20 cm</td>
</tr>
<tr>
<td><strong>Beams (dimensions in cm.)</strong></td>
<td>20 cm</td>
<td>20 cm</td>
<td>20 cm</td>
</tr>
<tr>
<td><strong>All</strong></td>
<td>30 cm</td>
<td>30 cm</td>
<td>30 cm</td>
</tr>
</tbody>
</table>
3. RESULTS

3.1. Global Yield Criteria

Since the yield point is not clear in the plot of base shear versus top displacement, an idealized elasto-plastic system was assumed to find the approximated yield point in the global response of the structure. Yield displacement is based on the idealized elasto-plastic system with reduced stiffness which is evaluated as the secant stiffness at 75% of the ultimate strength.

A sample of the displacements corresponding to the yield points for different vertical and lateral loads for the 5-story is given in Table 3. Same calculations are repeated for pushover curves in the negative direction and the results are similar to those given in Table 3.

Table 3: Displacements (cm.) corresponding to the yield points for different loads

<table>
<thead>
<tr>
<th>Loads</th>
<th>Fx</th>
<th>Fy</th>
<th>Px</th>
<th>Py</th>
</tr>
</thead>
<tbody>
<tr>
<td>GR1</td>
<td>17</td>
<td>17</td>
<td>18</td>
<td>18</td>
</tr>
<tr>
<td>GR2</td>
<td>17</td>
<td>17</td>
<td>18</td>
<td>18</td>
</tr>
<tr>
<td>GR3</td>
<td>17</td>
<td>17</td>
<td>18</td>
<td>18</td>
</tr>
</tbody>
</table>

3.2. Performance Curves

To obtain the capacity curve, seismic loads are calculated and distributed over the height of the frame using both rectangular and triangular forms. Some examples of the resulting capacity curves for the three buildings are shown in Figure (2). All curves show similar features. They are linear initially but start to deviate from linearity when inelastic actions start to take place. With the increase of displacements, the capacity curves become linear, but with much smaller slopes that sometimes approaching flat shapes. Furthermore, it can be concluded that the curves obtained for all the three gravity loads are approximately similar to each other while they are more sensitive to the type of lateral loads, as shown in Figure (3).
Figure 2. The performance curves for (a) 3-story building (b) 4-story building (c) 5-story building

Figure 3. The performance curves for 5-story building for GR3
3.3. The Performance Point

The performance point for a given set of values is defined by the intersection of the capacity curve and the single demand spectrum curve. Results for the 5-Story buildings are given in Tables 4 and 5.

<table>
<thead>
<tr>
<th>Lateral Loads</th>
<th>Displacements at the Performance Point (cm)</th>
<th>Forces at the Performance Point (ton)</th>
<th>Lateral Loads</th>
<th>Displacements at the Performance Point (cm)</th>
<th>Forces at the Performance Point (ton)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PX1</td>
<td>19.73</td>
<td>69.29</td>
<td>-PX1</td>
<td>-19.87</td>
<td>-69.27</td>
</tr>
<tr>
<td>PX2</td>
<td>19.34</td>
<td>68.23</td>
<td>-PX2</td>
<td>-19.46</td>
<td>-68.26</td>
</tr>
<tr>
<td>PX3</td>
<td>19.53</td>
<td>68.84</td>
<td>-PX3</td>
<td>-19.67</td>
<td>-68.80</td>
</tr>
<tr>
<td>FX1</td>
<td>17.73</td>
<td>84.57</td>
<td>-FX1</td>
<td>-17.87</td>
<td>-84.54</td>
</tr>
<tr>
<td>FX2</td>
<td>17.32</td>
<td>83.78</td>
<td>-FX2</td>
<td>-17.44</td>
<td>-83.83</td>
</tr>
<tr>
<td>FX3</td>
<td>17.33</td>
<td>83.50</td>
<td>-FX3</td>
<td>-17.67</td>
<td>-84.20</td>
</tr>
</tbody>
</table>

Table 5: Performance points for the 5-story buildings for a fixed gravity load and different lateral loads in the (y) and (-y) directions

<table>
<thead>
<tr>
<th>Lateral Loads</th>
<th>Displacements at the Performance Point (cm)</th>
<th>Forces at the Performance Point (ton)</th>
<th>Lateral Loads</th>
<th>Displacements at the Performance Point (cm)</th>
<th>Forces at the Performance Point (ton)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PY1</td>
<td>20.97</td>
<td>66.98</td>
<td>-PY1</td>
<td>-21.03</td>
<td>-66.98</td>
</tr>
<tr>
<td>PY2</td>
<td>20.58</td>
<td>66.89</td>
<td>-PY2</td>
<td>-20.62</td>
<td>-66.13</td>
</tr>
<tr>
<td>PY3</td>
<td>20.77</td>
<td>67.00</td>
<td>-PY3</td>
<td>-20.83</td>
<td>-66.97</td>
</tr>
<tr>
<td>FY1</td>
<td>18.77</td>
<td>81.23</td>
<td>-FY1</td>
<td>-18.83</td>
<td>-81.24</td>
</tr>
<tr>
<td>FY2</td>
<td>18.37</td>
<td>81.27</td>
<td>-FY2</td>
<td>-18.37</td>
<td>-81.27</td>
</tr>
<tr>
<td>FY3</td>
<td>18.57</td>
<td>81.25</td>
<td>-FY3</td>
<td>-18.63</td>
<td>-81.25</td>
</tr>
</tbody>
</table>

3.4. Maximum Displacement

Considering the maximum roof displacement of the buildings, the results obtained denote that all of the frames in both directions are within the life safety performance level.

3.5. Inter-Story Drift

On the structure level, the inter-story drift ratio (ID) is one of the simplest and most commonly used damage indicators. Similar comparisons are carried out on the prediction of the maximum inter-storey drift ratios. Samples of the results are presented in Figure (4). In this figure, inter-storey drift ratios are compared to the limit values subscribed by FEMA 356 [18] for the life safety and immediate occupancy performance levels.
Figure 4. Performance of 5-story RC frame based on maximum inter-storey drifts ratios X-direction (b) Y-direction

3.5. Plastic Hinge Formation

The damage state of the structure at the peak base shear for the 3-story building is given in Figure 5. For more details on the formation of plastic hinges at different performance level is given in Reference [19].

Figure 5. Plastic hinges in the 3-story building under PX₁
4. SUMMARY AND CONCLUSIONS
Since current codes fall short of providing simplified analytical tools for irregular structures, it is necessary to use an analytical procedure that can describe the seismic response of such buildings. The present paper utilizes the pushover analysis, a procedure based on "the capacity curve" concept, to investigate irregular buildings. The results obtained show that:
(a) Performance curves obtained for all the three gravity loads, for a given lateral load, are approximately the same while they show more sensitivity to the type of lateral loads.
(b) Irregularities similar to that investigated in the present paper have little influence on the total behavior of the building.
(c) The results obtained denote that all of the frames in both directions are within the life safety performance level. It can be concluded that using Iranian codes to design irregular reinforced concrete frames of three to five stories is acceptable for providing life safety performance level.
(d) Comparing the formation of plastic hinges in the three buildings, the 5-story building has shown better performance than the other two.
However, given the preliminary nature of this study, additional work considering different 3D buildings need to be carried out before any definitive conclusions and recommendations might be made.

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DESIGN OF MASONRY INFILLED REINFORCED CONCRETE FRAMES IN DIFFERENT SEISMIC CODES

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²Expert, Building and Housing Research Centre, Tehran, Iran

ABSTRACT
Masonry infilled reinforced concrete frames are among the most widely used types of buildings in Iran. In the past, masonry infill walls have often been treated as nonstructural elements in buildings, and their effects are not included in the analysis and design procedure. Furthermore, the interaction between infill and frame is usually ignored in the design procedure. Past experience has shown that infill walls have significant positive or negative effects on the global behavior of buildings and, therefore, should be addressed appropriately. This paper reviews and compares analysis and design provisions of this system in various seismic design codes and identifies the most important issues that are related to it. Stiffness, strength, natural period, response reduction factor, irregularities, and effect of openings are among the items discussed in this paper.

Keywords: infill, concrete frame, seismic design, design codes, masonry

1. INTRODUCTION
Unreinforced masonry (URM) infill panels are widely used throughout the world, including seismically active regions. They are usually used as interior partitions and external walls in concrete frames, but they are treated as nonstructural elements and not included in the analysis and design procedure. Such a simplified design approach does not predict the level at which the damage in the infill panel occurs, on the other hand it does not consider the global and local effects of having these stiff and brittle elements coupled with the primary lateral load-resisting system [1]. However, and contrary to common practice, field experience and experimental investigations [1-4] show that infill walls, if effectively confined by the frame, are remarkable in increasing the initial stiffness, strength and energy dissipation of RC (reinforced concrete) frames, especially if the structural system itself has little engineered earthquake resistance.

Typically, MI (masonry infill) walls are made of brittle materials that lose capacity in a rapid manner. Accordingly, the combined effect of brittleness and high stiffness has a negative implication on the seismic performance of the bounding frames. In particular, loss of integrity of the infills in the ground storey may produce a soft storey and trigger global collapse [5]. Furthermore, if infills are non-uniformly distributed in planes or in elevation, inelastic deformation demands will concentrate in the part of the building which has more sparse infills (i.e., to the
“flexible” side of a building asymmetrically infilled in plan, or to the “weak” or “soft” storey of the infilled frame) [5]. Generally, improper arrangement of infill walls causes a significant increase in the demand forces on the diaphragm and collector elements (adjacent beams and columns) that results in brittle shear failures, short column phenomena, and torsional response to the translational horizontal components of the seismic action. In such cases, both the frame and the floor system should be adequately designed for such increase in the demand forces. From the structural point of view, the structural response of infilled frames depends on numerous parameters. Overall geometry of infills, dimensions of concrete members, the variability of mechanical properties of infill and concrete members, reinforcement configurations, the relative frame to infill stiffness, location and dimension of openings, distribution of MI walls throughout the story and construction details are some of these important parameters. Although, a large amount of research related to infilled frame structures has been conducted, some uncertainties still remain. One important source of uncertainty is the type of interaction between the infill and the frame. The interaction between the frame and the infill panel sometimes changes the structural response significantly.

This paper reviews and compares analysis and design provisions related to infilled RC frames in seismic design codes. In designing RC frames, in general, infills can be grouped into two categories: isolated infills and shear infills. However, few seismic codes specify recommendations on isolated infills. When ductile RC frames are designed to withstand large displacements without collapse, masonry infills should be isolated from the confining frame by sufficient gaps at the top and on both sides. The isolation (gaps) between the infill and the frame must be greater than any possible deformation expected by the frame, thus prohibiting any infill/frame interaction. These infills are not considered as structural elements. In this manner, MI walls do not affect the frame performance and frame displacements are not restrained. Another advantage of the isolated MI is that the walls remain undamaged, thereby reducing post-earthquake repair costs. In the following sections, some of the important issues discussed in the seismic codes are reviewed.

2. NATURAL PERIOD

Natural periods of vibration of buildings depend upon their mass and lateral stiffness. Presence of non-isolated MI walls in buildings increases both the mass and stiffness of buildings. Consequently, the natural period of an MI-RC frame is normally lower than that of the corresponding bare frame.

All seismic codes rely heavily on empirical formulae for the natural period for estimating design seismic force. However, few codes specify formulae for MI-RC frames. The comparison of these formulae for different structural systems is given in Table 1. Beside empirical formulae, most seismic standards recommend the use of Rayleigh formula for natural period [6, 10], or other general dynamic methods. According to Crowley and Pinho [11], the use of uncracked section in the computation of elastic natural periods of RC structures is inadequate because it would lead to an underestimation of the displacement demands. Cracking of critical
elements such as beams generally occurs under gravity loading alone, and even in those cases where cracking is not found to have occurred before the design seismic level of excitation, it will occur early on in the response to excitation and thereafter the stiffness will reduce rapidly. As a result, many seismic codes like IS 2800 provides provisions for calculations of natural periods based on effective stiffnesses [6]. Others like NEHRP 2003 [9] and EC8 [10] have based their equations on the measured periods of buildings during earthquakes where at least a limited amount of cracking of the MI-RC frame occurred. On the other hand, the optional use of $T = 0.1N$, given by NEHRP 2003 [9] and many other codes, has been found inadequate for MI-RC frames [12]. More details about this subject can be found in Reference [13].

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel moment-resisting frames</td>
<td>0.08(H)$^{0.75}$</td>
<td>0.0853(hn)$^{0.75}$</td>
<td>0.0724(hn)$^{0.8}$</td>
<td>0.085(H)$^{0.75}$</td>
</tr>
<tr>
<td>RC moment-resisting frames</td>
<td>0.07(H)$^{0.75}$</td>
<td>0.0731(hn)$^{0.75}$</td>
<td>0.0466(hn)$^{0.9}$</td>
<td>0.075(H)$^{0.75}$</td>
</tr>
<tr>
<td>For structures with MI walls</td>
<td>Steel moment frames: 0.8*$0.08(H)^{0.75}$</td>
<td>0.0743(hn)$^{0.75}$/ $\sqrt{A_c}$ (see note no.1)</td>
<td>-</td>
<td>0.075(H)$^{0.75}$/ $\sqrt{A_c}$ (see note no.2)</td>
</tr>
<tr>
<td></td>
<td>Concrete moment frames: 0.8*$0.7(H)^{0.75}$</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

1- $A_c = \sum A_e [0.2 + (D_e/h_n)^2]$

$A_c$ is the combined effective area, in m$^2$, of the MI shear walls in the first story of the structure.

2- $A_c = \sum A_i (0.2 + (l_{wi}/h))^2$

$A_c$ is the total effective area, in m$^2$, of the MI shear walls in the first story of the structure.

$A_e$ is the minimum cross-sectional area in any horizontal plane in the first story of the building, in m$^2$. $D_e$ is the length, in m, of the wall $e$ in the first story in the direction parallel to the applied forces. $h_n$ is the cross-sectional depth in m above the base to Level n. $D_e/h_n$ should not exceed 0.9.

$L_{wi}$ is the length, in m, of the wall $i$ in the first story in the considered direction, and $h$ is the cross-sectional depth in m. $l_{wi}/h$ should not exceed 0.9.
3. RESPONSE REDUCTION FACTOR

The response reduced factor (R) is an empirical factor intended to account for damping, overstrength, and the ductility inherent in the structural system at displacements great enough to surpass initial yield and approach the ultimate load displacement of the structural system [9]. The (R) values, contained in most seismic codes are largely based on engineering judgment of the performance of the various materials and systems in past earthquakes [9]. Furthermore, it is difficult to compare (R) values given in different codes since they use different design philosophies and safety and load factors. Therefore, (R) values need to be compared for different building systems within a particular code only. (R) value for MI-RC frames is generally less than that for bare frames, thus most codes require MI-RC frames to be designed for higher force levels than the corresponding bare frames (about 1.15 to 3.0 times). Comparison of the response reduction factors for different structural systems is given in Table 2.

<table>
<thead>
<tr>
<th>Lateral Resisting System</th>
<th>Allowable Stress</th>
<th>Ultimate Strength</th>
<th>EC8 [10]</th>
</tr>
</thead>
<tbody>
<tr>
<td>CSMF¹</td>
<td>10</td>
<td>12</td>
<td>8.5</td>
</tr>
<tr>
<td>CIMF²</td>
<td>7</td>
<td>8</td>
<td>5.5</td>
</tr>
<tr>
<td>COMF³</td>
<td>4</td>
<td>5</td>
<td>3.5</td>
</tr>
<tr>
<td>CSMF¹ + MI Walls</td>
<td>10</td>
<td>8</td>
<td>5.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CIMF² + MI Walls</td>
<td>7²</td>
<td>7</td>
<td>4.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>COMF³ + MI Walls</td>
<td>-</td>
<td>6</td>
<td>4.2</td>
</tr>
</tbody>
</table>

1. CSMF = Concrete Special Moment Resisting Frame
2. CIMF = Concrete Intermediate Moment Resisting Frame
3. COMF = Concrete Ordinary Moment Resisting Frame
4. SMW = Special Masonry Shear Wall
5. IMW = Intermediate Masonry Shear Wall
6. OMW = Ordinary Masonry Shear Wall
7. This reduction factor is for buildings without infill. For infilled frames, natural period is calculated according to table 1
8. This is for RC frames with MI in contact with the frame

4. LATERAL LOAD SHARING BETWEEN INFILL AND FRAME

The RC frame and MI walls must resist the prescribed lateral seismic force in accordance with their relative rigidities considering fully the interaction of the
walls and the RC frames as a single system. This analysis must be made in accordance with the principles of structural mechanics considering the relative rigidities of the elements and torsion in the system. Deformations imposed upon members of the RC frame by their interaction with the MI walls must be considered in this analysis [9]. According to most codes, the frame alone is required to be designed to independently resist full vertical loads and at least 25% of the design seismic forces [6, 9, and 13]. MI walls, which are normally very stiff initially, attract most of the lateral forces, but may fail prematurely because of the brittle behavior. In such cases, RC frames must have sufficient backup strength to avoid the collapse of the structure. Accordingly, EC8 [10] puts more strict regulations by requiring that RC frames need to resist at least 50-65% of the total lateral loads in addition to the full vertical loads.

5. PLAN IRREGULARITIES
A building may have a symmetrical geometric shape without re-entrant corners or wings but still be classified as irregular in plan because of distribution of mass (i.e., asymmetric placement of MI walls) or vertical, seismic-force-resisting elements [9]. According to EC8 [10], slight plan irregularities may be taken into account by doubling the accidental eccentricity. In case of severe plan irregularities, due to excessive unsymmetrical placement of MI walls, three-dimensional analysis is required considering stiffness distribution related to the uncertain position of MI walls.

6. VERTICAL IRREGULARITIES
Vertical irregularities are introduced into MI-RC frames due to reduction or absence of MI walls in a particular story compared to adjacent stories, e.g., buildings with parking space in the first story and MI walls on upper stories. In general, this gives rise to mass, stiffness, and strength irregularities along the height of buildings. Vertical irregularities in the bottom stories make the beams and columns of those stories more susceptible to damage or failure [9]. Open ground story buildings have consistently shown poor performance during past earthquakes across the world.

According to IS: 1893 [16], all the columns of the soft/weak storey should be designed for 2.5 times the seismic demand. On the other hand, EC8 [10] recommends an increase in the resistance of columns of soft stories by a factor $\eta$ that is given by:

$$\eta = 1 + \frac{\Delta V_{RW}}{\sum V_{ED}} \leq q \quad \text{(Units: } \Delta V_{RW}, \sum V_{ED} \text{ in N)}$$

(1)

where $q$ is the response reduction factor given in Table-2, $\Delta V_{RW}$ is the total reduction in lateral resistance of MI walls in a story compared to the story above, and $\sum V_{ED}$ is the sum of seismic shear forces acting on all structural vertical elements of the story concerned. The design forces are not required to be increased
if the factor $\eta$ is less than 1.1.

7. STRENGTH OF MASONRY INFILL
In designing infill panels, simple analytical tools that encompass the wide variety of possible failure mechanisms of infilled frames should be developed to assist in the design and performance evaluation of these structures. Although strength of MI walls does not have any direct implications on the ultimate strength of ductile RC frames; in some cases, failure modes of MI walls control the failure modes of non ductile RC frames. Many formulae had been developed in the past [17-19], however, these have only been reflected recently in seismic design codes. In cases where the infill component controls the stiffness, FEMA 306 [20] and NZSEE [21] specify four inplane modes of failures, namely, sliding shear failure, compression failure, diagonal tension failure of panel and general shear failure of panel. On the other hand, panel strength in FEMA 356 [22] is given by the shear sliding (bed-joint) strength only with no enhancement for axial stress.

8. STIFFNESS OF MASONRY INFILL
The stiffness of any structure generally affects both forces and displacements. For calculation of design seismic force, the use of a lower estimate of the stiffness leads to unconservative results. On the other hand, controlling the drift requirements under seismic loads, it is unconservative to make a higher estimate of stiffness. Hence, some standards have suggested the use of two different analytical models for buildings:

a. the model to be used for calculation of design seismic force should include all stiffness contributions, including those of nonstructural members.
b. the model to be used for drift calculation should include all possible contributions to flexibility and should not include stiffness contributions of members that cannot be relied upon to provide stiffness at large displacements, such as MI walls.

For example, to calculate forces in the structure, NEHRP 2003 [9] has suggested the use of the natural periods given in Table-1. However, to prevent the use of a flexible frame, an upper bound on the value of natural period that can be used to calculate the design force has been specified. On the other hand, most seismic codes including NEHRP 2003 [9] put lower bound on the overall seismic design force. For determining the story drift limits, NEHRP 2003 has permitted the use of computed natural periods without using the upper limit [9].

MI walls are laterally much stiffer than RC frames, and therefore, the initial stiffness of the MI-RC frames largely depends upon the stiffness of MI walls. Accordingly, it is quite important to have a reliable method to estimate the stiffness of the infill. For global building analysis purposes, the compression struts representing infill stiffness of solid infill panels may be placed concentrically across the diagonals of the frame, effectively forming a concentrically braced frame system. This model has been adopted by many seismic codes [10, 20-22] and is based on the work of Mainstone [23]. In this model, however, the forces imposed
on columns and beams of the frame by the infill are not represented. To account for these effects, compression struts may be placed eccentrically within the frames [21-22]. If the analytical models incorporate eccentrically located compression struts as shown in Figure (1), the results should yield infill effects on columns directly. Diagonally concentric equivalent struts may also be used to incorporate infill panel stiffnesses into analytical models for perforated infill panels (e.g., infills with window openings). Analysis of local effects, however, must consider various possible stress fields that can potentially develop within the infill. As an alternative to the approach described above, FEMA 356 [22] suggests the use of multiple compression struts, as have been proposed by Hamburger [24].

![Figure 1. Modeling the adverse effect of an infill panel on the performance of the perimeter frame showing (a) the placement of the strut, and (b) the moment pattern on the columns](image)

**9. EFFECT OF OPENINGS IN MASONRY INFILL ON STRENGTH**

Presence of openings in MI walls changes the actual behavior of RC frames because of reduction in lateral strength and stiffness. Such infills pose the hazard of out-of-plane collapse. Hence, it is best to avoid situations that lead to infill panels of large width or height [16]. Unfortunately, there is little information on the effects of openings on the strength and stiffness of MI-RC frames in seismic codes [13].

The effect of opening in the infill wall is to reduce the lateral stiffness and strength of the frame. This can be represented by a diagonal strut of reduced width. The reduction factor is defined as ratio of reduced strut width to strut-width corresponding to fully infilled frame. Using IS: 1893 [16], equation for the reduction factor $\rho_w$ is given as:

$$\rho_w = 1 - 2.5A_r, \rho_w \geq 0$$  \hspace{1cm} (2)

where, $A_r$ is the opening area ratio, which is the ratio of face area of opening to the face area of infill. On the other hand, NZSEE [21] specifies different reduction factor $\lambda_{\text{opening}}$ based on the width of opening measured across a horizontal plane $L_{\text{opening}}$ and given by Equation (3):

$$\lambda_{\text{opening}} = \frac{L_{\text{opening}}}{L_{\text{ref}}}$$  \hspace{1cm} (3)
$$\lambda_{\text{opening}} = 1 - \frac{1.5L_{\text{opening}}}{L_{\text{inf}}}, \lambda_{\text{opening}} \geq 0$$  \hspace{1cm} (3)

According to EC 8 [10], large openings are required to be framed with RC elements across the full length and thickness of walls. Vertical RC elements of at least 150 mm dimension are required at both sides of any opening larger than 1.5 m² area. Longitudinal steel in the element shall not be less than 300 mm² or 1% of the cross-sectional area of the element. Shear reinforcement in the form of stirrups of at least 5 mm diameter is required with a minimum spacing of 150 mm [10].

10. OUT-OF-PLANE STRENGTH OF MASONRY INFILLS

During earthquakes, MI walls are subjected to high in-plane shear forces because of their high initial stiffness. Tension cracks are formed along the loaded diagonal in MI walls, which causes reduction in their lateral strength. In addition, connection between the RC frame and MI wall is generally weak and MI wall may get separated from RC frames during the in-plane or out-of-plane ground motion, and thus become susceptible for collapse in the out-of-plane direction. However, such an out-of-plane collapse is not common for walls of low slenderness value and for well-confined masonry infill walls. From the above statements, it is clear that isolated infill walls are more susceptible to collapse than shear infill walls in the out-of-plane direction.

Different seismic codes require that nonbearing wall panels that are attached to or enclose the structure be designed to resist the inertial forces and to accommodate movements of the structure resulting from lateral forces [6, 9] or temperature change [9]. This is particularly important for systems composed of brittle materials or materials with low flexural strength [9]. Once masonry walls crack, continued shaking can easily cause collapse in the heavy infill blocks and pose a serious life safety threat to building inhabitants. Furthermore, panel support systems often lack redundancy and failure of a single connection can have catastrophic consequences. In recognition of this, different codes require fasteners to be designed for approximately 4 times the required panel force and that the connecting member be ductile [6, 9]. This is intended to ensure that the energy absorption takes place in the connecting member and not at the connection itself and that the more brittle fasteners remain essentially elastic under seismic loading [9].

The out of plane strength of MI walls has been given by many seismic codes [20-22]. On the other hand, EC8 [10] suggests several preventive measures to avoid brittle failure, premature disintegration, and out-of-plane failure of masonry infill walls during earthquakes, especially for slender walls (ratio of the smaller in length or height to thickness greater than 15). The measures includes providing light wire meshes adequately anchored on masonry infill walls and on RC frames, wall ties fixed to columns and cast into bedding planes of masonry, and concrete posts and belts across the panels and through the full thickness of the masonry infill. On the other hand, FEMA 356 [22] suggests that MI panels not in tight contact with perimeter frame members should be restrained for out-of-plane forces. This may be accomplished by installing steel angles or plates on each side of the infills, and
welding or bolting the angles or plates to the perimeter frame members.

11. LOCAL EFFECTS DUE TO MASONRY INFILLS

Presence of infills modifies and magnifies the shear demands on the frame members by shortening the distance between in-span plastic hinges (Figure 1). The shear demand will be a maximum when flexural plastic hinges form at each end of this so-called "short column". EC8 [10] requirements for local effects are as follows:

1) Because of the particular vulnerability of the infill walls of ground floors, a seismically induced irregularity is to be expected there and appropriate measures should be taken. If a more precise method is not used, the entire length of the columns of the ground floor should be considered as the critical length and confined accordingly.

2) If the height of the infills is smaller than the clear length of the adjacent columns, as shown in Figure (2), the following measures should be taken:
   a) The entire length of the columns (L_{eff}) is considered as critical region and should be reinforced with the amount and pattern of stirrups required for critical regions;
   b) The consequences of the decrease of the shear span ratio of those columns should be appropriately covered. In this calculation the clear length of the column L_{cl} should be taken equal to the length of the column not in contact with the infills.
   c) The transverse reinforcement to resist this shear force should be placed along the length of the column not in contact with the infills and extend along a length h_c (dimension of the column cross-section in the plane of the infill) into the column part in contact with the infills.
   d) If L_{eff}, the length of the column not in contact with the infills is less than 1.5 h_c, the shear force should be resisted by diagonal reinforcement.

3) Where the infills extend to the entire clear length of the adjacent columns, and there are masonry walls on only one side of the column (e.g. corner columns), the entire length of the column should be considered as a critical region and be reinforced with the amount and pattern of stirrups required for critical regions.

![Figure 2. The effect of partial infills on frame performance](image-url)
4) The length, of columns $L_{cl}$ over which the diagonal strut force of the infill is applied, should be verified in shear for the smaller of the following two shear forces:

a) The horizontal component of the strut force of the infill, assumed to be equal to the horizontal shear strength of the panel, as estimated on the basis of the shear strength of bed joints; or

b) The shear force computed in accordance with Equation (4), depending on the ductility class.

$$V = \gamma_{Rd} ((M_{Rd,c1} + M_{Rd,c2})/ L_{cl})$$

where $L_{cl}$ is the contact length ($L_{c1eff}$ or $L_{c1eff1}$), and $\gamma_{Rd}$ is an overstrength factor.

**SUMMARY AND CONCLUSIONS**

In the present paper, design provisions for MI-RC in different seismic codes are reviewed. Taking the current practices into consideration, these provisions provide a good base for design and construct masonry infill panels. However, major issues in various seismic codes need further attention. These issues can be summarized as follows:

**Natural Period:** Empirical estimation of natural period addresses very simple and regular MI-RC frames. Because of practical reasons, most RC buildings become irregular when masonry infill walls are added in RC frames. Therefore, most of the empirical equations may not estimate the natural periods of such buildings with sufficient accuracy.

**Weak and Soft Stories:** Design of weak/soft-story frame members is done in different seismic codes based on empirical or semi-empirical relations. Very limited literature is available in support of these relations. Hence there is an urgent need for more research in this area.

**Strength and stiffness of MI-RC frame:** In calculating the strength and the stiffness of MI-RC frames, many simplified assumptions are used. Neglecting the effect of nonstructural components and the presence of openings in masonry infill walls are some examples of such simplifications. The current ‘state-of-the-art’ method used to account for infill panels is to model an equivalent strut to represent the stiffness of the panels. It has been reported that this model give good results within the linear range. However, using these models beyond the mortar cracking or failure of the infill walls needs further studies. Furthermore, results from experimental and finite element investigations suggest a strong interaction between in-plane and out-of-plane capacities of the infill walls. Neglecting this interaction may lead to unconservative seismic risk evaluation. Accordingly, reflecting these issues in the new editions of seismic codes is of high priority.

**Response Reduction Factor:** There is no consensus in various seismic codes on values of response reduction factor, which reflects that more research is needed on reliable estimation of strength and ductility of such buildings.

**Irregularities:** Seismic codes address the problems associated with plan and vertical irregularities in MI-RC frames in different ways. However, in case of
severe irregularities in plan due to the unsymmetrical arrangement of the infills, spatial models need to be specified for the analysis of the structure, including, if necessary, a sensitivity analysis regarding the position and the stiffness of the infills.

**Local effect:** Local effects that occurred due to the frame-infill-interaction need to be taken into account. Efficient strengthening methods of nonductile columns need to be specified in seismic codes in order to avoid irreparable damage and catastrophic failure of the structure.

**REFERENCES**

MODELING OF NONLINEAR BEHAVIOR OF RC SHEAR WALLS UNDER COMBINED AXIAL, SHEAR AND FLEXURAL LOADING

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ABSTRACT
Predicting the behavior of RC shear walls under bending moment without existence of interaction of any other kind of loadings like shear or axial load is simple and can be conducted with good accuracy. But what is a great concern, is predicting their behavior under the interaction of shear, axial and flexural loadings. In this research, there is an effort to investigate the behavior of RC shear walls under this condition of loading with a novel approach. A general but simple macro model is proposed that can include flexural and shear behavior of the wall by considering the effects of pull put and slippage of reinforcing bars as well as concrete tension softening, stiffening and confinement. This simple model is applicable to different wall shapes with different reinforcement ratios and its prediction has good agreement with experimental results. The predicted behavior of the walls is compared with some available experimental results to show the accuracy of the proposed method.

Keywords: RC shear wall, behavior, nonlinear analysis, P-M-V interaction, modeling

1. INTRODUCTION
Reinforced concrete walls are very effective in resistance of lateral loads imposed by earthquakes. They provide high strength and stiffness and if truly designed, can also provide good ductility for structures. So many analytical and experimental researches have been carried out to study the seismic behavior of RC walls and RC frame-wall systems. The response of these elements is complex and their overall behavior is influenced by a combination of flexural, shear, and axial deformations. Prediction of the exact inelastic response of RC walls requires accurate analytical material models that consider the important characteristics and behavioral response features such as concrete tension-stiffening, nonlinear shear behavior and effects of loading condition, confinement, transverse reinforcement, and reinforcing bars slippage on strength, stiffness and deformation capacity.
Analytical modeling of the inelastic behavior of RC wall systems can be accomplished either by using microscopic finite element models or macroscopic models. Various analytical models have been proposed for predicting the inelastic
behavior response of RC walls through a microscopic or macroscopic approach. Although microscopic finite element models can provide a refined and detailed definition of the local response, their efficiency, practicality, and reliability are questionable due to complexities involved in developing the model and interpreting the results. Macroscopic models, on the other hand, are practical and efficient, although their application is restricted, based on the simplifying assumptions upon which the model is based[1].

A common macro modeling approach is using a beam-column element model. This model consists of an elastic flexural element with a nonlinear rotational spring at each end to account for the inelastic behavior of critical regions (Error! Reference source not found.). To model the RC walls more realistically, improvements, such as multiple spring representation [2], varying inelastic zones [3], and specific inelastic shear behavior [4] have been introduced into simple beam column models. However, inelastic response of structural walls subjected to horizontal loads is dominated by large tensile strains and fixed end rotation due to bond slip effects, associated with shifting of the neutral axis. This feature cannot directly be modeled by a beam-column model, which assumes that rotations occur around points on the centroidal axis of the wall and this method disregards the important features of the experimentally observed behavior, including the variation of the neutral axis of the wall cross section, rocking and reinforcing bars slippage [1].

Kabeyasawa et al. [5] proposed a new macroscopic three-vertical-line element model (TVLEM) to account for the features that beam-column model cannot capture. In this model, the wall was idealized as three vertical line elements with rigid beams at the top and bottom levels (Figure 1). In this model, shear stiffness degradation was incorporated, but was assumed to be independent of the axial load and bending moment.

This method is improved by different authors such as Vulcano et al. [6], [7], Kabeyasawa [8] and Colotti [9].
In this paper a novel macro model is proposed that can include flexural and shear behavior of the wall by considering the effects of reinforcing bars pull put and slippage, concrete tension softening and stiffening and confinement. This simple model is applicable to different wall shapes and reinforcement ratios and shows good agreement with experimental results.

2. ANALYTICAL MODEL

The adopted method for nonlinear analysis of RC walls in this paper can take into account the effects of flexural and shear behavior. Flexural behavior is computed by considering a macro fiber model for the wall with including the effects of confinement and reinforcing bars pull-out. Shear behavior is predicted according to a nonlinear analysis of RC elements under in-plane stresses through a fixed smeared crack analysis approach.

In this method, as is shown in Figure 3, the force-displacement curve of the flexural behavior, reinforcement pull out and shear behavior of the wall is computed separately and will be combined to obtain the total nonlinear behavior of the wall. The total behavior of the wall is computed by adding the displacements caused by each of the three behavioral modes that have been mentioned above for any shear value (Figure ). If one of the behavioral modes has lower strength than the others, it will be the controlling behavior (e.g. Shear behavior in Figure ).
3. FLEXURAL MODELING

3.1. Moment Curvature Analysis
Adopted method here for flexural analysis of RC wall resembles a macro fiber model. In this model, the wall is divided into a series of uniaxial elements (Error! Reference source not found.) and then by considering the appropriate material uniaxial nonlinear models, the moment-curvature analysis of the wall will be done with considering the effects of confinement and reinforcing bars pull out from the foundation. Each fiber in the model can have different material properties and steel ratio. Maekawa's [10] material models are used for modeling the uniaxial behavior of the concrete and reinforcing bars.

Knowing the applied axial force on the wall, the moment curvature analysis is done by assuming a linear strain distribution across the section and calculating the stresses in each fiber (Error! Reference source not found.) and controlling if Eq.(1) is satisfied. If this equation is satisfied, the moment and curvature in the section can be computed according to Eq.(2) and Eq.(3) in that step, and if it is not, then the assumed strain distribution should be corrected in an iterative procedure. This procedure is repeated in several steps until the failure of the steels or crushing of the concrete occurs.
\[ \sum \sigma_i A_i = N \] (1)

\[ \sum \sigma_i A_i y_i = M \] (2)

\[ \kappa = \frac{\varepsilon_l + \varepsilon_t}{l} \] (3)

where, \( \sigma_i \) is the stress in each fiber, \( A_i \) is the area of each fiber, \( N \) is the constant axial force applied to the wall, \( y_i \) is the fiber distance to the neutral axis of the section, \( \kappa \) is the curvature of the section, \( \varepsilon_l \) is the first layer strain, \( \varepsilon_t \) is the last layer strain and \( l \) is the length of the section.

Figure 6. Moment-curvature behavior of a RC wall

3.2. Shear Displacement Curve Due to Flexural Behavior

By using the Eq. (4) the shear corresponding to the moment of the wall can be computed in each step and the curvature is convertible to the wall base rotation, \( \theta \), by using the Eq. (5). The top displacement of the wall can also be calculated by using Eq. (6).

\[ V = \frac{M}{h} \] (4)

\[ \theta = \int_0^h \frac{\kappa x}{h} \, dx \] (5)

\[ \delta = \theta \frac{1}{h} \] (6)

where \( h \) is the wall height. So by computing the shear and the corresponding displacement in each step, the shear displacement curve of the wall due to flexural behavior is attained.
4. REINFORCEMENT PULL OUT

In reinforced concrete members, local discontinuities, such as pulling out of reinforcing bars from the thicker element and sinking the thinner element to the thicker one, tend to take place as a result of abrupt changes in the section stiffness at the joint planes connecting two components of different thickness [11]. This phenomenon has an important effect on the displacements of the wall that should be considered in the analytical methods to obtain good results in comparison to experimental results. Here, the Maekawas pull out model [11] is used to consider this important effect. This model describes a relation between steel strain and loaded end slip or relative displacement of steel bar to concrete and is applicable to both elastic and plastic stress states. This model is capable of giving a unique strain-slip relation for a bar that has a long embedded length and that has slip at the free-end prevented (Eq. (7)).

\[
\begin{align*}
    s &= \varepsilon_s (2 + 3500\varepsilon_y) \quad \text{for } \varepsilon < \varepsilon_y \\
    s &= s_y \quad \text{for } \varepsilon_y < \varepsilon < \varepsilon_{sh} \\
    s &= s_y + 0.047(f_u - f_y)(\varepsilon_s - \varepsilon_{sh})
\end{align*}
\]

where \(\varepsilon_s\) is the bar strain, \(s_y\) is the normalized slip when bar strain is equal to yield strain, \(f_u\) is the tensile strength of steel bar, \(f_y\) is yield strength of the bar and \(s\) is normalized slip that is related to bar diameter and concrete compressive strength as follows:

\[
s = \frac{\text{Slip}}{D} \left( \frac{f^c}{200} \right)^{0.67}
\]

where \(D\) is the bar diameter, \(f^c\) is the concrete compressive strength and \(s\) is normalized slip as defined in Eq. (8). Using this model, the moment rotation curve of the wall due to slipping can be computed (Error! Reference source not found.).
5. SHEAR MODELING
Nonlinear analysis of the wall in shear is done through a fixed smeared crack approach by considering the wall as a RC element.
In the smeared crack approaches the cracks and reinforcing bars are idealized as being smeared over the element. The cracks, once generated, are not modeled directly but their effects will be considered by changing the material constitutive models. Generally, the smeared crack approach is conducted by two methods. Rotating crack approach and fixed crack approach. The rotating crack approach assumes that the crack direction coincides with the principal direction of average strain. Accordingly, it can be changed or rotated following the stress condition (Error! Reference source not found.), and because the shear stress vanishes on the continually updated principle planes, no shear model is needed in this method. In the step-by-step computation, one crack is considered and the previous ones are erased. So the rotating crack approach does not explicitly account for shear slip and shear stress transfer due to aggregate interlock. In the fixed crack approach the crack direction once generated will not change during the analysis until the direction changes more than a specific value. So there will be shear stress in the assumed crack direction and the shear transfer due to aggregate interlock will be considered. In this paper the fixed smeared crack approach is used to model the shear behavior of RC wall in an iterative procedure as shown in.

![Figure 10. Rotating smeared crack approach](image1)

![Figure 11. Fixed smeared crack approach](image2)
6. VERIFICATION
To control the accuracy of the adopted method, the analysis results are compared with the experimental behavior of some RC walls. The experimental data for the first example are taken from (Lanker and Mang[12]). The wall shape and reinforcement arrangement are shown in. This wall is analyzed with the adopted method and the results are compared in Figure 13.

The second RC wall is taken from Oesterle et al. [13]. This wall is also analyzed with the adopted method and the results are compared in Figure 15.
Figure 13. Experimental and analytical behavior of the first wall

Figure 14. Geometry and reinforcement arrangement of the second wall[14]
7. CONCLUSION
In this paper a general macro modeling method is proposed that can include flexural and shear behavior of the wall by considering the effects of reinforcing bars pull put and slippage, concrete tension softening, stiffening and confinement. This method can be used to predict the nonlinear response of RC walls by considering all important characteristics and behavioral response features, and is applicable to different wall shapes and the amount of reinforcement. To study the accuracy of the proposed method, the results are compared with some experimental results and their good agreement is shown.

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SEISMIC EVALUATION OF IRREGULAR REINFORCED CONCRETE STRUCTURES

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ABSTRACT
Seismic evaluation of the behavior of irregular structures is one of the most important steps in the retrofitting process of such structures. Irregularity can be in the elevation or in the plan of a structure. Irregularity in plan shape which is due to the difference between the position of the center of stiffness and the mass center of a structure caused by architectural requirements is usually inevitable. In this study the analytical and experimental models of project CM-4 Structural Retrofit Strategies, part of the Mid-America Earthquake Center Core Research Program under the Thrust Area Consequence Minimization, tested at full scale at the European Laboratory for Structural Assessment (ELSA) of the Joint Research Center (JRC), Ispra, Italy are modeled in the general purpose ABAQUS. The analytical results of the models generated in ABAQUS compare favorably with the experimental and analytical results of the project. Having verified the reliability the accuracy of the adopted analysis methods the seismic behavior of an irregular reinforced concrete building designed according to the Iranian Code of Practice for Seismic Resistant Design of Buildings, Standard No. 2800 has been investigated using the model. The investigated parameters include the capacity of the building, the maximum displacement, the relative displacement of the stories and the dynamic characteristics of the building.

Keywords: seismic evaluation; irregular structures; maximum displacement; relative displacement

1. INTRODUCTION
One of the most important factors in design of each structure against lateral loads or in retrofitting an existing building is the better understanding of the behavior and response of it against such loads. In fact, having enough information about the behavior of an structure can result in more accurate, safer, and more economical design of it. Seismic evaluation of an structure is greatly influenced by its geometry, regularity, and irregularity. Different aspects of response and behavior of regular structures can be predicted to some extent, but in case of irregular ones it is otherwise. Thus according to different building codes much attention should be paid to the design of such structures for which the results of nonlinear and dynamic analyses are used to determine the validity of a design. Given the observations noted above, seismic evaluation of irregular structures is essential. In this study
having verified the capability of an analytical model to simulate the nonlinear behavior of an irregular structure investigated experimentally, the provisions of the Iranian seismic code concerning design of irregular structures are also checked through the nonlinear static and dynamic analyses of an irregular RC building designed according to the mentioned provisions.

2. GENERAL DESCRIPTION OF THE TEST BUILDING
2.1. Geometry of the Test Model
The structure is a simplification of an actual three-story building which is a representative of older construction in Southern Europe without earthquake design provisions. It is also similar to pre-seismic code construction in many other parts of the world. The test building has been designed for gravity loads alone, using the concrete design code applied in Greece between 1954 and 1995. Total dead loads and 30% of live loads are used for the gravity loads in the analysis. An overview of the test building and the plan of a typical repetitive floor are presented in Figure 1. Infill walls and stairs are omitted in the model. Dimensions of the building are represented in Figure 2 and details of member dimensions and reinforcement are represented in Figure 3. Slabs are omitted in the analytical model and their contribution to beam stiffness and strength is reflected by effective width of the T-section. For the modeling of beams, a reinforced concrete T-section is utilized and the effective flange width is assumed to be the beam width plus 7% of the clear span of the beam on either side of the web (Fardis, 1994).
The plan of the test structure in Figure 1(b) shows that beams adjacent to C6 are not in alignment, thus gaps between center lines of beams (B5 and B6) and the column (C6) should be considered in the modeling of the beam column connection at C6. As shown in Figure 4, rigid elements are utilized to connect center lines of beams and columns in order to model the force transfer between members and torsion due to gaps between center lines of members.

### 2.2. Assumed Material Properties And Modeling Assumptions

Assumed material properties and assumptions for the analytical modeling of the test structure are summarized in Table 1.

### 2.3. Static Pushover Analysis

Nonlinear static pushover analyses are performed in order to estimate overall capacity and basic characteristics of the test structure such as peak base shears and weak directions. The 1st mode shape is utilized in calculating the base shear and distribution of the lateral forces on the structure. It should be noted that according to FEMA356 in case of irregular structures lateral load should be applied in both directions with 100% load in one direction and 30% in the orthogonal direction. Distributions of equivalent lateral load on the plan and pushover curves of the test structure are represented in Figure 5 and Figure 6, respectively. Using the assumptions summarized in Table 1 nonlinear static pushover analysis of the test structure is performed in ABAQUS and the curve obtained is compared with the one of ELSA model in Figure 7.
Dynamic time history analysis using ground motion accelerations, as recommended mostly by building codes, has been used to investigate the seismic behavior of the test structure. Failure prevention was considered as an important criterion for selection of a record to obtain more controllable results and a stream of good response data in the real test. After observation and comparison of analysis results Montenegro 1979 (Herceg Novi) was selected. Two orthogonal components of the selected semi-artificial record, Montenegro 1979 (Herceg Novi) with peak ground acceleration (PGA) intensity of 1g are presented in Figure 8.
Table 1: Assumed Material Properties and Assumptions for the Analytical Modeling
Both of ELSA Model & Main Model

<table>
<thead>
<tr>
<th>Items in analytical modeling</th>
<th>Assumptions</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Analytical Modeling in ZeusNL</td>
</tr>
<tr>
<td>Reinforcement steel</td>
<td>(FeB32K from Italian market)</td>
</tr>
<tr>
<td></td>
<td>$f_y=459$ MPa ($\Phi 12$)</td>
</tr>
<tr>
<td></td>
<td>$f_y=377$ MPa ($\Phi 20$)</td>
</tr>
<tr>
<td></td>
<td>$E_2/E_1=0.0032$ ($\Phi 12$)</td>
</tr>
<tr>
<td></td>
<td>$E_2/E_1=0.0056$ ($\Phi 20$)</td>
</tr>
<tr>
<td></td>
<td>$E_1=206000$ MPa</td>
</tr>
<tr>
<td></td>
<td>Compressive strength $f_c=25$ MPa</td>
</tr>
<tr>
<td>Concrete</td>
<td>K = 1.01, from Mander et al. (1988)</td>
</tr>
<tr>
<td></td>
<td>Reinforcement steel</td>
</tr>
<tr>
<td></td>
<td>Bilinear Elasto-plastic model</td>
</tr>
<tr>
<td></td>
<td>Concrete</td>
</tr>
<tr>
<td></td>
<td>based on Mander et al. (1988)</td>
</tr>
<tr>
<td>Stress-strain relationship</td>
<td>Self weight of RC member</td>
</tr>
<tr>
<td></td>
<td>Gravity loads</td>
</tr>
<tr>
<td></td>
<td>Seismic dead load for mass calculation</td>
</tr>
<tr>
<td></td>
<td>Mass distribution</td>
</tr>
<tr>
<td></td>
<td>P-delta effect</td>
</tr>
<tr>
<td></td>
<td>Viscous damping</td>
</tr>
<tr>
<td>Structural Modeling</td>
<td>Analysis program</td>
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<tr>
<td></td>
<td>Element model Centerline dimensions</td>
</tr>
<tr>
<td></td>
<td>Rigid offset at beam column connection</td>
</tr>
<tr>
<td></td>
<td>M-M-N interaction</td>
</tr>
<tr>
<td></td>
<td>Effective flange width of T-beams</td>
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<td></td>
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<td></td>
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</tr>
</tbody>
</table>
2.3. NONLINEAR DYNAMIC ANALYSIS

After scaling down their PGA to 0.12g, 0.14g and 0.16g, they were applied to the building in eight different sets of directions as shown in Figure 9. Each combination of directions is defined as D1-D8, respectively.

Montenegro 1979 (Herceg Novi) record at 0.15g PGA in the direction D1 had been selected as an appropriate earthquake scenario for the test. Using the same record at 0.15g PGA in the direction D1 nonlinear dynamic analysis of the test structure is performed in ABAQUS and the curve obtained is compared with the one of ELSA model in Figure 10 and Figure 11. Which verifies the accuracy of the adopted analysis methods.

Having verified the reliability of the analytical model generated and analyzed in ABAQUS, the seismic behavior of an irregular reinforced concrete building designed according to the Iranian Code of Practice for Seismic Resistant Design of Buildings, Standard No. 2800, has been investigated using the model, so that the
possible deficiencies of the mentioned code becomes evident. (The designed model is called the main model at the rest of the paper)

![Figure 10. Comparison of the Top Displacement Histories of ELSA Investigation and the Model Generated in ABAQUS (X direction)](image)

![Figure 11. Comparison of the Top Displacement Histories of ELSA Investigation and the Model Generated in ABAQUS (Y direction)](image)

3. DESIGN OF AN IRREGULAR RC BUILDING

The type of irregularity of the building should be selected in such a way that covers most of the existing or under construction buildings. As observed in most of the structures, due to architectural and urban planning provisions, irregularity is usually in plan shapes. Thus a building with irregular plan shape and simple enough to be simulated in ABAQUS has been selected (Figure 12). The structure of the model has been designed according to the Iranian Code of Practice for Seismic Resistant Design of Buildings, Standard No. 2800, and ACI 318-99 in ETABS.V.8.2.7.
3.1. Simulation of the Main Model in Abaqus
Assumed material properties and assumptions for the analytical modeling of the main model are summarized in Table 2. A model proposed by Mander et al. (1988) is adopted for stress-strain relationships of confined concrete and evaluating the confining effect K (Figure 13).

![Stress-Strain Relationship of Confined Concrete](image)

Figure 13. Stress-Strain Relationship of Confined Concrete

3.2. Modal Analysis
In order to understand the overall response of the structure, periods and mode shapes are obtained using 3D modeling in ABAQUS. Three main mode shapes and corresponding frequencies are presented in Figure 14.
3.3. Nonlinear Static Analysis

As mentioned in previous sections nonlinear static pushover analyses are performed in order to estimate overall capacity and basic characteristics of a structure such as peak base shears and weak directions, and according to FEMA356 in case of irregular structures lateral load should be applied in both directions with 100% load in one direction and 30% in the orthogonal direction. Thus in this phase of the study the model has been subjected to lateral load first in one direction (X) and then in both directions (100% load in one direction and 30% in the orthogonal direction) (Figure 15), and the results are presented in Figure 16.

3.4. Nonlinear Dynamic Analysis

Time-history dynamic analysis using ground motion accelerograms has been recommended mostly by building codes. According to the Iranian Code of Practice for Seismic Resistant Design of Buildings, Standard No. 2800 ground motion effects can be applied either by acceleration spectrum or time history. In this study, according to the code, three scaled accelerograms have been used. (Table 3 and
Table 4). The results of the nonlinear dynamic analysis subjected to Montenegro 1979 (semi artificial) ground motion, are indicated in Figure 17, Figure 18, and Figure 19.

<table>
<thead>
<tr>
<th>PGA (y)</th>
<th>PGA (x)</th>
<th>Earthquake</th>
</tr>
</thead>
<tbody>
<tr>
<td>1g</td>
<td>1g</td>
<td>Montenegro1979 (semi artificial)</td>
</tr>
<tr>
<td>0.933g</td>
<td>0.878g</td>
<td>Tabas</td>
</tr>
<tr>
<td>0.312g</td>
<td>0.214g</td>
<td>El-Centro</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>Max PGA after Scaled</th>
<th>Scale Factor</th>
<th>Max PGA</th>
<th>Items</th>
<th>Earthquake</th>
</tr>
</thead>
<tbody>
<tr>
<td>Montenegro1979 (semi artificial)</td>
<td>0.35g</td>
<td>0.35</td>
<td>1g</td>
<td>PGA (X)</td>
<td></td>
</tr>
<tr>
<td>Tabas</td>
<td>0.457g</td>
<td>0.52</td>
<td>0.878g</td>
<td>PGA (X)</td>
<td></td>
</tr>
<tr>
<td>El-Centro</td>
<td>0.5151g</td>
<td>1.801</td>
<td>0.214g</td>
<td>PGA (X)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.3868g</td>
<td>1.249</td>
<td>0.312g</td>
<td>PGA (Y)</td>
<td></td>
</tr>
</tbody>
</table>

4. CONCLUSIONS
Since the building has been designed according to the Iranian Standard No. 2800, its provisions and the observations of similar building codes can be used to evaluate the results of the analyses.

Figure 17. Base Shear-Time Response of the Model in Both Directions (X, Y)

Figure 18. Top Displacement-Time Response of Column C5 in Both Directions (X, Y)
4.1. The Conclusions of the Nonlinear Static Analysis

1. According to the capacity curve obtained through static pushover analysis, the maximum inelastic displacement of the structures with a period of about 0.7 second, 2.5%, determined by FEME and Iranian seismic design code, is observed in the main model and softening of the model begins at the displacement of about 40cm.

2. The displacement of about 40cm is observed in the obtained capacity curve. In this region, the amount of the force does not change considerably, and the structure withstands the applied load at the large displacement caused by yield and failure of the longitudinal reinforcement of the members until the collapse of the building. Comparing the results with the results of the model tested by ELSA, it can be concluded that the structure designed according to the Iranian seismic code is much more ductile and can absorb much more energy.

3. As mentioned before, according to the provisions of seismic codes, irregular structures should be loaded in both directions which, as observed in the ELSA model, accounts for the reduction in the ductility of the structure (Figure). While the results obtained by the analysis of the main model (designed by the authors) prove the fact that in the provisions of the Iranian seismic code this phenomenon has been taken into account and no reduction in the ductility of the model due to bidirectional loading has been observed.

4.2. The Conclusions of the Nonlinear Dynamic Analysis

1. The relative lateral displacement of stories of the structure subjected to El Centro
and Tabas accelerograms has not exceeded the maximum amount defined in the Iranian seismic code, except for the lateral displacement of the third story resulted from Montenegro accelerogram which was about 4cm and 1.33% of the allowable relative displacement in the code.

2. The amount of the torsion is considerable and as illustrated in Figure 14 the torsional mode shape is the third mode shape with a period of 0.536 sec which is about the period of the first mode shape (0.659 sec). as was expected, column C5 is subjected to the maximum amount of torsion.

4.3. Final Conclusion
Considering all the results stated above it can be concluded that the provisions of the Iranian Code of Practice for Seismic Resistant Design of Buildings, Standard No. 2800 are appropriate for seismic design of the irregular reinforced concrete structures having 5 stories or being 18 meters in height.

REFERENCE
11. ABAQUS USER MANUAL Ver. 6.5.1
NONLINEAR ANALYSIS OF REINFORCED CONCRETE FRAMES INCLUDING MODELING OF JOINT AND BEAM-COLUMN ELEMENTS

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ABSTRACT
In this paper a new method for nonlinear analysis of two dimensional reinforced concrete frames is proposed. In numerical modeling each frame is divided into two types of joint and beam-column elements. The effect of bond-slip has been considered in the formulation of beam-column element by removing perfect bond assumption from fiber method. Joint elements are formulated upon major behaviors including Pull-out of embedded longitudinal bars, shear and flexural deformation of joint panel and shear slip in interface section between joint and neighboring element. Four types of joint elements have been generated according to their position in the frame. Each element type has been modeled based on the major behaviors of that through the combination of one or more defined mechanisms and sub-elements. The reliability of the method has been assessed through the comparison of numerical and experimental results for a one bay two storey frame and a good agreement between experimental and analytical results is observed.

Keywords: nonlinear analysis, RC frames, bond-slip, joint element

1. INTRODUCTION
Much effort has been devoted in the last forty years to the development of models of nonlinear analysis of reinforced concrete frame (RCF). These researches can be classified into three categories: behavior of steel and concrete materials, interaction between bars and concrete and finally, numerical method for nonlinear analysis. In the field of material behavior, numerous models have been proposed, among which the Kent-Park model [1] is the most popular model for the stress-strain relationship of concrete and that of Giuffre-Menegotto-Pinto (GMP) [2,3] model for steel bars. In the field of interaction between concrete and bars, Eligehausen et al. [4] proposed a model for bond stress-slip between bars and concrete. Also other researchers such as Muguruma et al., Hawkins et al., Mirza et al. and Mehlhorn et al. proposed their models for bond stress-slip relationship [5]. Other researchers complemented and modified the previous models according to their experimental works and proposed their models [6]. In the beginning two-component model was proposed by Clough et al. [7] for the numerical analysis of RCF. After that several concentrated plasticity constitutive models have been proposed to date. Such
models include stiffness degrading in flexure, pinching in shear, and fix-end rotations due to bar’s Pull-out. Then a more accurate description of the inelastic behavior of reinforced concrete members became possible with distributed nonlinearity models. The most promising model for the nonlinear analysis of reinforced concrete elements is, presently, fiber section model. In this model the element is subdivided into longitudinal steel and concrete fibers. The constitutive relation of the section is derived by integration of the response of the fibers, which follows the uni-axial stress-stain relationship of materials. The fiber model, basically, adopts the perfect bond assumption [8]. Limkatanyu & Spacone [9] have suggested a method based on fiber section for modeling beam or column reinforced concrete element, but instead of the perfect bond assumption, they have considered bond-slip effect. Meanwhile, microscopic modeling of RCF and their elements with and without bar-concrete interaction in finite element domain has been proposed, but because of its cost the researchers prefer to suggest the simpler methods. Moreover a variety of beam-column joint models have been proposed by researchers. Some of the earliest works to simulate the inelastic response of RCF were based on the calibration of the Plastic-Hinge formation within beam-column elements to introduce the inelastic action of the joint [10]. Another generation of joint models is decoupling the inelastic response of the beams, columns, and joints to facilitate model calibration. One such model is the zero-length rotational spring element that has been used in order to connect beam to column elements and thereby represent the shear distortion of the joint [11]. More recently, researchers have begun using continuum type elements to represent the response of reinforced concrete joints. This type of formulation greatly increases the computational effort of the analysis but offers the potential for high resolution, accurate, and objective modeling of the joint region. One of these models is proposed by Lows et al. [12]. The major sources of deformation in reinforced concrete frame (RCF) are flexural rotation in beams and columns, shear deformation of joints including shear sliding and bar-concrete interaction such as bar’s slip. In this study the behavior of frame elements arises from a combination of these deformation mechanisms. In order to achieve this goal two types of elements have been modeled, one is beam-column element which hereafter is called “BCE” and the other is joint element that is called “JE”. BCE has been generated based on fiber method but the effect of bar-concrete interaction is imposed into equilibrium equations. Also JE is made up of a few mechanisms and sub-elements.

2. BEAM-COLUMN ELEMENT

The free body diagram of an infinitesimal segment $dx$ of BCE is shown in Figure 1. In formulation only the bond stress tangential to the bars is considered and the bar’s dowel effect is neglected. Each BCE is a combination of one 2-node concrete frame element and $n$ number of 2-node bars with bond interfaces. $n$ is the number of longitudinal bars in cross section of BCE. This element has been proposed by Limkatanyu and Spacone [9]. Slippage has been allowed to occur because the nodal degrees of freedom of the concrete element and the bars are different. Based
on the small deformation assumption, all of the equilibrium conditions have been considered. Considering axial equilibriums in the concrete element and steel bars and also vertical and moment equilibriums in the segment $dx$ lead to matrix form of equations which is shown in Equation 1.

$$\partial B^T D_B(x) - \partial B^T D_b(x) - P(x) = 0$$  \hspace{1cm} (1)$$

Where: $D_B(x) = [\mathbf{D}(x) : \mathbf{D}(x)]^T$ is BCE section forces. $\mathbf{D}(x) = \{N_a(x) M_a(x)\}^T$ is concrete element section forces. $\mathbf{D}(x) = \{N_i(x)....N_n(x)\}^T$ is bar axial forces. $D_b(x) = \{D_{01}(x)....D_{0n}(x)\}^T$ is bond section forces. $P(x) = [0 \quad p_i(x) \quad 0.....0]^T$ is BCE force vector. $\partial_B, \partial_b$ are differential operators and are defined in the following forms:

$$\partial_B = \begin{bmatrix} \partial_B & 0 \\ 0 & \partial_B \end{bmatrix}, \quad \partial_b = \begin{bmatrix} d \quad 0 \\ dx & 0 \quad d^2 \quad dx^2 \end{bmatrix}$$

$$\partial_B = \begin{bmatrix} \frac{d}{dx} & 0 & ... & 0 \\ 0 & \frac{d}{dx} & ... & 0 \\ ... & ... & ... & 0 \\ 0 & 0 & ... & \frac{d}{dx_{-n+1}} \end{bmatrix}, \quad \partial_b = \begin{bmatrix} -1 & \frac{d}{dx} & 1 & ... & 0 \\ ... & ... & ... & ... & ... \\ -1 & \frac{d}{dx} & 0 & ... & 1 \end{bmatrix}_{n \times (2+n)}$$  \hspace{1cm} (2)$$

$y_n$ is the distance of bar $n$ from section reference axis (Figure 1). The BCE section deformation vector conjugate of $D_b(x)$ is $d_B(x) = [\bar{\mathbf{D}}(x) : \bar{\mathbf{D}}(x)]^T$. In which
\( \bar{d}(x) = [\bar{e}_{B}(x) \quad \kappa_{B}(x)]^{T} \) contains concrete element section deformations and 
\( \bar{\varepsilon}(x) = [\varepsilon_{1}(x) \ldots \varepsilon_{n}(x)]^{T} \) contains the axial strain of the bars. Displacement vector in the
cross section of BCE is defined as \( \mathbf{u}(x) = \left[ \mathbf{\bar{u}}(x) : \mathbf{\bar{u}}(x) \right]^{T} \). In which
\( \mathbf{\bar{u}}(x) = [u_{1B}(x) \quad u_{2B}(x)]^{T} \) contains concrete element axial and transversal
displacements, respectively, and \( \mathbf{\bar{u}}(x) = [u_{1}(x) \ldots u_{n}(x)]^{T} \) contains the axial
displacements of the bars. From small deformation assumption, the element
deformations are related to the element displacements through the following relation:

\[
\mathbf{d}_{B}(x) = \partial_{B} \mathbf{u}(x) \quad (3)
\]

The bond slips of bars are determined by the following relation between the bar
and concrete element displacements:

\[
u_{bi}(x) = u_{i}(x) - u_{1B}(x) + y_{i} \frac{d u_{2B}(x)}{dx} \quad (4)
\]

Where \( u_{bi}(x) \) is the bond slip between bar \( i \) and surrounding concrete. By
introducing the bond deformation vector as \( \mathbf{d}_{b}(x) = [u_{b1}(x) \ldots u_{bn}(x)]^{T} \), Equation
4 can be written in the following matrix form:

\[
\mathbf{d}_{b}(x) = \partial_{b} \mathbf{u}(x) \quad (5)
\]

The weak form of displacement based finite element formulation is determined
through the principle of stationary potential energy. The BCE nodal displacement
(\( \mathbf{U} \)) which is shown in Figure 2 serve as primary element unknowns and the section
displacement \( \mathbf{u}(x) \) are related to it through displacement shape function matrix. The
relation between nodal displacements and internal deformations can be written
through transformation matrix.
The nonlinear behavior of BCE derives from the nonlinear relation between the section forces and the section deformations through section and bond stiffness matrices \((k_s(x), k_b(x))\). Section stiffness matrix included axial and bending stiffness of concrete element \((EA(x)\) and \(EI(x)\)) also axial stiffness of the bars \((E_nA_n(x))\). Bond stiffness matrix is diagonal and included slope of bond force-slip relationship of each bar \((k_{bn}(x))\). By using the fiber section method, section stiffness matrix is derived. In this method, the stress-strain relationships of steel and concrete are needed. The bond stiffness matrix is derived through the bond stress-slip relation and perimeter of each bar. These relationships are selected according to Table 1.

<table>
<thead>
<tr>
<th>Concrete stress-strain for compressive region</th>
<th>Park –Kent model [1] and later extended by Scott et al. [13]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete stress-strain for tensile region</td>
<td>Gilbert &amp; Warner model [14]</td>
</tr>
<tr>
<td>Steel stress-strain</td>
<td>Giuffre-Menegoto-Pinto model [3]</td>
</tr>
<tr>
<td>Bond stress-bond slip</td>
<td>Eligehausen et al. model [4]</td>
</tr>
<tr>
<td>Shear stress-shear slip</td>
<td>Walraven model [15]</td>
</tr>
<tr>
<td>Shear stress-shear deformation</td>
<td>Anderson et al. model [16]</td>
</tr>
</tbody>
</table>

3. JOINT ELEMENT

In order to model the response of such JE, two sub-elements and two significant mechanisms have been considered. The sub-elements are: a concrete and a reinforced concrete deep beam. In which the effects of shear and flexural deformations has been considered based on Timoshenko beam theory. The mentioned mechanisms are: Pull-out of beam or column longitudinal bars embedded in the joint (Pull-out failure), and shear-transfer at the BCE-joint interfaces (shear slip). The number of degrees of freedom in each side of JE is compatible with the degrees of freedom in the ends of BCEs that are in the neighboring of the JE. Thus, it will be possible to assemble the global matrix and vectors of RCF. In numerical modeling, depending on the position of JE in the RCF, four types of JE can be defined through the combination of sub-elements and mentioned mechanisms. So, firstly, the sub-elements and the mechanisms have been described.

4. REINFORCED CONCRETE SUB-ELEMENT

Reinforced concrete sub-element hereafter is called “RCSE”. In a similar way to which described for BCE, the infinitesimal segment of RCSE has free body diagram similar to Figure 1. In this sub-element the effect of shear deformation has been considered based on Timoshenko beam theory. Also slippage has been allowed to occur. Considering axial equilibriums in the concrete part and steel bars, also vertical and moment equilibriums in the segment \(dx\), lead to the matrix form of equations which is presented in Equation 1. The definitions in this equation are valid but \(\mathbf{D}(x)\) has been rewritten as \(\mathbf{D}(x) = [N_g(x) V_g(x) M_g(x)]^T\) and \(\mathbf{D}_B\) and \(\partial_b\)
have been rewritten as Equation 6.

\[
\bar{\delta}_n = \begin{bmatrix}
\frac{d}{dx}
& 0
& 0

\frac{d}{dx}
& 0
& -1

0
& \frac{d}{dx}
& -1
\end{bmatrix}, \quad \bar{\delta}_b = \begin{bmatrix}
-1 & 0 & y_1 & 1 & 0 & \ldots & 0
-1 & 0 & y_2 & 0 & 1 & \ldots & 0
\ldots & \ldots & \ldots & \ldots & \ldots & \ldots & \ldots
-1 & 0 & y_n & 0 & 0 & \ldots & 1
\end{bmatrix}^{T_n(3+n)}
\]

The RCSE section deformation vector conjugate of \( D_n(x) \) is \( d_n(x) = [\bar{d}(x) : \bar{a}(x)]^T \). In which \( \bar{a}(x) = [\epsilon_\beta(x) \quad \gamma_\beta(x) \quad \kappa_\beta(x) ]^T \) contains axial, shear and bending deformations of section of concrete element, respectively. \( \bar{a}(x) \) has similar definition to that of BCE. The following displacements are defined at the sub-element level: \( u(x) = [\bar{u}(x) : \bar{u}(x)]^T \) is displacement vector along the RCSE, in which \( \bar{u}(x) = [u_{1B}(x) \quad u_{2B}(x) \quad u_{3B}(x) ]^T \) contains axial, transversal and rotational displacements of concrete element, respectively. \( \bar{u}(x) \) has similar definition to that of BCE. From small deformation assumption, the element deformations are related to the element displacements through the Equation 3. The bond slips of bars are determined by the following relation between the bar and concrete element displacements:

\[
u_{bi}(x) = u_i(x) - u_{1B}(x) + y_i u_{3B}(x)
\]

By introducing the bond deformation vector as \( d_b(x) \), Equation 7 can be written as Equation 5. The weak form of displacement based finite element formulation is determined through the principle of stationary potential energy. RCSE nodal displacement vector is similar to that of BCE (Figure 3-a). The section displacement vector is related to nodal displacement vector through the matrix of shape functions. Then the section deformations and bond slips could be determined through Equations 3 and 5.

The nonlinear behavior of RCSE derives from nonlinear relation between the section forces and the section deformations through section and bond stiffness matrices. The section stiffness matrix included axial, shear, and bending stiffness of concrete element \( (E_A(x), G_A(x) \quad \quad E_I(x)) \), also axial stiffness of the bars \( (E_n A_n(x)) \). The bond stiffness matrix and method for calculation of these matrices are similar to which described for BCE. The section shear stiffness derives from shear stress-shear deformation relationship which is selected according to Table 1. The external load vector of this sub-element derives from the external distributed loads along that, which is shown in Figure 3-a as \( p_{y_1}(x) \) and \( p_{y_2}(x) \), by using the shape function matrix. The distributed loads derive from the internal loads in the JE side sections, which are parallel to that sub-element, based on stress value in concrete and steel fibers of the mentioned side sections. The external load vector will be updated in each load step of nonlinear analysis.
5. CONCRETE SUB-ELEMENT
Concrete sub-element hereafter is called “CSE” and is not reinforced. That is a regular 2-node concrete frame element with three degrees of freedom in each of two ends (Figure 3-b). The formulation of CSE derives from Timoshenko beam theory, fiber method, and material behavior similar to concrete part in RCSE. Also, external load which affects on this sub-element is similar to that of RCSE.

6. PULL-OUT MECHANISM
According to Figure 3-c, simulation of stiffness and strength loss associated with bond strength deterioration for longitudinal reinforcement embedded in the joint is considered. If nodal displacement vector related to Pull-out behavior is defined as
\[
\mathbf{u} = [u_1^1, u_2^1, u_3^1, u_1^2, u_2^2, u_3^2]^T,
\]
the slippage of the bars can be defined as below:
\[ \text{slip} = \begin{bmatrix} s_1^i \\ s_2^i \\ \vdots \\ s_n^i \end{bmatrix} = \begin{bmatrix} -1 & 0 & y_1 & 1 & 0 & 0 \\ -1 & 0 & y_2 & 0 & 1 & 0 \\ \vdots & \vdots & \vdots & \vdots & \vdots & \vdots \\ -1 & 0 & y_n & 0 & 0 & 1 \end{bmatrix} U = A_{\text{slip}} U \] (8)

In which, \( y_n \) is the distance of bar \( n \) from reference line. The relationship between Pull-out force and slip for embedded bar number \( n \) in the section 1 can be defined as \( f_n^i = k_{\text{slip}}^i \times s_n^i \). In which \( f_n^i \) is Pull-out force and \( k_{\text{slip}}^i \) is slip stiffness of Pull-out behavior. This equation derives from the bond stress-slip relationship related to Pull-out behavior which is selected according to Table 1, embedded length of the bar, conditions of end of the bar and perimeter of the bar section. Because Pull-out force is summation of bond stress on around surface of each embedded bar. The relationship between Pull-out force and slip of all bars in the section 1 can be written in the following matrix form:

\[ \text{f}_{\text{slip}} = k_{\text{slip}}^i \times \text{slip} \] (9)

Where \( k_{\text{slip}}^i \) is a diagonal matrix which includes \( k_{\text{slip}}^i \) and \( \text{f}_{\text{slip}} \) is Pull-out force vector according to \( \text{slip} \) vector.

The nodal force vector can be expressed in the following form:

\[ \text{F} = A_{\text{slip}}^T \text{f}_{\text{slip}} = A_{\text{slip}}^T k_{\text{slip}}^i \text{slip} = A_{\text{slip}}^T k_{\text{slip}}^i A_{\text{slip}} U = K_{\text{slip}} U \] (10)

From Equation 10, Pull-out stiffness matrix related to section 1 can be written as \( A_{\text{slip}}^T k_{\text{slip}}^i A_{\text{slip}} \). The Pull-out stiffness matrix will be imposed into stiffness matrix of \( JE \). Also, in order to calculate resisting force vector related to Pull-out behavior and impose it into the resisting force vector of \( JE \), it can be written as \( A_{\text{slip}}^T \text{f}_{\text{slip}} \).

7. SHEAR SLIP
According to Figure 3-d, in this method an interface shear component has been considered to represent shear slip and reduction sliding shear. According to degrees of freedom in shear direction in the specified side of \( JE \), shear slip can be defined as below:

\[ \Delta_{\text{shear slip}} = U_{2}^i - Y = \begin{bmatrix} 0 \\ 1 \end{bmatrix} \begin{bmatrix} Y \\ U_{2}^i \end{bmatrix} = A_{\text{shear slip}}^T \begin{bmatrix} Y \\ U_{2}^i \end{bmatrix} \] (11)

If shear force-shear slip relation in the side of \( JE \) can be defined as \( f_{\text{shear slip}} = k_{\text{shear slip}} \Delta_{\text{shear slip}} \), the stiffness matrix related to this mechanism and
specified degrees of freedom can be written as $A_{\text{shear slip}}^T k_{\text{shear slip}} A_{\text{shear slip}}$. Also, in order to calculate resisting force vector and impose it into the resisting force vector of $JE$, it can be written as $A_{\text{shear slip}}^T f_{\text{shear slip}}$. Shear force-shear slip relation is generable by help of shear stress-shear slip relationship according to Table 1 and integration of shear stress over the side surface of the $JE$.

8. TYPE OF JOINT ELEMENTS

<table>
<thead>
<tr>
<th>Joint element type</th>
<th>2*Concrete sub-element</th>
<th>3*Reinforced concrete sub-element</th>
<th>4*Reinforced concrete sub-element</th>
</tr>
</thead>
</table>

Table 2. Types of joint elements in a two dimensional RCF

♦ Number of sub-elements which is used in assembling.

9. NONLINEAR ANALYSIS AND NUMERICAL VALIDATION

In order to analyze $RCF$ based on the proposed method, a computer program has been developed. The solution of equilibrium equations is typically accomplished by an iterative method through a convergence check. In this research the Newton-Raphson method is used as nonlinear solution algorithms [17]. Also the Gauss-Lobatto method is used for numerical integration in which the number of integration points is equal to five. For demonstrating the ability and reliability of the proposed method, verification for a one bay two storey $RCF$ is presented. This frame is loaded laterally at the level of second storey and was tested by Vecchio and Emara [18]. The geometry of the specimen and the details of the cross sections are shown in Figure 4. Two 700kN of axial loads was imposed on the column before applying lateral load. Required information for numerical modeling such as bar and concrete material specifications are used as reported in [18]. In numerical modeling, beams and columns will be divided into enough number of $BCE$s. Because, the formulation is displacement based and the response is depend on element size and it is needed the length of $BCE$ be enough short. As a simple suggestion, the length of $BCE$ can be selected equal or smaller than average crack spacing in beam or column. In these cases convergence will be achieved in the numerical results. The equation which is given by CEB-FIP [19] is adapted for calculation of average crack spacing. Average crack spacing has been calculated as 112 and 90 mm for beams and columns, respectively. So, in numerical modeling first storey columns, second storey columns and the beams are divided into 20, 17 and 28 $BCE$s, respectively. Numerical analysis is carried out utilizing two approaches. In one approach the bond between bars and concrete is assumed to be perfect and in the other one, the effects of bond-slip such as Pull-out of bars in the joint and slip in $BCE$s are considered. Figure 5-a shows the analytical and
experimental load-displacement responses. Results show that by perfect bond assumption the estimation of stiffness and capacity is higher than experimental values, but for the case with bond-slip effects, the proposed method has good precision for estimating both cases of stiffness and capacity in numerical analysis.

Figure 4. Geometry and details of the tested frame

In order to show the capability of the proposed method in considering Pull-out effect, four analyses with a variety of embedded length of the longitudinal bars of the columns in the footing is carried out. The results are shown in Figure 5-b in which, the case with 300 mm embedded length is the existing value of the model and the others are assumed. The results for two cases with 300 and 1000 mm length are approximately similar. It can be concluded for the existing model because of the sufficiency of embedded length, the Pull-out of the bars in the base is very insignificant.

Figure 5. Load-displacement response of the tested frame (a): comparing bond-slip effect; (b): comparing Pull-out effect

10. CONCLUSION
In this paper a new method for nonlinear analysis of two dimensional RCF is proposed. Each RCF is divided into two types of joint (JE) and beam-column
(BCE) elements. The effect of bond-slip has been considered in the formulation of $BCE$. Formulation of the $BCE$ is based on the displacement method and displacement shape functions have been used in order to express the internal displacements in term of nodal displacement. $JE$s are formulated upon major behaviors including pull-out of embedded longitudinal bars, shear and flexural deformation of $JE$ and shear slip in interface section between joint and neighboring element. Four types of $JE$s have been generated according to their position in the frame based on major behaviors of them through the combination of one or more defined mechanisms and sub-elements.

In order to utilize the nonlinear analysis based on the proposed method, a computer program is developed and reliability of the method has been assessed through the comparison of numerical with experimental results for a one bay two storey $RCF$. The comparison shows a good agreement between experimental and analytical results. The proposed method can be used as an efficient numerical method for nonlinear analysis of $RCF$ because of its capability for modeling of $JE$, with the consideration of bond-slip effect, pull-out of bars in the joints, shear slip and shear deformation.

REFERENCES
THE ANALYSIS OF FLEXIBLE CONCRETE FOUNDATIONS ON COARSE ALLUVIUM OF TEHRAN

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²Associate Professor of Geotechnical Engineering, Civil Engineering Dept., University of Tehran, Iran

ABSTRACT

Regarding the increasing expansion of construction in Tehran, the design and construction of raft or grid concrete foundations are very common for tall buildings. Winckler springs are often used by professionals to model soils in the design of flexible concrete foundation. However, it presents a substantial problem because the Winckler springs are not coupled. Such a subgrade reaction theory is too simplified, i.e. it considers a fixed constant value for the stiffness of Winckler springs which leads to an incorrect design. Continuum mechanics theory and numerical simulation tools are available to be used for soil modeling but they are time consuming and most engineers prefer to use Winckler springs to model the soil. In the presented research, a computer program was firstly developed based on finite elements method. It could model a flexible foundation on springs but the springs are coupled by considering stress distribution within soil mass. The used method is very similar to Winckler springs model and tries to modify it. The method is employed for different geological formations in Tehran to show the effects of the properties of coarse alluvium of Tehran on the analysis of concrete foundations.

Keyword: tehran coarse grain alluvium, concrete foundation, stress distribution, deformation

1. INTRODUCTION

Importance of mat foundations:

In recent years, methods of analysis and structure construction have undergone many changes but the use of mat and grid foundations for tall buildings goes back to many years and such concrete foundations have shown their proven role in the transfer of construction forces to the ground.

A number of mat concrete foundation analysis methods, such as strips method, equivalent slab method, beam on flexible foundation or elasticity methods (Backer, 1957), have been used. Despite this, with advances in the methods of finite elements, the analysis of a slab on flexible foundation is the most used and spring is used for soil simulation [1, 2, and 3].

Mat concrete foundations can be assumed as a flexural beam or slab. Only this beam/slab relies on the ground at various points. Since the soil has less stiffness than concrete, ground response could be the most important variable in the design
of flexible concrete foundations and consequently plays an important role for analysis of concrete foundations. Therefore, the soil response is discussed for various geological formations of Tehran soil in this paper.

2. GEOLOGY OF TEHRAN DEPOSITS [4,13]:
Riben (1966) was a European geologist who worked for the Geological Survey of Iran (GSI) in the mid-20th century. His classification of Tehran alluvia is still widely used by local geologists and engineers. Riben (1966) divided the Tehran coarse-gained alluvia in two four categories, identified as A, B, C and D, where A is the oldest and D the youngest. Some simple profiles, produced by Fakher et al to schematically show the sequence of Tehran alluvia formation, are presented in Figure 1 and specifications of these alluvia are presented in Table 1 using conventional terminology.

3. CONVENSIONAL ANALYSIS OF FLEXIBLE MAT FOUNDATIONS [5]
One of the most common methods of designing foundation is the model of beam/slab on elastic subgrade and the displacement of each point is obtained through the following equations:
If we consider a strip footing with the width of B and unlimited length, the differential equation of deformation of this beam is according to Equation 1:

\[ M = E_p I_p \frac{d^2z}{dx^2} \]  

M is internal bending moment of each section, and \( E_p \) and \( I_p \) are elasticity coefficient and moment of inertia of beam section respectively. On the other side,
Equation 2 is valid for any beam:

\[
\frac{d^2M}{dx^2} = Q
\]  

(2)

Q is soil reaction toward the lower surface of foundation. Winckler, 1967 presented soil reaction theory and in fact it depends on the soil elastic module. Soil Reaction theory is simplified elastic theory. \(Q=KZ\) in which q is the pressure under the foundation, Z is elastic settlement, and K is the subgrade reaction coefficient.

<table>
<thead>
<tr>
<th>Factor</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
</tr>
</thead>
<tbody>
<tr>
<td>Age</td>
<td>5Ma</td>
<td>700ka</td>
<td>50ka</td>
<td>10ka</td>
</tr>
<tr>
<td>Lithology</td>
<td>Homogeneous</td>
<td>Heterogeneous</td>
<td>Alluvial fan</td>
<td>Recent alluvial</td>
</tr>
<tr>
<td>Cementation</td>
<td>Cemented and hard</td>
<td>Variable, but</td>
<td>Cementation less</td>
<td>non-cemented</td>
</tr>
<tr>
<td></td>
<td></td>
<td>usually weak</td>
<td>than A and non-hard</td>
<td></td>
</tr>
<tr>
<td>Grain size</td>
<td>Clay to 100-250 mm</td>
<td>Very variable up</td>
<td>Clay to 100-200 mm</td>
<td>Clay to several</td>
</tr>
<tr>
<td></td>
<td></td>
<td>to several meters</td>
<td>mm</td>
<td>meters</td>
</tr>
<tr>
<td>Dip layer (deg)</td>
<td>0-90</td>
<td>0-15</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Thickness (m)</td>
<td>Maximum 1200</td>
<td>Maximum 60</td>
<td>Maximum 60</td>
<td>&lt;10</td>
</tr>
<tr>
<td>Sedimentary environment</td>
<td>Fluvial</td>
<td>Fluvio-glacial and</td>
<td>Fluvial</td>
<td>Fluvial</td>
</tr>
<tr>
<td></td>
<td></td>
<td>periglacial</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Other name (local name)</td>
<td>Hezardareh alluvial formation</td>
<td>Tehran alluvial formation</td>
<td>Recent alluvial</td>
<td>Recent alluvial</td>
</tr>
<tr>
<td>Location of</td>
<td>North area</td>
<td>North area</td>
<td>North and central area</td>
<td>Recent and old riverbed</td>
</tr>
<tr>
<td>observation in Tehran</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The main difference of E and K is that E is among properties of materials but K depends on dimensions and shape of foundation. To calculate subgrade reaction coefficient we have various field methods but Vesic presented an Equation 3 to find the subgrade reaction coefficient which is [6]:

\[
K = 0.65 \left( \frac{E_s}{E_f} \right)^{0.2} \left( \frac{E_f}{E_s} \right)^{0.8} \frac{1}{3.5(1-\nu_f)^{0.5}} \frac{1}{3.5(1-\nu_s)^{0.5}}
\]

(3)

In which \(E_s\) and \(5s\) are respectively elasticity module and soil Poisson coefficient. Subgrade reaction coefficient is a useful method in designing concrete pavements.
of roads and airports. Maximum tensile stress is made in the lower surface of concrete slab upon influence of load on a concrete pavement. To change these roles, we use analysis of slab on the elastic subgrade and for this analysis we need to make the subgrade reaction coefficient clear.

According to Equations 1 and 2, we have:

\[ -Z \cdot K \cdot B = E_F \cdot l_F \frac{\partial^2 z}{\partial x^2} \]  

(4)

By solving the said equation we will have:

\[ Z = e^{-\alpha x} \left( A_1 \cos(\beta x) + A_2 \sin(\beta x) \right) \]  

(5)

In which \( A_1 \) and \( A_2 \) are fixed amounts which are calculated according to boundary conditions and loading. In above relation, \( \beta \) coefficient is gained from Equation 6.

Based on recommendation of commission 436 of US Concrete Institute, if the distances between columns in a strip are less than \( 1.75/\beta \), the foundation is rigid otherwise the foundation will be assumed to be flexible.

\[ \beta = \frac{4 \cdot E \cdot l}{E_F \cdot l_F} \]  

(6)

4. MAIN ASSUMPTIONS OF CONVENSIONAL ANALYSIS

To use soil reaction theory, the substantial assumptions of this theory should be considered in full. The assumptions used in the soil reaction theory (Winckler model [7]) are:

a) The relationship between load (Q) and deflection (Z) is regarded as linear. It means Q is proportionally related to Z.

b) \( K_v \) (soil reaction coefficient in vertical direction) is regarded as fixed at all points of contact of foundation with soil. In fact in Winckler model, soil is considered as a series of elastic and independent (uncoupled) springs but it is not an accurate assumption. Not considering the reciprocal influence of springs on each other means that the spring will change its shape due to application of force to each spring and other springs will not be influenced by this force. In another words, soil in Winckler model has not been regarded as a continuum environment. Although due to connection between the constituting particles, soil is a continuum environment. Therefore in order to obtain the correct answer, springs must be related and connected to each other. In this way if a spring changes its form, some of this deformation will transfer to the adjacent springs.

To solve the problem of uncoupled springs, one can model the soil as a continuum body in a finite elements program. But due to the great number of elements, foundation analysis will be very time consuming. Another option is to consider stress distribution in soil in Winckler springs module.

In this paper the second option has been used to analyze concrete foundation.
5. METHOD OF ANALYSIS USED IN THE PRESENTED RESEARCH

In the presented research, a computer program was developed. It uses finite elements method to solve governing Equations. The soil is modeled as spring but stress distribution within soil mass is considered so the springs are couples. The following steps are undertaken to do the analysis.

**Step 1:**
Concrete foundation is modeled using 4-node flexural plate elements. Each node has 3 degrees of freedom and the stiffness of foundation is presented by Kr [8]. To verify the accuracy of foundation stiffness matrix, the displacement of a rectangular slab (Figure 2) under a uniform load of q is calculated in Table 2. The results are compared with the result of an analytical solution [9] and it shows the results are in a close agreement.

![Square slab with fully fixed boundaries](image)

**Table 2. Displacement in the center-point slab**

<table>
<thead>
<tr>
<th>Number of element</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>analytical solution</th>
</tr>
</thead>
<tbody>
<tr>
<td>Displacement</td>
<td>0.00148</td>
<td>0.00144</td>
<td>0.00140</td>
<td>0.00130</td>
<td>0.00128</td>
<td>0.00126</td>
<td>0.00126</td>
</tr>
</tbody>
</table>

**Step 2:**
Stiffness of Winckler springs is obtained by the use of Vesic relation [6]. To assemble the stiffness matrix of foundation and subgrade, it is enough to add the stiffness of springs to the stiffness related to vertical displacement of each node (the elements on the main diameter of Kr matrix).

**Step 3:**
Displacement of each node is obtained according to \( [F] = [K] \cdot [\Delta] \). In this step, the distribution of stress within soil mass has not been considered so far and foundation analysis is done using the conventional analyses of foundation on
Step 4:
Based on the displacements gained from Step 2 and stiffness of springs calculated in Step 2, we can obtain the forces applied to soil. Now using Equation 7, we can find the displacement of j node due to applying of F force on the ith node [10].

\[
W_j = \frac{R^4 (1 - \nu^2)}{4 \pi R E_f}
\]  

(7)

In which R is the distance of points i and j. Therefore displacement of each node is gained through Equation 8.

\[
\Delta_i = \Delta_i + \sum_{j=1}^{n} \Delta_{ij}
\]

(8)

In which \(\Delta_i\), \(\Delta_t\) and \(\Delta_{rt}\) are respectively new displacements of node i, (by considering stress distribution), displacement of node i from Step 3 (without considering stress distribution), and displacement from real forces on various nodes on ith node.

Step 5:
The displacements obtained in Step 4 and Step 3 are compared to each other. If the precision is not enough, new stiffness of springs will be obtained according to the displacement and the forces applied to soil in Step 4, and the program will be implemented again from Step 3. These Steps are repeated until the displacements gained in Steps 3 and 4 have a slight difference.

6. VERIFICATION OF DEVELOPED PROGRAM
A number of examples have been used to assess the validity of the program. One example is presented here. A rectangular mat foundation in 2 m dimensions and thickness of 20 cm is considered on an elastic half–space (E= 40 MPa and \(\nu=0.3\)). A concentrated force is applied equivalent to the unit to the middle of the edge of foundation (Point C in Figure 3). The results of used method and also the results of mathematical analysis are shown in Table 3 for comparison. The result of "fixed springs" in Table 3 are related to conventional method but the results of "variable spring" are derived when the stiffness of springs were varied according to Step 4, as described above. The comparison shows that the results of Variable spring and Mathematical solution are in a close agreement but the results of fixed spring are different.
Table 3. The results of the used method and mathematical analysis.

<table>
<thead>
<tr>
<th>Difference in percentage of pressure applied below the foundation with regard to mathematical solution</th>
<th>Pressure under the foundation</th>
<th>Difference in percentage of moment in Y direction toward the mathematical solution</th>
<th>Moment in Y direction</th>
<th>Moment difference percentage in X direction toward mathematical solution</th>
<th>Moment in X direction</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.306</td>
<td>-0.07</td>
<td>0.05</td>
<td>Mathematical solution [11, 12]</td>
<td></td>
<td></td>
</tr>
<tr>
<td>%7</td>
<td>0.2845</td>
<td>%1</td>
<td>-0.069</td>
<td>%2</td>
<td>0.049</td>
</tr>
<tr>
<td>%70</td>
<td>0.5202</td>
<td>-%56</td>
<td>-0.109</td>
<td>%28</td>
<td>0.036</td>
</tr>
</tbody>
</table>

7. INFLUENCE OF SOIL STIFFNESS IN FOUNDATION ANALYSIS

Regarding the increasing expansion of construction in Tehran and the type of coarse-grained soil in Tehran, it is highly important to determine the engineering parameters of this soil (the stiffness of spring) for mat concrete foundations analysis. According to the published data from local tests, the stiffness of Tehran soils are summarized in Tables 4&5 for formations of A and C [4, 13]. It should be noted that no accurate data is available for B formation because the size of aggregates are large and consequently large scale in-situ tests should be done to find stiffness of springs.
The values presented by Amini (1996) have been recommended based on the conducted studies on Lines 1 and 2 of Tehran Subway and presented as general suggestions. The values presented by Soil Mechanical Engineering Services Company (SES, 1996) have been presented based on drilling several boreholes, sampling and conducting field and laboratory tests. The values presented by Pahlavan (2002) have been recommended based on field tests (plate loading, shear wave and Menard type pressure meter testing). Values presented by Asghari (2002) have been proposed based on plate bearing and large direct shear test. Values presented by Cheshomi (2006) have been recommended based on the direct shearing experiments and plate bearing and seismic inside well loading.

According to the above mentioned researches, the elasticity module of formations A and C are assumed to be 140 MPa and 70 MPa respectively in this study. Poisson ratio of 0.3 is used for both the types of formation. The soil is considered as linear elastic springs.

A number of examples have been used to study the effect of subgrade stiffness on the analysis of mat foundation. One example is presented in Table 7. A foundation measuring 5×5 meter with the thickness of 500 mm, under uniform load of 15 ton/m² is divided into 64 elements and the results of the analysis are shown in Table 7 for both the types of formations A and C.
### Table 6: Characteristics foundation

<table>
<thead>
<tr>
<th>Foundation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young module (MPa)</td>
</tr>
<tr>
<td>Poisson's ratio</td>
</tr>
<tr>
<td>Bulk module (MPa)</td>
</tr>
<tr>
<td>Shear module (MPa)</td>
</tr>
</tbody>
</table>

### Table 7: Result of investigation on coarse grain alluvium

<table>
<thead>
<tr>
<th>Type of formation</th>
<th>Es</th>
<th>s</th>
<th>Displacement (fixed spring) (m)</th>
<th>Displacement (variable spring) (m)</th>
<th>Percent Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>140</td>
<td>0.3</td>
<td>5.16E-3</td>
<td>9.92E-3</td>
<td>92%</td>
</tr>
<tr>
<td>C</td>
<td>70</td>
<td>0.3</td>
<td>1.05E-2</td>
<td>1.98E-2</td>
<td>85%</td>
</tr>
</tbody>
</table>

8. CONCLUSIONS

According to the presented study, the following conclusions could be proposed:

- When fixed stiffness Winckler springs are used and stress distribution within the soil mass is neglected in the analysis of flexible mat foundations, the obtained moments have a great difference with the mathematical solution. Therefore, the design of concrete mat foundations is not reliable when fixed stiffness Winckler springs is used.

- Based on comparisons with mathematical solutions, the presented program (used coupled spring method) has appropriate accuracy. In addition to Tehran soil, the method presented for analysis of concrete foundations is usable for all types of soils.

- The influence of considering stress distribution within soil mass (coupled springs) in Formation A is more than Formation C. Therefore, the use of fixed stiffness Winckler springs (conventional method used by professional engineers) is highly recommended for extensive usage in Formation A in Tehran for the design of concrete mat foundations.

REFERENCES

3D AND THREE PHASE'S MICROMECHANICAL CONSTITUTIVE MODEL FOR THE UNIAXIAL COMPRESSION TEST OF CONCRETE

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³Associate Prof. of Civil Eng., Civil Eng. Faculty, K.N.T University

ABSTRACT
The mechanical behavior of concrete materials is strongly influenced by its microstructure. The macroscopic properties of concrete materials such as strength and stiffness are dependent on the properties of micromechanics. The advance of composite mechanics and advanced computing technologies has made possible the micromechanical analysis of concrete materials. At first the status of micromechanical modeling with special emphasis on the advantage and disadvantage of each model is presented.

The current paper focuses on the geometrical description and numerical simulation of normal-weight concrete at the mesoscale. In the first part the numerical representation of concrete at the mesoscale is introduced. The internal structure of concrete is considered at the micro level, and is treated as a three phase material comprised of aggregate particle, matrix, and the aggregate-matrix interfaces the generation of the mesoscale geometry, the finite element discretisation and the applied material laws with micro plane theory are described.

The main objective of this paper is to investigate the macroscopic behavior and progressive failure of concrete materials under static loading, as influenced by the properties of its constituents at the meso level.

Keywords: three phases, concrete, itz, finite element, micromechanical

1. INTRODUCTION
Concrete is one of the most popular construction materials, and people have been using it for a long time. Many models, theories and numerical techniques have been developed to represent its mechanical behavior, including a large variety of constitutive models, damage models and other novel developments such as the micro plane model. However, progressively more elaborated constitutive relations have also required a large number of parameters, sometimes difficult to obtain and with no clear physical meaning.

In the 1980s, the meso mechanical approach, sometimes known as numerical concrete, was proposed by Roelfstra et al., and was then followed by others [1]. It consisted of discretizing the first level of material (meso structure) and assigning to each material component its individual geometry and properties. There is no doubt
that the complexity of the nonlinear behavior of concrete may be largely associated to its heterogeneity and components. Therefore, it seems reasonable that considering explicitly each material component (geometry and mechanical properties) will allow us to consider a simpler constitutive assumption in exchange for an increasing size of the global problem. After the pioneering work of Roelfstra, different methodologies for considering the meso structure have been proposed such as lattice models, particle models, continuum meso models, DEM models [2] and FEM models [3].

2. CONCRETE WITH THREE PHASES
Concrete is an artificial heterogeneous composite material which consists of aggregates which are bonded together by cement paste. In numerical simulations of concrete structures a homogenous material is usually assumed at the macroscale. Specific constitutive models allow simulating the fracture process at this macroscopic scale [4]. For different experimental setups other parameters are obtained, which implies, that the predictive capacity of the numerical model is restricted to a specific experiment. The underlying material structure, e.g. the distribution of the aggregates or the aggregate shape, is normally disregarded by these models.[5]

The influence of the material structure on the macroscopic material behavior can be analyzed using mesoscale models. In literature, concrete at the mesoscale is separated into three main components: the homogenous mortar matrix, the aggregates with a diameter greater than 2.0 mm and the interfacial zone between them. The mortar matrix is mainly composed of cement paste and aggregates with a diameter less than 2.0 mm. Porosities within the matrix are disregarded at this length scale. The interfacial transition zone (ITZ) is about 20 − 100 µm deep [6]. The experimental bond strength of the ITZ was about 33% to 67% of the tensile strength of the matrix [7].

3. CONTINUUM DAMAGE MECHANICS
Continuum damage mechanics is a constitutive theory that describes the progressive loss of material integrity due to the propagation and coalescence of micro cracks, micro voids, and similar defects. These changes in the microstructure lead to a degradation of material stiffness observed on the macro scale. The basic premise of continuum damage mechanics is that micro structural defects (micro cracks, micro voids) in a material can be represented by a set of continuous damage variables. An illustration of this concept is given in Figure 1. The value of the damage variable $D$ at a certain point of the continuum is a measure of the number and size of defects in a small volume at this point.

It is assumed in the sequel that the development of damage does not introduce anisotropy into the material behavior and that a single, scalar damage variable suffices to describe the local damage state. In the more general, anisotropic case, a set of damage variables (or a tensor) must be used (Krajcinovic et al., 1981[8]; Lemaitre, 1996[9] Fichant et al., 1995[10]. The damage variable $D$ is defined as $0 \leq D \leq 1$, where $D = 0$ represents the initial, undamaged material and $D = 1$
represents a state of complete loss of integrity. Strictly speaking, the initial material always contains some defects, but it is assumed that these are accounted for in the virgin material properties, so that the initial damage can be set at zero.

After a certain amount of loading, three regions can generally be distinguished in the material domain \( W \) as shown in Figure (1). No damage may have developed at all in a part \( \Omega_0 \). The damage variable still has its initial value \( D = 0 \) in this region and the material properties are those of the virgin material. In a second region \( \Omega_d \), some development of damage has occurred, but the damage is not yet critical \((0 < D < 1)\). The limiting value \( D = 1 \) has been reached in the third region \( \Omega_c \), i.e., the mechanical integrity and strength have been completely lost in this region. The completely damaged region \( \Omega_c \) is the continuum damage representation of a crack.

It is important to realize that the local, complete loss of strength in \( \Omega_c \) implies that stresses are identically zero for arbitrary deformation fields.

![Figure 1. Damage distribution in a continuum [9]](image)

4. LOCAL ISOTROPIC DAMAGE MODEL

Isotropic damage models are based on the simplifying assumption that the stiffness degradation is isotropic, i.e., stiffness moduli corresponding to different directions decrease proportionally, independently of the direction of loading. Since an isotropic elastic material is characterized by two independent elastic constants; a general isotropic damage model should deal with two damage variables. The simplified model with a single variable uses an additional assumption that the Poisson ratio is not affected by damage. The stress-strain law is written in the form:

\[
\sigma = (1 - \omega)D^e : \varepsilon
\]  

(1)

Where \( \sigma \) is the column matrix of stress components, \( \varepsilon \) is the column matrix of engineering strain components, \( D^e \) is the elastic material stiffness matrix, and \( \omega \) is the damage variable. The growth of the damage variable must be described by a suitable evolution equation \( w \), a tensor with multiple components. We define a
scalar measure of strain called the equivalent strain, $\tilde{\varepsilon}$ and evaluate the internal variable $\kappa$ that drives the damage evolution as the maximum value of $\tilde{\varepsilon}$ ever reached in the previous history of the material. Under monotonic loading, $\kappa$ coincides with $\tilde{\varepsilon}$, but during unloading $\kappa$ remains constant while $\tilde{\varepsilon}$ decreases. The choice of a specific expression for the equivalent strain directly affects the shape of the elastic domain in the strain space. For instance, one could define the equivalent strain as the scaled energy norm:

$$\tilde{\varepsilon} = \sqrt{\frac{\varepsilon^T D \varepsilon}{E}}$$

(2)

Scaling by Young’s modulus $E$ is introduced in order to obtain a strain-like quantity that is equal to the longitudinal strain in the special case of uniaxial loading. The multiaxial formulation is then a natural extension of the uniaxial one, and the function $g$ that links $\kappa$ to $\omega$ is the same as for the uniaxial model. For the energy-based equivalent strain, the elastic domain is ellipsoidal and symmetric with respect to the origin and the evolution of damage under tensile loading is the same as under compressive loading. For quasi brittle materials such as concrete, damage evolves much faster under tension than under compression. To take that into account, Mazars [11] proposed a definition of equivalent strain in the form

$$\tilde{\varepsilon} = \sqrt{\sum_{i=1}^{3} \langle \varepsilon_i \rangle^2}$$

(3)

where $\varepsilon_i, i = 1; 2; 3$, are the principal strains, and the brackets $\langle \rangle$ denote the “positive part” operator, given by $\langle x \rangle = \max(0; x)$, i.e., $\langle x \rangle = x$ for $x$ positive and $\langle x \rangle = 0$ for $x$ negative.

5. MICRO PLANE FORMULATION WITH KINEMATICS’ CONSTRAINT

The orientation of a micro plane is characterized by the unit normal $n_i$ (indices $i$ and $j$ refer to the components in Cartesian coordinate’s $x_i$). In the formulation with a kinematics constraint, which makes it possible to describe softening behavior of plane concrete in a stable manner, the strain vector $\varepsilon_N$ on the micro plane (Figure 2) is the projection of the macroscopic strain tensor $\varepsilon_{ij}$ so the components of this vector are:
The normal strain on the micro plane is \( \varepsilon_N = n_i \varepsilon_{Ni} \), that is:

\[
\varepsilon_N = N_{ij} \varepsilon_{ij} \quad N_{ij} = n_i n_j
\]

where repeated indices imply summation over \( I=1, 2, 3 \). The mean normal strain, called the volumetric strain \( \varepsilon_V \) and the deviatoric strain \( \varepsilon_D \) on the microplane can also be introduced which are defined as follows:

\[
\varepsilon_V = \frac{\varepsilon_{kk}}{3} \quad \varepsilon_D = \varepsilon_N - \varepsilon_V
\]

This separation of \( \varepsilon_V \) and \( \varepsilon_D \) is useful when the effect of the hydrostatic pressure for a number of cohesive frictional materials, such as concrete, needs to be captured. To characterize the shear strains on the micro plane (Figure 2), we need to define two coordinate directions \( M \) and \( L \), given by two orthogonal units coordinate vectors \( m_i \) and \( l_i \) lying on the micro plane. To minimize directional bias of \( m \) and \( l \) among micro planes, one of the unit vectors \( m \) and \( l \) tangential to the plane is considered to be horizontal (parallel to \( x - y \) plane). The magnitude of the shear strain components on the micro plane in the direction of \( m \) and \( l \) are as \( \varepsilon_{ij} = m_j (\varepsilon_{ij} n_j) \) and \( \varepsilon_{ij} = l_j (\varepsilon_{ij} n_j) \). Because of the symmetry of tensor \( \varepsilon_{ij} \), the shear strain components may be written as follows:

\[
\varepsilon_M = M_{ij} \varepsilon_{ij}, \varepsilon_L = L_{ij} \varepsilon_{ij}
\]

in which the following symmetry tensors were introduced:

\[
M_{ij} = (m_i n_j + m_j n_i) / 2 \quad L_{ij} = (l_i n_j + l_j n_i) / 2
\]

Once the strain components on each micro plane are obtained, the stress components are updated through micro plane constitutive laws, which can be
expressed in algebraic or differential forms. In the kinematics constraint micro plane models, the stress components on the micro planes are equal to the projections of the macroscopic stress tensor $\sigma_{ij}$ only in some particular cases, when the micro plane constitutive laws are specifically prescribed in a manner such that this condition can be satisfied. This happens for example in the case of elastic laws at the micro plane level, defined with elastic constants chosen so that the overall macroscopic behavior is the usual elastic behavior. In general, the stress components determined independently on the various micro planes will not be related to one another in such a manner that they can be considered as projections of a macroscopic stress tensor. Thus the static equivalence or equilibrium between the micro level stress components and macro level stress tensor must be enforced by other means. This can be accomplished [12a, b, c] by application of the principle of virtual work, yielding

$$\sigma_y = \sigma_y \delta_y + \frac{3}{2\pi} \int_{\Omega} \left[ \sigma_D \left( N_y - \frac{\delta_y}{3} \right) + \sigma_L L_y + \sigma_M M_y \right] d\Omega$$

(9)

Where $\Omega$ is the surface of a unit hemisphere, $\sigma_D$, $\sigma_L$, and $\sigma_M$ are the volumetric and deviatoric part of normal stress component and $\sigma_L$ and $\sigma_M$ are as shear stress components on the micro planes respectively. Equation (9) is based on the equality of the virtual work inside a unit sphere and on its surface. The integration in equation (9) is performed numerically by Gaussian integration using a finite number of integration points on the surface of the sphere. Such an integration technique corresponds to considering a finite number of micro planes, one for each integration point. An approximate formula consisting of 26 integration points is proposed in this study. In Table 1, direction cosines and weights of the integration points and in Figure 2, their positions on the surface of the unit sphere are shown. Based on the formulation, macroscopic constitutive matrix in the proposed model is obtained as follows:

$$D_{ijkl} = \frac{3}{4\pi} \int_{\Omega} \left[ E \left( N_y - \delta_y/3 \right) N_{kl} + M_{ijkl} + L_{ijkl} \right] d\Omega + \frac{E}{1-2v} \frac{\delta_{kl}}{3} \delta_{ij}$$

(10)

In which $E$ and $v$ are as elastic modulus and Poisson’s coefficient.[12,a,b,c,d,e,f,g,h,i]

6. ANISOTROPY DAMAGE FUNCTION FORMULATION FOR THREE PHASES

Total deviatoric part of constitutive matrices is computed from superposition of its counterparts on the micro planes that such counterparts in turn, are calculated based on the damages occurred on each plane depending on its specific loading conditions. This damage is evaluated according to the five separate damage functions; each of them belongs to the particular loading states. These five loading
conditions are as follows: (hydrostatic compression, hydrostatic extension, pure shear, shear + compression, shear + extension) that for three phases (aggregate, matrix and ITZ) are shown in Table (3). The stress-strain diagrams of both aggregate and cement paste are almost linear, except at very high relative stress levels as shown in Figure (3). The stress-strain relationship of concrete, however, has a curved shape, due to the fact that aggregate and cement paste, having different stiffness characteristics, are connected in one bearing system. In concrete types where the stiffness of the matrix is close to the stiffness of the aggregate, the stress-strain relationship of concrete will also be close to linear.

Table 1: Definition of micro planes

<table>
<thead>
<tr>
<th>Direction cosines of integration points</th>
<th>Weights</th>
</tr>
</thead>
<tbody>
<tr>
<td>( m_n )</td>
<td>( n )</td>
</tr>
<tr>
<td>( \frac{1}{3} )</td>
<td>( \frac{1}{3} )</td>
</tr>
<tr>
<td>( -\frac{1}{3} )</td>
<td>( \frac{1}{3} )</td>
</tr>
<tr>
<td>( -\frac{1}{3} )</td>
<td>( \frac{1}{3} )</td>
</tr>
<tr>
<td>( \frac{1}{2} )</td>
<td>0.0</td>
</tr>
<tr>
<td>( -\frac{1}{2} )</td>
<td>0.0</td>
</tr>
<tr>
<td>( \frac{1}{2} )</td>
<td>( \frac{1}{2} )</td>
</tr>
<tr>
<td>( -\frac{1}{2} )</td>
<td>( \frac{1}{2} )</td>
</tr>
<tr>
<td>0.0</td>
<td>( -\frac{1}{2} )</td>
</tr>
<tr>
<td>0.0</td>
<td>( \frac{1}{2} )</td>
</tr>
<tr>
<td>1.0</td>
<td>0.0</td>
</tr>
<tr>
<td>0.0</td>
<td>1.0</td>
</tr>
<tr>
<td>0.0</td>
<td>0.0</td>
</tr>
</tbody>
</table>
Table 2: Damage functions for three phases

<table>
<thead>
<tr>
<th>Load Phases</th>
<th>Matrix</th>
<th>Aggregate</th>
<th>ITZ</th>
</tr>
</thead>
<tbody>
<tr>
<td>hydrostatic compression</td>
<td>$\omega_{\text{agg}} = 1 - \frac{1}{1 + \frac{\varepsilon_{\text{eq}}}{\varepsilon_{\text{eq}}}}$</td>
<td>$\omega_{\text{agg}} = 0$</td>
<td>$\omega_{\text{agg}} = 1 - \frac{\varepsilon_{\text{eq}}}{\varepsilon_{\text{eq}}} \cdot \frac{\varepsilon_{\text{eq}}}{\varepsilon_{\text{eq}}} \cdot \frac{\varepsilon_{\text{eq}}}{\varepsilon_{\text{eq}}} \cdot \frac{\varepsilon_{\text{eq}}}{\varepsilon_{\text{eq}}} \cdot \frac{\varepsilon_{\text{eq}}}{\varepsilon_{\text{eq}}} = 0$</td>
</tr>
<tr>
<td>hydrostatic extension</td>
<td>$\omega_{\text{agg}} = 0$</td>
<td>$\omega_{\text{agg}} = 0$</td>
<td>$\omega_{\text{agg}} = 0$</td>
</tr>
<tr>
<td>pure shear</td>
<td>$\omega_{\text{agg}} = (\omega_s + \omega_f)$</td>
<td>$\omega_{\text{agg}} = (\omega_s + \omega_f)$</td>
<td>$\omega_{\text{agg}} = (\omega_s + \omega_f)$</td>
</tr>
<tr>
<td>shear + compression</td>
<td>$\omega_{\text{agg}} = 0$</td>
<td>$\omega_{\text{agg}} = 0$</td>
<td>$\omega_{\text{agg}} = 0$</td>
</tr>
<tr>
<td>shear + extension</td>
<td>$\omega_{\text{agg}} = 0$</td>
<td>$\omega_{\text{agg}} = 0$</td>
<td>$\omega_{\text{agg}} = 0$</td>
</tr>
</tbody>
</table>

On each micro plane at each step of loading, there exists one specific loading situation that may be in one of the five mentioned basic loading conditions. For every mood, a specific damage function according to the authoritative laboratory test results available in the literature is assigned. Then, for each state of on plane loading, one of the five introduced damage functions will be computed with respect to the history of micro-stress and strain components. These five damage functions are as below parameters $a - k$ in the above relations are computed according to laboratory results obtained for each specific concrete. In equation (9), $\varepsilon_{\text{eq}}$ is as average strain and in the other relations is as the magnitude of projected
deviatoric strain vector on each microplane.

Figure 4. Local stresses around an aggregate particle [16]

7. FAILURE MECHANISM
Figure 4 describes the stress situation around an aggregate particle. The usual failure sequence, independent of the character of loading, is the exceeding of:

- Tensile bond strength;
- Shear bond strength;
- Tensile and shear strength of the cement matrix and
- Tensile strength of the aggregate particles.

The difference in the area of the failure surface results in differences in compressive and tensile properties of both mortar and concrete.

8. GEOMETRICAL MODEL

With a grading curve, therefore we can use simple methods instead of complex numeral and randomness or x-ray methods to find effective diameter and use it to determine two arrangements with maximum and minimum aggregate volume as a repeatable basically element. As a result we can use this element to model the behavior of sample concrete in meso scale and three phases. In the mesoscale model, to simplify the problem, the shape of the coarse aggregate is assumed to be circular and the ITZ zone is modeled as a thin boundary layer around the aggregate. ITZ is a zone in the vicinity of a coarse aggregate, which is formed between bulk cement paste and the aggregate. Its formation is due to the water filled pores near the aggregate and the wall effect. According to static experimental and numerical results, it is well known that ITZ plays a very important role in a concrete mix and it is considered to be the weak link in a concrete composite.

For distribution of aggregates a cubic element fill of isometric spheres on two arrangements could be considered:

- a): regular lap
- b): regular compact

For modeling, two composites as shown in Figures (5, 6) are considered under a loading state of simple uniaxial compression. Due to symmetry, only a quarter of each composite needs to be analyzed and its finite element idealization is shown in the same Figures so it has a thin thickness and the usual continuum elements used in a conventional finite element analysis can not model this reign with real thickness.
satisfactorily, such an element would cause computational difficulties as its thickness decreases, then t=0.1 cm is employed for its thickness.

Figure 5. Arrangement a-meshing with three phases

Figure 6. Arrangement b-meshing with three phases

9. MODEL PARAMETERS
In this formulation, we consider just three basic material parameters for every phase's elasticity and Poisson's coefficients and fracture energy that is based on extensive experimental data from literature for mortar and concrete according to table No. 3 are given. [13, 14]

<table>
<thead>
<tr>
<th>Material parameter</th>
<th>Aggregate</th>
<th>Mortar Matrix</th>
<th>ITZ</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young’s Modulus (E) (MPa)</td>
<td>60,000</td>
<td>30,000</td>
<td>10,000</td>
</tr>
<tr>
<td>Poisson’s Ratio ((\alpha))</td>
<td>0.2</td>
<td>0.23</td>
<td>0.2</td>
</tr>
<tr>
<td>Tensile damage threshold</td>
<td>4e-4</td>
<td>2e-4</td>
<td>1e-4</td>
</tr>
<tr>
<td>Compressive damage threshold</td>
<td>4e-3</td>
<td>2e-3</td>
<td>1e-3</td>
</tr>
<tr>
<td>Compressive strength (MPa)</td>
<td>80</td>
<td>50</td>
<td>13</td>
</tr>
<tr>
<td>Tensile strength (MPa)</td>
<td>16</td>
<td>3.4</td>
<td>2</td>
</tr>
</tbody>
</table>
10. NUMERICAL EXAMPLE
In this example the proposed micro-plane isotropic damage model is implemented in the finite element code. To establish the validity of the proposed 3d and three phases concrete model, correlation studies of analytical results with experimental evidence from the stress-strain response of concrete specimens under different loading conditions are presented in the following.

11. UNIAXIAL COMPRESSION (UC) TEST
As can be seen in Figure 7, there is a good agreement between the results that were fulfilled by the proposed model and experimental evidences. Experimental observations were experienced by Kupfer and his co-workers in 1969[.]. Obviously, there exists an excellent coincidence between the analytical and laboratory data.

12. CONCLUSION
A new damage formulation has been employed into the micro-plane model. This damage formulation has been built on the basis of five fundamental force conditions that can essentially occur on each micro-plane for three phases. Consequently, any arbitrary change of six strain/stress components and change of the characteristics of component (aggregate, matrix, ITZ), normal concrete, high strength concrete led to a combination of five introduced on plane conditions. Therefore, the proposed model is capable of predicting the concrete behavior with any change. The five damage evolutions are functions of equivalent strain that were formulated for any of the five stated conditions. The equivalent strain for the first two conditions are defined as limitation in volumetric strain and for the others is the superimposed projections of deviatoric strain tensor on the corresponding micro-plane. The proposed model has excellent features such as pre failure strain distribution inside material which led to the final failure mechanism, however the basis of its formulation is simple, logical and has some physical insights that make it convenient to perceive.

Figure 7. Uniaxial compression test of concrete obtained with proposed micro plane damage model
REFERENCE


MODELING CONCRETE BEHAVIOR UNDER REVERSED CYCLIC LOADING

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ABSTRACT
The computational simulating and analysis of reinforced concrete structures nonlinear behavior is subjected to reversed cyclic loading dependent on the modeling of nonlinear constitutive laws of materials. Nonlinear behavior of structural concrete includes cracking, crushing, tension stiffening, compression softening and bond slip where reversed cyclic loadings introduces further complexities, such as stiffness degradation in concrete and the Bauschinger effect in reinforcing steel. In this paper the reliability of presented constitutive models for concrete subjected to reversed cyclic loading that considers transition curve between compression and tension by using crack closing model is investigated. In the analysis of reinforced concrete structures, a number of diverse approaches have been used for material modeling. These include plasticity-based procedures, fracture mechanics procedures, and various nonlinear elastic models where this simulation is related to constitutive models based on elasticity approach. For these aims, by reviewing the results of experimental tests on concrete specimens under cyclic loading in both compression and tension, and by reviewing suggested constitutive models by various researchers especially important simple models, the accordance of these constitutive models with experimental results are investigated.

Keywords: constitutive models, reinforced concrete structures, nonlinear behavior, reversed cyclic loading, crack closing model

1. INTRODUCTION
Although experimental programs in laboratories for identifying nonlinear behavior of reinforced concrete (RC) structures give real results but they are limited to the knowledge of particular cases under restricted structural dimensions, sizes, shapes, loading and boundary conditions but the computational simulation approach has no limit to its application. A significant research effort to characterize the monotonic and cyclic behavior of concrete has been devoted to this task and these research efforts have increased even more with recent development of computational methods applicable to RC structures. A large variety of concrete models have been produced in the last years. These models can be categorized according to three approaches: models which are based on theory of elasticity, models which are based on theory of plasticity and models based on fracture mechanics (CEB [1]).
Also, some combinational models based on plasticity and fracture mechanics theory have been developed. Although it has been proved that the models derived from theory of plasticity and fracture mechanics theory can accurately simulate the observed behavior of concrete, its application in the engineering practice is reduced. This is motivated by the great amount of parameters that are usually needed and the difficulty to obtain them through conventional laboratory tests. In the context of this study, only simplified models which are essentially mathematical formulations derived from the generalization of test results for concrete under various loading histories are treated.

Many of these models have been documented in the literature, like Sinha et al. [2], Karsan and Jirsa [3], Yankelevsky and Reinhardt [4], Mander, Priestley and Park [5], Chang and Mander [6], Bahn and Hsu [7], Elmorsi et al. [8], Palermo and Vecchio [9], Mansour and Hsu [10] and Sima et al. [11] among others. Most of them refer only to the compressive cyclic behavior of concrete and only a few consider the cyclic tension response. Also, some other researchers have considered tension behavior of concrete under cyclic and monotonic loading. Several expressions have been documented in the literature to represent the softening branch, including straight lines (Baźant and Oh [12]), polylinear curves (Hillerborg et al. [13]), exponential curves (Gopalaratman and Shah [14]) and Sima et al. [11]), polynomial curves (Lin and Scordelis [15]) and their combinations.

Sinha et al. [2] were the first researchers to describe qualitatively and quantitatively the stress-strain response of concrete under cyclic loading. The experiment was undertaken on a series of 48 tests that were performed on concrete cylinders with compressive strength from 20 to 28 MPa and subjected to cyclic axial compressive loading in order to determine the main factors governing the cyclic response of concrete. Karsan and Jirsa [3] later demonstrated that unloading and reloading are not unique and are dependent on the previous load history. They developed an experimental program consisting of 46 short rectangular columns of plain concrete under cyclically varying axial loads to investigate further the findings of Sinha et al. [2]. They concluded that there exists an envelope curve that can be represented by monotonic response of similar concrete properties. They considered the residual plastic strain as the principal parameter to determine the unloading curve equation and proposed an empirical formula to correlate the residual plastic strain with the point on the envelope from which unloading starts. When reloading starts from zero stress to meet the envelope curve, it is found that the reloading curve becomes rather flat in most of its range and maybe represented by a simple straight line (Sinha et al. [2]) or a second-order parabola (Karsan and Jirsa [3]).

Bahn and Hsu [7] developed a parametric study and an experimental investigation on the behavior of concrete under random cyclic compressive loading. They studied a set of parameters in a semiempirical way that controls the overall shape of cyclic stress-strain curve. This was carried out by combining the theoretical simulation and a series of experimental results. A power type equation was proposed for the unloading curve and a linear relationship for the reloading curve. Palermo and Vecchio [9] proposed a constitutive model for concrete consistent with a compression field approach. The concrete cyclic model presented by the
authors considers concrete in both compression and tension. The unloading and reloading curves are linked to the envelope curves, which are represented by the monotonic response curves. Unloading is modeled using a Ramberg–Osgood formulation, considering boundary conditions at the onset of unloading and at a zero stress. Reloading is modeled as a linear curve with degrading reloading stiffness. Mansour and Hsu [10] developed an extension of the Softened Membrane Model (Hsu and Zhu [16]) subjected to reversal cyclic shear stresses. This work includes a cyclic uniaxial constitutive relationship for concrete that takes into account a “softening” of the concrete compressive strength caused by a constant tensile strain in the orthogonal direction. The unloading and reloading curves were formed by a set of pieced linear curves. Sima et al. [11] developed a constitutive model for concrete subjected to cyclic loadings in both compression and tension. Particular emphasis has been paid to the description of the strength and stiffness degradation produced by the load cycling in tension and compression, the shape of unloading and reloading curves and the transition between opening and closing of cracks. Two independent damage parameters in compression and in tension have been introduced to model the concrete degradation due to increasing loads. However, some authors (Okamura and Maekawa [17], Hordijk [18]) have provided an accurate approximation of the complete unloading–reloading cycle in tension. In this paper, firstly the proposed monotonic stress–strain curves (envelope curves) in literature for concrete were compared with each other and experimental result tests in monotonic compression and tension loading. As a result of these comparisons, the suitable envelope curves were selected for cyclic constitutive models. Secondly, the developed constitutive models for concrete under cyclic compression loading in literature were compared with each other and experimental result tests in cyclic compression and tension loading. As a result of these comparisons, the simple and reliable models that have more consistency with experimental result tests are selected for simulation of RC structures.

2. THE COMPUTATIONAL SIMULATION OF NONLINEAR BEHAVIOR OF CONCRETE UNDER REVERSED CYCLIC

2.1. Envelope Curve in Compression Loading

The monotonic curve adopted as envelope should verify some desirable characteristics: the slope at the origin should be equal to the initial modulus of deformation, it should describe correctly the ascending and the descending post peak (softening) branch and it should permit us to adjust the post peak behavior to experimental results.

In this paper, the most important monotonic compression stress–strain curves relationships of concrete are summarized in Table1.

2.2. Envelope Curve in Tension Loading

In the pre-peak branch, a linear elastic relationship represents well the behavior in tension and most researchers have used this approach. The post-peak behavior is in some cases modeled as an abrupt fall to zero stress (perfect-brittle material). However, this simplification in the post-peak behavior does not agree with the
experimental results and can produce incoherent results when it is applied in a computational model. In this paper, the most important monotonic tension stress–strain curves relationships of concrete are summarized in Table 2.

2.3. Unloading and Reloading Curves for Cyclic Compressive Constitutive Models
When a concrete specimen is monotonically loaded up to a certain strain level and then unloaded to a zero stress level in a typical cyclic test, the unloading curve is concave from the unloading point and characterized by high stiffness at the beginning. The stiffness gradually decreases and becomes very flat at low stress levels and the residual plastic strains are considerably reduced. When reloading is performed from zero stress up to the envelope curve, it has been observed that the curve is rather flat in almost all of its length. The aim of modeling the shape of the unloading and reloading curves is to capture the damage accumulation and the energy dissipation of the material due to cyclic loading. Several types of curves have been used to reproduce the unloading curve also, like the Ramberg–Osgood equation used by Palermo and Vecchio [9] or Chang and Mander [6], the power type used by Bahn and Hsu [6] or the multilinear curve. In turn, reloading can be accurately modeled by a linear curve as is done by most researchers (Palermo and Vecchio [9], Bahn and Hsu [7], among others). The most important cyclic compression constitutive models (include: unloading curve, reloading curve, plastic strain point, common point and etc.) of concrete are summarized in Table 3. Based on cyclic compression constitutive models in Table 3, two constitutive models consisting of Bahn and Hsu [7], Elmorsi et al. [8] are selected, also, Sinha et al. [2] experimental results are selected to control the accuracy and ability of these cyclic constitutive models (Figures 1 and 2). The drawings of these constitutive models have been performed by using MATLAB programming software.

2.4. Unloading and Reloading Curves for Cyclic Tensile Constitutive Models
The response of concrete under cyclic tension has been studied in detail by Reinhardt [20] and Reinhardt et al. [21]. More than 100 tests were performed on plain concrete under cyclic tension and numerical expressions for the softening branch and the unloading and reloading curves were derived. It was observed (like in the case of plain concrete under cyclic compression loadings) that the reloading curve does not return to the envelope curve at the previous maximum unloading strain and further straining is needed for taking up the envelope curve again. This phenomenon is less important than in compression. The energy dissipated in a tension cycle without incursions in the compression zone can be neglected when it is compared with the energy dissipated in a complete compression cycle. The most important cyclic tension constitutive models (include: envelope curve, unloading curve, reloading curve, plastic strain point, common point etc.) of concrete are summarized in Table 4. Based on cyclic tension constitutive models in Table 4 two constitutive models consisting of Foster and Marti [22] (used by Petersson [23] envelope curve) and Sima et al. [11] are selected, additionally, Reinhardt [20] experimental results are selected to control the accuracy and ability of these cyclic constitutive models.
(Figures 3 and 4). The drawings of these cyclic tension constitutive models have been performed by using MATLAB programming software.

### 2.5. Transition Curves

A series of tests attempting to characterize the effect of damage in tensions when the specimen is loaded in compression were developed by Ramtani et al. [29]. These test results have shown that completely closing the cracks requires a certain amount of compression. Once the crack is closed, the stiffness of the concrete is not affected by accumulated damage in tension. The transition curve from tension to compression, once the damage in tension is produced, closing the cracked zones is assumed to be linear which is in agreement with the experimental results. The Elmorsi et al. [8] and Sima et al. [11] (using Legeron et al. [30] transition curve) transition curves are compared with Reinhardt [20] experimental result test (Figures 5 and 6). The drawings of these cyclic tension constitutive models have been performed by using MATLAB programming software.

### Table 1: The Monotonic Compression Stress-Strain Curves Relationships of Concrete

<table>
<thead>
<tr>
<th>Envelope Curve</th>
<th>Envelope Curve relationships</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modified Hognestad [24]</td>
<td>( \sigma_c = f'<em>c \left[ 1 - 0.15 \left( \frac{e'<em>c - e</em>{uc}}{e</em>{uc} - e_c} \right) \right] ) if ( e'<em>c \leq e_c \leq e</em>{uc} ) ( \sigma_c = f'<em>c \left[ 2e</em>{uc} \left( \frac{e_c}{e_{uc}} \right)^2 \right] ) if ( e_c \leq e'_c )</td>
</tr>
<tr>
<td>Young and Smith [25]</td>
<td>( \sigma_c = E_c e )</td>
</tr>
<tr>
<td>Saenz [26]</td>
<td>( \sigma_c = \frac{E_c e}{1 + \left( \frac{1}{n} \left( \frac{e}{e_c} \right) \right)} ) if ( e_c / e'_c &lt; 1 ) ( k = 1 )</td>
</tr>
<tr>
<td>Collins and Mitchell [27]</td>
<td>( \sigma_c = \frac{n}{n-1} \left( \frac{e}{e_c} \right) ) if ( e_c / e'_c &gt; 1 ) ( k = 0.67 + \frac{e'}{62} ) Mpa</td>
</tr>
<tr>
<td>Mazars and Pijaudier-Cabot [28]</td>
<td>( \sigma = \begin{cases} \varepsilon E_c &amp; \varepsilon \leq \varepsilon_0 \ \varepsilon_0 \left( 1 - A \right) + A \varepsilon E_c &amp; \varepsilon &gt; \varepsilon_0 \ \end{cases} ) ( A = \frac{f'_c - \varepsilon_0 E_c}{\varepsilon_0} )</td>
</tr>
</tbody>
</table>

Description:
- \( e'_c \): the equivalent strain at peak stress \( f'_c \): the peak compressive stress
- \( \varepsilon_0 \): the strain at the elastic limit
- \( E_c \): the initial modulus of concrete
Table 2: The Monotonic Tension Stress-Strain Curves Relationships for Concrete

<table>
<thead>
<tr>
<th>Envelope Curve</th>
<th>Envelope Curve relationships</th>
<th>Envelope Curve Shape</th>
</tr>
</thead>
<tbody>
<tr>
<td>Heillerborg et al. [13]</td>
<td>Polylinear curve: Bilinear stress-strain model $\alpha_1 = 0; \quad \alpha_2 = \alpha_3 = \frac{2E_cG_f}{l_{ch}f_t^2}$</td>
<td><img src="image" alt="Polylinear curve: Bilinear stress-strain model" /></td>
</tr>
<tr>
<td></td>
<td>Polylinear curve: The linear softening law $\alpha_1 = l/3; \quad \alpha_2 = \frac{2}{9}\alpha_1 + \alpha_1; \quad \alpha_3 = \frac{18}{5}\frac{E_cG_f}{l_{ch}f_t^2}$</td>
<td><img src="image" alt="Polylinear curve: The linear softening law" /></td>
</tr>
<tr>
<td>Petersson [23]</td>
<td>$A = \frac{k_0\varepsilon_0 - f_t}{\varepsilon_0 - f_t}$</td>
<td><img src="image" alt="Petersson" /></td>
</tr>
<tr>
<td>Yankelevsky and Reinhardt [31]</td>
<td>$A = \frac{k_0\varepsilon_0 - f_t}{\varepsilon_0 - f_t}$</td>
<td><img src="image" alt="Yankelevsky and Reinhardt" /></td>
</tr>
<tr>
<td>Izumo, shin, Maekawa and Okamura [32]</td>
<td>$\sigma_{th} = f_t \left(\frac{2E_c}{E}\right)^{0.4}$</td>
<td><img src="image" alt="Izumo, shin, Maekawa and Okamura" /></td>
</tr>
<tr>
<td>Sima et al. [11]</td>
<td>$\sigma = E_0\varepsilon_{eq}\left(\frac{l}{l_{eq}}\right), \quad \alpha = \left(\frac{G_fE_0}{f_t^2}\right)\frac{l}{l_{eq}}, \quad \varepsilon_{eq} \geq 0$</td>
<td><img src="image" alt="Sima et al." /></td>
</tr>
</tbody>
</table>

Description:
- $l_{ch}$: the characteristic length
- $G_f$: the fracture energy
- $f_t$: the peak tension stress
- $\varepsilon_{th}$: the cracking strain
- $E_c$: the initial modulus of concrete
- $w$: crack width
- $\varepsilon_{eq}$: the equivalent strain at peak stress

3. DISCUSSION ON RESULTS

The concrete cyclic models consider concrete in compression and concrete in tension. The unloading and reloading rules are linked to backbone curves, which are represented by the monotonic response curves. The backbone curves are adjusted for compressive softening and confinement in the compression regime, and for tension stiffening and tension softening in the tensile region. In Figures 1-2, comparison of the Sinha et al. [2] experimental test with two cyclic compression constitutive models consisting of Bahn and Hsu [7] and Elmorsi et al. [8] are shown. As result of this comparison, Bahn and Hsu [7] constitutive model has a suitable result but plastic strain points that was used in these models relatively is not suitable, also, Elmorsi et al. [8] model is a simple model that can be improved by considering suitable unloading and reloading curves. In Figures 3-4, the comparison of Reinhardt [20]...
experimental test with two cyclic tensions constitutive models consisting of Foster and Marti [22] and Sima et al. [11] are shown. As a result of this comparison Sima et al. [11] Foster and Marti [22] and constitutive models have suitable outcome but these models can improve their ability by using compatible plastic strain points. In Figures 5-6, the comparison of Reinhardt [20] experimental test with two transition curves consisting of Elmorsi et al. [8] and Sima et al. [11] are shown. As a result of this comparison Sima et al. [11] (using Legeron et al. [30] transition curve) model has shown a suitable result but these models can improve their ability by using compatible plastic strain points. In addition, Elmorsi et al. [8] has a relative suitable ability to simulating transition curve.

Table 3: The Cyclic Compression Constitutive Models of Concrete

<table>
<thead>
<tr>
<th>Model</th>
<th>Plastic Strain, Common and Reloading Points</th>
<th>Unloading and Reloading Curves</th>
</tr>
</thead>
<tbody>
<tr>
<td>Karsan and Jirs [3]</td>
<td></td>
<td>Unloading Branch: ( \varepsilon_p = \varepsilon_c \left( \frac{1.013}{\varepsilon_c} \right) )</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Reoading Branch: ( \varepsilon_p = \varepsilon_c \left( \frac{1.013}{\varepsilon_c} \right) )</td>
</tr>
<tr>
<td>Elmorsi et al. [8]</td>
<td></td>
<td>Unloading and Reloading Branch modelled by a linear response</td>
</tr>
<tr>
<td>Bahn and Hsu [7]</td>
<td></td>
<td>Unloading Branch: ( \sigma = \sigma_p + \varepsilon_p \left( \sigma_{un} - \sigma_p \right) )</td>
</tr>
<tr>
<td>Palermo and Vecchio [9]</td>
<td></td>
<td>Reoading Branch: ( \sigma = \sigma_{ppu} + \varepsilon_p \left( \sigma_{re} - \varepsilon_p \right) )</td>
</tr>
<tr>
<td>Sima et al. [11]</td>
<td></td>
<td>Unloading Branch: ( f_c(\varepsilon) = f_{un} + E_2(\varepsilon - \varepsilon_{un}) )</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Reoading Branch: ( f_c(\varepsilon) = f_{un} + E_2(\varepsilon - \varepsilon_{un}) )</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Unloading Branch: ( R = E_p \frac{\varepsilon_{un}}{\varepsilon_p} = \varepsilon_{un} / \varepsilon_p )</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Reoading Branch: ( R = E_p \frac{\varepsilon_{un}}{\varepsilon_p} = \varepsilon_{un} / \varepsilon_p )</td>
</tr>
</tbody>
</table>

\( \varepsilon, \sigma, \delta \) : the compressive damage at unloading point

\( \varepsilon, \sigma, \delta \) : the compressive damage at unloading point
Figure 1. Comparison between Sinha et al. [2] experimental test with Bahn and Hsu [7] constitutive model

Figure 2. Comparison between Sinha et al. [2] experimental test with Elmorsi et al. [8] constitutive model

Figure 3. Comparison between Reinhardt [20] experimental test with Foster and Marti [22] constitutive model
Figure 4. Comparison between Reinhardt [20] experimental test with Sima et al.[11] constitutive model

Table 4. The Unloading and Reloading Curves of Cyclic tension Constitutive Models of Concrete

<table>
<thead>
<tr>
<th>Model</th>
<th>Unloading and Reloading Curves</th>
</tr>
</thead>
</table>
| Yankelevsky and Reinhardt [33]| \[
\begin{align*}
\sigma_0 &= \frac{0.00}{225.0}; \sigma_1 = \frac{0.00}{5.0}; \sigma_2 = \frac{0.00}{75.0}; \sigma_3 = \frac{0.00}{125.0}; \\
\sigma_P &= \frac{-0.5}{f_1}; S_P = \frac{f_P}{\sigma_1} + 0.5S_2; \sigma_G = \frac{f_1}{S_2} + 0.5S_4; \\
\sigma_B &= \frac{b_3}{b_2} + \frac{b_1}{b_2}; S_3 = \frac{f_3}{S_4} + 0.5S_4; \\
\sigma_\alpha &= \frac{c_3}{c_2} - c_1 + \frac{c_2}{c_1}; S_4 = \frac{f_4}{S_4} + 0.5S_4; \\
\sigma_E &= \frac{0}{S_4} + \frac{f_4}{S_4} + \frac{f_4}{S_4}; S_5 = \frac{f_5}{S_4} + 0.5S_4.
\end{align*}
\] |

| Okumura and Maekawa [17]     | Loading: \( \sigma_\alpha \left( \frac{f_0}{f_1} + \epsilon \right) \leq R_f f_1 \), \( \epsilon \geq \epsilon_{\text{max}} \); \( \alpha = \text{slip} \left( \frac{\sigma_\alpha}{\sigma_0 \left( \epsilon - \epsilon_{\text{max}} \right)} \right) \left( \frac{\epsilon - \epsilon_{\text{max}}}{\epsilon_{\text{max}} - \epsilon_{\text{max}}} \right)^{\alpha} \)

\[
\begin{align*}
\sigma_\alpha &= \frac{\sigma_{\text{max}} - \sigma_0}{\epsilon_{\text{max}} - \epsilon}; \text{ Unloading Branch: } \sigma_\alpha &= \frac{E_{\text{pl}} \left( \epsilon - \epsilon_{\text{max}} \right) x + \sigma_0 \epsilon_{\text{max}}}{R_f f_1} \quad \epsilon < \epsilon_{\text{max}} \\
\sigma_\alpha &= \frac{\sigma_{\text{max}} - \sigma_0}{\epsilon_{\text{max}} - \epsilon}; \text{ ReLoading Branch: } \sigma_\alpha &= \frac{\sigma_{\text{max}} - \sigma_0}{\epsilon_{\text{max}} - \epsilon}; \sigma_\alpha \leq R_f f_1 \quad \epsilon < \epsilon_{\text{max}}
\end{align*}
\] |

| Foster and Marti [22]        | A straight line is used for the unloading branch in tension. The same curve is considered for the reloading branch when there is no incursion in compression during a cycle. The plastic strain point in unloading or reloading is located at half length of unloading strain point. |

| Palermo and Vecchio [9]      | \( \Delta \varepsilon = E_{\text{pl}} - \varepsilon - \varepsilon_{\text{pl}} = 146 \varepsilon_{\text{pl}} + 0.523 \varepsilon_{\text{pl}} \) |

| Sima et al. [11]             | A straight line is used for the unloading branch in tension. The same curve is considered for the reloading branch when there is no incursion in compression during a cycle. Based on experimental data from Reinhardt [13], the following criterion is proposed to account for the stiffness deterioration: \( E_{\text{pl}} = \frac{E_0 \epsilon_{\text{pl}}}{\epsilon_{\text{pl}} + 1.05} \) |
4. CONCLUSION

From the comparison of monotonic compression and tension stress–strain curves of concrete, cyclic compressive and tensile constitutive models of concrete and transition curves, the following conclusions were drawn.

1. Experimental results have shown that the monotonic compression and tension curves of concrete present a linear response until approximately a half of the compressive and tensile strengths. Due to that, a first linear relation is considered until the elastic limit is reached. A nonlinear type equation is considered for the envelope stress-strain curve of concrete beyond the elastic limit. As result of above comparisons, Collins and Mitchell [28] envelope curve in compression and Sima et al. [11] envelope curve in tension have good agreement with experimental data.

2. Unloading response is assumed nonlinear, in the case of full loading, terminates
at the plastic offset strain. Models for the compressive and tensile plastic offset strains have been formulated as a function of the maximum unloading strain in the history of loading. Reloading is modeled as linear with a degrading reloading stiffness. The reloading response does not return to the backbone curve at the previous unloading strain, and further straining is required to intersect the backbone curve. The degrading reloading stiffness is a function of the strain recovered during unloading and is bounded by the maximum unloading strain and the plastic offset strain. Based on the comparisons, Bahn and Hsu [7] cyclic compression constitutive model and Sima et al. [11] and Foster and Marti [22] cyclic tension constitutive models have shown suitable ability to simulation of nonlinear behavior of concrete.


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SEISMIC BEHAVIOR OF SHORT COLUMNS IN RC STRUCTURES

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ABSTRACT
Civil engineering structures as well as office or apartment building are affected by earthquakes. A common cause of failure seems to be shear stress. The earthquake forces developed at different floor levels in a building need to be brought down along the height to the ground by the shortest path. Short column phenomenon is one of the effective causes of buildings failure in past earthquakes. This destructive phenomenon is due to column height difference in a story level that is predominantly because of localing building on sloppy ground. These buildings have unequal height columns along the slope, which causes ill effects like twisting and damage in shorter columns. In some buildings, few or no walls are provided at the first story (pilot). In the structures with difference in story level, major problems is due to discontinuity of floor diaphragm that causes significant changes in period, stiffness distribution of earthquake force and seismic loading of structures. In this research, at first, seismic behavior of short column phenomenon is determined, then, nonlinear behavior of reinforced concrete short columns in 4, 8 and 10 story structures with story level difference is investigated. Short columns and mentioned structures are analysed under the earthquake record of Elcentro with different peak ground acceleration with IDARC software which is nonlinear dynamic analysis program. In this investigation, the results of maximum response, base shear, global damage index and displacement time history and effect of short column in structural failure is evaluated.

Keywords: building with different floor, reinforced concrete, short column, damage index, nonlinear dynamic analysis

1. INTRODUCTION
In dublex structures, story floors with level difference relative to each other are made in two or more different height levels. The effective length of column in interface of these structures are divided in smaller sizes, that each of them act as a short column. In structures, the important difficulties are of lake connection diaphragm. Diaphragms play important role in transering the lateral forces between resistant parts against earthquake as each disorder or separation in diaphragm floor cause stress concentration in their junction with vertical parts. Their most important role are transferring of intertial force of earthquake to
columns that regarding to stiffness difference of columns, more parts of these forces reach to short columns of floor that in the case of lack of suitable designing, severely damage, when earthquake occurs. (Figure 2). The important point in this structure is the height difference between two parts of dublex structure, that causes outstanding changes in period and stiffness and distribution of earthquake force and loading of seismic of structure. According to studies and researches, it has been recognized that shearing force in column (short columns) that connects two dublex structures increase 1.5 to 2.5 relative to shear force in two same column in ordinary structures [1].

With respect to this subject, many researchers investigated in this field, we can refer to Moretti and Tassios [2,3] that test 8 specimens of RC short columns under fixed axial load and cyclic static displacement they measured steel and concrete strain results of seismic designing with low sheer ratio and seismic behavior of short column. They have measured and surveyed and suggested one truss model for stimulating of short columns of failure mechanism and with distribution of forces in columns. Also experiment studies on nonlinear behavior of different specimens of short column with decreasing or increasing of stirrup of a when reinforced with CFRP and GFRP panles (Carbon alyaf), under the effects of laterp cyclic displacement and wind force fixed. According to loading changes, and ductility by researchers such as Colomb et al [4], Promis al [5], Galal al [6], Ghobarah and Galal [7], Ye al [8] and Galal and Ghobarah [9] has been done. Bakhshi and Tabeshpor [10] analyse nonlinear dynamic, the effect of middle plate and phenomenon of short column with the help of IDARC, Soft ware and with tabes earthquake with maximum acceleration of 0.35g. Abbasnia and barghi [11] surveying many kind of destruction of columns effect cyclic period parameters that physically have effects on kind of destruction have recognized and using experimental information and loading results on some specimen, introduced new models for predicting of column. Kheyroddin and Mirnezami [1] by analyzing three 5, 10 and 15 story metal building seismic parameters such as period changes. Displacement and also formation of short column and factor of destruct have surveyed and suggested. Method for static loading equivalent dublex buildings. Surveying nonlinear behavior of more than 30 model of steel dublex structure in 6 different detail and
comparing them in different condition including change, Kheyroddin and Mirnezami [12] suggested dublex floor level difference, effects if bending connects strength thening of web and flange hardening bond foil. The most confining and stiffness plate suitable method and detail of frames with floor level which have phenomenon of short column

2. RESEARCH SIGNIFICANCE AND METHOD

In this research, seismic behavior of short column in 3 dublex structures have been surveyed that have height level difference 1.6 meter. Plan of all 3 investigated supposed to be equal and have variable height and include 4,8 and 10 story structures. Dimension of structures plan are 19.8 ×14.8 which have five 4.95 meter bay in X direction and four 3.1 meter bay and one 2.4 meter bay in Y direction. Because this plan is practical, dimensions and bay are real and structures have been recognized symmetric. Lateral load resisting system in all structures according to Iranian Code of Practice for Seismic Resistant Design of Buildings [13] in respect of ductility have been used medium concrete flexural frame and for gravitational loading subject of national regulation. Since dublex structures is measured (counted) on irregular height, seismic loading has been done of equivalent static and spectra dynamic. Column and beam dimensions in 4 stories structures in 1st 2nd and 2 last floors are 45*45,40*40 and 35*35 cm respectively and beams dimensions 30*40 cm in both 2 floors have been brigade. In 8 story structures, columns at two 1st floor 40*40cm and 2 last floor 35*35 cm and for beams 45*50, 45*45, 35*40 and 30*35cm brigade respectively. In 10 storey structures, like 8 storey structures expect in 2 first floor, Column dimension is 55*55 and beams dimension is 50*55.

Damages on structural elements occurs. One progressive process. That causes its failure. This trend include damage stage in small scales, arising gathering damage in medium scales includes increasing of crack and their expending and damage at large scale that structure collapse. To survey actual behavior of structure when earthquake accrue, it is necessary that structure analyse under one nonlinear, analyse. For this IDARC v6.0 [14] nonlinear program has been used that been used
for nonlinear analyses of reinforced concrete structures and has the capability to make hysteresis cycle frame geometrician characteristics intersection reinforced concrete and its damage index is park-ang-wen. One of capabilities IDARC nonlinear software is modeling and indicating the structure behavior at one time step during earthquake to structures. In this research at first damage rate in short column on external frame of 4, 8 and 10 storys structures under 0.3g, 0.5g and 0.7g PGAs are surveyed and compared then choosing three elements of external frame of structures which include the last short column, medium and first column the following results are surveyed and compared:

- Time history of displacement answer of last short column, medium and first column.
- Time history of shear force in medium and first short column.
- Damage index at the top, down of medium and first short column.

Because Elsentor1940 earthquake known as international earthquake by researchers and has been many in designing and rehabilitation of structures all over the world and approximately has complete. Frequency content, intensity time and frequency contain. In this research it has been used for dynamic analysis. Elsentro earthquake in 1940 with maximum speed 0.319g and duration of its story shaking is 30 seconds and have relative long and irregular vibration that one features of earthquake with medium depth and rock bed [15].
2.1. Results of short column behavior under Elsentro Earthquake

Surveying of damage rate of short column in structures stories survey and comparison of from diagram indicates that in all structures increase of PGA the average damage rate in short column indifferent storey increase, Except in 8 floor structure in 0.3g damage in last short column has the most amount, this is because of frequency content in of Elsentro earthquake. Seismic Degree Damage of short column in floor building in all structures increase of structures height especially in upper storys damage index of short column has been increased. In 8 and 10 story structures, failure in short column by 4 and 6 storys are 0 and without failure (Figures 6 and 7).
2.2. Surveying Increasing of Figures Percent of Short Column in Different PGA
Since increasing PGA failure in short column has increased, comparing diagrams of Figures 9 indicates that average failure of short column in 0.7g, 0.5g relative to 0.3g in 1 storey structures has the most and in 8 storey structures has the lowest failure in short column.

2.3. Investigation of Short Column Influence in Structural Failure
Investigation of short column share in structures failure comparing diagrams of Figures 10 deducts that short column of short column of 4 storey in 0.3 g and 0.7 have more influence in total structures for example in 0.3g in 4,8 and 10 structures 21, 19 and 12 percent if total structures failure related to short column failure in other word in 0.3g and 0.7g increasing the structures height short column from total structures failure will be decreased and in 0.5g short of short column failure in 10 story structures is more.

2.4. Investigation History of Last Medium and First Short Column Displacement in 4 Story Structure
By surveying compression of following answer history conclude that for average
displacement history of last short column in 4, 8 and 10 story structures is more than first short column in all structures by increasing PGA. Time history of short column displacement increase except in 3 cases that its 8 and 10 story structures is approximately 25 to 30 percent and also in 4 story structures the first short column in 0.3g and 0.5g has the more displacement history then 0.7g.

Figure 11. Displacement answer history of first short column at 4 story structure

Figure 12. Displacement answer history of first short column at 4 story structure

Figure 13. Displacement answer history of first short column at 4 story structure
Paying attention to following Figures it can be concluded that displacement time history of first and medium short column in 4 story structures and last short column in 10 story. Structures is high relative to other structures. For example displacement history of last short column in 10 story structures increasing is 26 and 56 percent more than 4 and 8 story structures.

Figure 14. Displacement answer history increase of first short column at 4 story structure relation 8 and 10 story structures

Figure 15. Displacement answer history increase of mid short column at 4 story structure relation 8 and 10 story structures

Figure 16. Displacement answer history increase of last short column at 10 story structure relation 4 and 8 story structures
2.5. Investigation of Shear Force History of Last, Medium and First Short Column in 4 Story Structure

Structures by Surveying and comparing of diagrams related to history of shearing it can be concluded that the average of shear force history in first short column in 4 story structures and medium short column in 8 story structures and last short column in 10 story structures has the most mount than other column also by this conclusion we can find the exceptional cases in 4story column and last short column in 8 story structures and last short column in 10 story structure in .5 g relative to 0.7g.

Figure 17. Shear force answer history of first short column at 4 story structure

Figure 18. shear force answer history of mid short column at 4 story structure

Figure 19. Shear force answer history of last short column at 4 story structure
Pay attention following Figures it can be concluded that shear force time history in first short column in 4 story structures and medium short column in 8 story column and last short column in 10 story structures has the most among than other structure. For example in 1 story structures, the average shear force response in last short column is about 52 and 80 percent more than 4 and 8 story structure. The important point is that in last and medium short column in 0.5 g is more than 0.7 g that this amount is about 35 to 50 percent.

Figure 20. Shear force answer history increase of first short column at 4 story structure relation 8 and 10 story structures

Figure 21. Shear force answer history increase of mid short column at 8 story structure relation 4 and 10 story structures

Figure 22. Shear force answer history increase of last short column at 10 story structure relation 4 and 8 story structures
2.6. Damage Index in Up and Down of Last, Medium and First Story Structures Of Structures

Comparing Damage index of following Figures concluded that the most damages in 8 and 10 story structures is related to last story structures, especially in its part. Because the existence of force flagelliform And lack of suitable distribution of earthquake force in height. In medium short column by increasing of height and story of structures Damage at up and down of column has decreased. As in 0.3 g and 0.5g medium short column has the least Damage and even without Damage and failure rate in its up and down. In approximately equal. In first short column up and down part of 4 story structures in all PGA. Failure has been made but in 8 and 10 story structures failure in up and down part, failure is seem only at up for PGA as in 8 and 10 story structures in 0.3g and 0.5 of first short column in up and down part is lacke failure the first short column in 8 and 10 story structures in down part.

Figure 23. Damage index at up and down of 4 story structure short column

Figure 24. Damage index at up and down of 8 story structure short column
3. CONCLUSION

1. Damage rate in short column in different story increase by height and PGA increase except in 8 story structures that 0.3g of last short column tolerate the damage 0.5 and 0.7 g.
2. Short column 4 story structures in 0.3 g and 0.7g and short columns of 10 story structures in 0.5 has the most share in total structures failure.
3. The average history of first and medium short column displacement response in 4 story structures and last short column in 10 story structures has the most amount that other structures.
4. The average history of first short column shearing 4 story structures and medium short column in 8 story structures and last short column in 10 story structures has the most amount relative other structures in 8 and 10 story structures. The shearing force of last and medium short column in 0.5 g in more than 0.7 g.
5. The part of last short column and down part of first short column in 8 and 10 story structures has more damage and expriment damage at up and down of medium short column in all structures has the least amount and is experimentally equal and is less than 1st and last column totally under Elsentro earthquake to upper part of short column in 4 story structure more damage is inserted.

REFERENCES

NONLINEAR ANALYSIS OF REINFORCED CONCRETE ELEMENTS

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ABSTRACT
Introducing a new constitutive model a smeared rotating crack model was developed having the capability of predicting the entire load-deformation response of reinforced concrete elements. The important features of the model are considering the effect of reinforcement ratio on average stress-strain relationships of cracked concrete and considering the gradual reduction of average stiffness of steel bars embedded in concrete. The model applied to predict the response of available test panels and showed a good correlation. Using the simplified version of the model a new expression for shear capacity of reinforced concrete beams without stirrups was derived. The proposed relation captures the dependence of shear strength on size, shear span-to-depth ratio, longitudinal reinforcement ratio, maximum aggregate size, and concrete strength. The model agrees well with fracture mechanics concepts. The proposed relation was calibrated by least-square fitting of the existing experimental beam test database and showed a good agreement.

Keywords: reinforced concrete, shear strength, membrane elements, beams, size effect

1. INTRODUCTION
Shear behavior of reinforced concrete has been of interest to many researchers for several decades. Although several theories and design procedures for structural concrete subjected to shear have been proposed [1,2], ACI Code [3] has not modified its old formulas, yet. Of the many different theories and expressions about shear strength of RC beams that have been developed up to now, only a few consider the size effect. Among them the procedures based on modified compression field theory, MCFT, have gained some acceptance. This theory is the product of much experimental and analytical research conducted at the University of Toronto toward formulating the response of reinforced concrete elements subjected to the in-plane forces [4]. Since the development of MCFT several approaches based on it with different degrees of complexity has evolved. The simplest approach is the general shear design method [5] adopted in the AASHTO [6] and Canadian Code [7], and the most complicated are ones using MCFT in a finite element approach [8].
In the first part of this study, the MCFT is revised in a fundamental level. The literature shows that for panels containing less than 0.1% reinforcement in one direction or panels that were uniaxially reinforced the accuracy of MCFT deteriorates. This deficiency of MCFT arises from its simple constitutive models. Presented in this paper is a set of stress-strain relations for normal strength concrete and mild steel bars embedded in concrete having two salient features: 1) considering the effect of reinforcement ratio on average stress-strain relationships of cracked concrete; and 2) considering the gradual reduction of average stiffness of steel bars embedded in concrete. Incorporating equilibrium, compatibility, and the constitutive laws into a nonlinear analysis procedure, a new smeared rotating crack model was developed having the capability of predicting the entire load-deformation response of reinforced concrete elements. Corroboration study using experimental data from test panels showed that the model has a good accuracy in predicting the behavior of reinforced concrete panels throughout the loading history.

In the second part of the study, using simplified concepts of MCFT and the proposed model a new relation for predicting the shear behavior of reinforced concrete beams was obtained. MCFT based equations for shear take a simple account of crack opening and aggregate interlock across the dominant shear crack, which results in size effect on shear capacity of concrete members. According to simplified expressions derived from MCFT, shear strength for very large sizes is inversely proportional to the beam size [9,10]. However, the limit solution of shear failure load for very large sizes, obtained based on fracture mechanics, reports an exponent of $-1/2$ for beam size [11-13]. In this study, considering more refined assumptions about the opening of the dominant shear crack of the beam, new expressions for shear strength of RC beams were developed that display size effect in accordance with fracture mechanics for large sizes, asymptotically. The derived equations were compared to the other equations and were applied to existing database of ACI-445F [14] and resulted in a relatively low coefficient of variation.

2. NONLINEAR ANALYSIS OF MEMBRANE ELEMENTS

2.1. Average Stress–Strain Relationship of Steel

The basic information needed for analyzing a reinforced concrete membrane element is stress-strain relations describing the average characteristics of materials subjected to loading. As the approach of this paper is based on smeared crack concept and average stresses and strains are used, the bond slipping along the bars and shear sliding along cracks are implicitly included and we only need average constitutive laws for steel and concrete.

Steel reinforcement is generally assumed transmit axial force only and hence a uniaxial stress-strain relation is adopted. MCFT uses the usual bilinear uniaxial stress-strain relationship shown in Figure 1(a) for modeling the average behavior of steel bars embedded in concrete [4,15,16]. However, because of disturbed stress field in the cracked reinforced concrete the behavior of an embedded steel bar in concrete is different from that of a bare bar. After the occurrence of the first yielding of reinforcement at a crack location, the average stress-strain curve of
steel bars embedded in concrete shows a gradual reduction in stiffness and it continues until its complete yielding. Therefore, in the present study, a trilinear piecewise stress-strain relationship is adopted [Figure 1(b)]. The initial part of the proposed curve is a straight line with a slope of $E_s$, the modulus of elasticity of steel, up to $0.8 f_{\text{yield}}$. Then it changes to a slope of $E_s/20$ and reaches the peak at $f_{\text{yield}}$ where remains constant until failure. The details of this curve have been obtained based on an extensive data fitting on the test panel results introduced later in this paper. To maintain the simplicity of the model it is assumed that the average stress-strain relation of steel is independent of concrete specifications. The model allows for elastic unloading [Figure 1(b)].

2.2. Average Compressive Stress-Strain Relationship of Concrete

It is often recognized that cracked concrete in compression subjected to transverse tensile strains has lower strength and stiffness than uniaxially compressed concrete. This phenomenon called compression softening is usually quantified by incorporating a softening coefficient into a basic stress-strain curve [4,15,17]. Our suggested basic compressive stress-strain curve consists of Hognestad curve [18] for ascending portion and another second-degree parabola for descending portion. This part is specified by $\varepsilon_f$ which is the strain when the stress has fallen to zero (Figure 2). This strain derived based on Kent and Park [19] research is:

![Figure 2. The proposed stress-strain relation-ship for cracked concrete in compression](image-url)
where $f'_c$ is the concrete uniaxial compressive strength in MPa and $\varepsilon'_c$ is the strain in concrete cylinder at the peak stress $f'_c$ ($\varepsilon'_c$ is a negative quantity). The suggested curve for falling branch reflects the phenomenon that a low strength concrete has a low-slope descending portion. It must be noted that the proposed model is calibrated for reinforced concrete specimens 890 mm square × 70 mm thick. To use this model in a FEM program with different mesh sizes, appropriate size effect factors must be incorporated into the constitutive laws.

Studying the available panel test results, a definite effect of reinforcement ratio on average stress-strain relationship of cracked concrete in compression was recognized. To capture this phenomenon two modification factors of $\alpha$ and $\mu$ are introduced as follow for adjusting the values of $f'_c$ and $\varepsilon'_c$, respectively:

\[
f'_c^* = \alpha f'_c, \quad \alpha = 1 + 0.03(100 \rho_x)^2 (100 \rho_y)^2 \tag{2}
\]

\[
\varepsilon'_c^* = \mu \varepsilon'_c, \quad \mu = 1 + 0.04(100 \rho_x)^2 (100 \rho_y)^2 \tag{3}
\]

where $\rho_x$ and $\rho_y$ are the reinforcement ratios of the orthogonally reinforced concrete panel in $x$ and $y$ directions, respectively. For describing the softening effect, the basic compression curve, defined above, is modified in term of the peak stress attainable. By adopting the modification factor introduced by Vecchio and Collins [4]:

\[
\beta = \frac{1}{0.8 - 0.34 \varepsilon'_{c1} / \varepsilon'_c} \leq 1 \tag{4}
\]

the suggested compression curve will be obtained as (Figure 2):

\[
f'_{c2} = \beta f'_{c}\left[\left(\frac{\varepsilon'_{c2} - \varepsilon''_{c2}}{\varepsilon'_c}\right) - 1\right] \frac{\varepsilon'_{c2}}{\varepsilon'_c} \leq 1 \tag{5a}
\]

\[
f'_{c2} = \beta f'_{c}\left[\left(\frac{\varepsilon'_{c2} - \varepsilon''_{c2}}{\varepsilon'_f - \varepsilon'_c}\right) - 1\right] \frac{\varepsilon'_{c2}}{\varepsilon'_c} > 1 \tag{5b}
\]

where $f'_{c2}$ and $\varepsilon'_{c2}$ are the average principal compressive stress and strain in the cracked concrete, respectively and $\varepsilon'_{c1}$ is the co-existing principal tensile strain.
2.3. Average Tensile Stress-Strain Relationship of Concrete

The stiffening effect of post-cracking tensile stresses in the concrete between cracks has been recognized for quite some time. Neglecting this contribution of concrete called tension stiffening can cause a significant overestimation of post-cracking deformation in reinforced concrete structures [20]. The gradual reduction in stiffness due to progressive cracking is referred to as strain softening. These phenomena have been quantified by some constitutive laws obtained from reinforced concrete panel tests [4,16,21-23].

The tensile tests on reinforced concrete panels have revealed that there are three rather distinct areas in the average tensile stress-strain response of cracked concrete [24]: a) a linearly ascending portion before cracking; b) a fluctuating portion, called crack formation phase, where must cracks form; and c) a descending portion with a stable crack pattern, regarding this behavior, the conceptual model proposed for average tensile stress-strain relationship of cracked concrete is shown in Figure 3. According to this Figure, the proposed model can be expressed as:

\[ f_{c1} = E_c \varepsilon_{c1} \quad \varepsilon_{c1} < \varepsilon_{cr} \quad (6a) \]

\[ f_{c1} = f_{cr} \quad \varepsilon_{cr} < \varepsilon_{c1} < \varepsilon'_{cr} \quad (6b) \]

\[ f_{c1} = \frac{f_{cr}}{1 + \sqrt{k \varepsilon_{c1} - \sqrt{k \varepsilon'_{cr}}}} \quad \varepsilon'_{cr} < \varepsilon_{c1} \quad (6c) \]

Figure 3. The proposed stress-strain relationship for cracked concrete in tension.

The parameters of the model are described as follows. \( f_{c1} \) is the average principal tensile stress in the cracked concrete. \( E_c \) is the elastic modulus of the concrete in tension which can be taken as \( 2f'_{c}/\varepsilon'_{c} \). \( f_{cr} \) is the first cracking strength of concrete in a reinforced panel. Examining the available panel test results showed that in addition to \( f'_{c} \) the reinforcement ratio has also an influence on \( f_{cr} \) and the following relation in MPa units was obtained:
where $\alpha'$ was interestingly obtained equal to $\alpha$ defined in Eq. (2). The $\varepsilon_{cr}'$ in Eq. (6) is the average tensile strain at which the concrete begins cracking and $\varepsilon_{cr}'$ is the corresponding value at the end of crack formation phase. A survey of aforementioned database revealed that $\varepsilon_{cr}'$ has a direct relationship with the reinforcement ratio of the concrete panel. Examining different formulas and best fitting of the database with them the following relation was obtained:

$$
\varepsilon_{cr}' = \eta \varepsilon_{cr} , \quad \eta = 1 + 6(100\rho_x)(100\rho_y)
$$

(8)

To account for the dependence of post-cracking tensile response of concrete on the reinforcement ratio, as already described, some alternative relations were examined and finally the best relation with the use of data fitting was obtained as:

$$
k = 300 + 250(100\rho_x)(100\rho_y)
$$

(9)

2.4. Validation of the Proposed Model

The problem at hand is to determine the load-deformation response of an orthogonally reinforced concrete membrane element subjected to monotonically increasing in-plane stresses shown in Figure 4. The proposed constitutive laws along with the equilibrium and compatibility equations form a system of nonlinear equations needing a numerical method to solve.

As an example of the theoretical model’s application to the analysis of reinforced concrete elements, an analysis was made of Panel PV20 tested at university of Toronto by Vecchio and Collins [4]. This specimen was tested in pure shear. For this panel the compressive strength of the concrete was 6.19 MPa. The predicted response for this element based on MCFT [4] and DSFM [16] and the experimental curve is summarized in Figure 5. Note that for this panel the proposed model provides excellent correlations with experimental results.
3. A SIMPLIFIED MODEL FOR SHEAR CAPACITY PREDICTION OF BEAMS

3.1. Size Effect in Shear According to Simplified MCFT

Simplified-MCFT explanation for size effect in shear is that the shear strength of beams not containing stirrups is a function of the shear crack width. Crack widths, in turn, increase nearly both with the tensile strain in the reinforcement and with the spacing between the cracks [10]. Larger members have more widely spaced cracks and therefore are expected to fail at lower shear stress. To maintain beam action up to the failure, nearly all of the shear force $V_c$ must be transmitted by the shear stress across the main diagonal cracks. For most of the concretes, cracking will occur along the interface between the aggregates and the cement paste. The resulted rough cracks are capable to transfer shear by aggregate interlock. Decreasing the aggregate size reduces this capacity. In the development of the MCFT, Vecchio and Collins [4] suggested that for cracks transmitting only shear stress, the limiting stress would be:

$$
V_{ci} = \frac{0.18 \sqrt{f'_{ct}}}{0.31 + 24w/(d_a + 16)} \quad (10)
$$

where $w$ is the crack width (crack opening displacement) in mm and $d_a$ is the maximum aggregate size in mm. Using the parameters identified by the MCFT, Collins and Kuchma [9] proposed the following relation for the shear capacity of members without shear reinforcement:

$$
V_c = \frac{V_c}{bd} = \frac{245}{1275 + 35(0.9d)/(d_a + 16)} \sqrt{f'_{ct}} \quad (11)
$$

with $f'_{ct}$ not to be taken greater than 70 MPa. This equation represents a strong stress singularity with power of $-1$ around the crack tip that yields a size effect on
shear strength with power of $-1$ for very large sizes. This very strong stress singularity and size effect is objectionable based on fracture mechanics. Exponent $-1/2$ is the strongest size effect possible [12].

3.2. A Simplified Model for Size Effect in Shear

The failed part of a beam, subjected to two concentrated loads, with shear span $a$, containing the dominant shear crack, is shown in Figure 6. The function that defines crack opening along its length is a complex function. This complexity is the result of many factors such as the effect of the shear stresses transmitted across the crack, the presence of longitudinal bars crossing the crack, and the size effect on the crack shape. The current procedures based on MCFT, approximate the crack opening profile by a constant value that is the average crack opening [5,9,10]. In the present study, we use a linear function for the crack opening that satisfies the essential boundaries of the crack; zero opening at the crack tip and maximum opening at the bottom. Thus, we introduce the following relation:

$$w(y) = \frac{y}{d} w_{\text{max}}$$  \hspace{1cm} (12)

![Figure 6. Left part of a beam containing the dominant shear crack](image)

where $y$ is the distance from the crack tip and $w_{\text{max}}$ is the crack opening at the level of tensile bars. On the other hand, $w_{\text{max}}$ can be considered as a function of the beam geometry and material properties calculated from the following relation [13]:

$$w_{\text{max}} = k\frac{(a/d)v_{\text{cr}}}{\rho E_s} d$$  \hspace{1cm} (13)

where $k$ is a constant factor and $E_s$ is the modulus of elasticity of longitudinal reinforcement. Combining Eqs. (10), (12), and (13), we obtain a relation for the shear transfer capacity of crack surface as a function of $y$:

$$v_{\text{cr}}(y) = \frac{0.18\sqrt{f_c'}}{0.31 + \frac{0.24kv_c}{d_s\sqrt{f_c'}} y}$$  \hspace{1cm} (14)
Where

\[
d_s = \frac{\rho E_v (d_e + 16)}{100 (a/d) \sqrt{f_c}}
\] (15)

As seen from Eq. (15), \(d_s\) is a function of the beam geometry and the materials properties. On the other hand, Eq. (14) shows that \(d_s\) has the dimension of length. Thus, we name \(d_s\) the depth scale.

To calculate the shear capacity of the beam, we consider it at the moment prior to unstable crack growth. From shear-compression interaction behavior of the concrete we know that increasing the compression stress above a certain value causes decreasing in the shear capacity of the concrete. Thus, the compression zone above the crack tip, which is subjected to high compression stresses, has a small shear capacity. Therefore, we ignore the shear contribution of this area and only rely on aggregate interlock capacity across the crack [13]. For calculating shear capacity of the beam, \(V_c\), it is sufficient to integrate the vertical component of \(v_{ci}\) over the whole crack length:

\[
V_c = \frac{V_c}{bd} = \frac{1}{d} \int_{crack} v_{ci}(y) \sin \alpha \, dl
\] (16)

\[\text{Figure 7. Free-body-diagram of the left part of the beam at shear failure.}\]

where \(\alpha\) is the crack inclination angle at the position \(y\) (Figure 7) and \(dl\) is crack length element. Using geometrical relation and substituting Eq. (14) into Eq. (16), the integral is calculated easily and we obtain:

\[
\frac{V_c}{V_0} = \frac{4v_0 d_0}{v_c d} \ln \left(1 + \frac{v_c d}{4v_0 d_0} \right)
\] (17)

where \(k'\) is a constant factor (Figure 7) and

\[
d_0 = Ad_s, \quad v_0 = B \sqrt{f_c}
\] (18)
Two constants $A$ and $B$ are related to $k$ and $k'$ by $A = 0.556 / kk'^2$ and $B = 0.581 k'$ and determined empirically from data fitting. Checking limit states of Eq. (17) shows that it yields a size independent relation for shear capacity of very small beams, in accordance with plastic solution, and for very large sizes the shear strength converges to linear elastic fracture mechanics solution, i.e. a size effect with power of $-1/2$. Hence, the asymptotic behavior of Eq. (17) is in agreement with theoretical expectations.

Employing Eq. (17) for design is complicated, as it needs iteration or a computer. Thus, it is appropriate to simplify this equation. The limits of Eq. (17) for the both very small and very large sizes, obtained from l’Hopital rule [15], yield the same results. As an approximation, we can use the asymptotic function to estimate $v_c / v_0$ for the full size range. It is interesting that we can do similar simplification twice [13] and we obtain:

$$v_c = \beta \sqrt{f_c'}, \quad \beta = \frac{B}{\sqrt{1 + d / Ad_s}}$$

(19)

This relation has the same general form of the relation obtained from fracture mechanics and verified by various test results [11, 26]. Best fitting of the ACI-445F database contained 398 data [14] by this equation, yields $A = 1.33$ and $B = 0.50$ and the coefficient of variation is $\text{cov} = 19.2\%$.

To provide additional safety margin, the curve of a design formula must be passed near the lower border of a scatter band. This is usually obtained as a 5% cut-off according to the least-square method. Thus the value of $B$ is reduced to 0.35 [13]. Figure 8 represents the mean and lowered fitting of ACI-445F database by the proposed equation, and compare it with ACI’s simple formula [3], shown with dashed line. As seen in this Figure the ACI formula may continue to be used safely within a certain range, and beyond that it is unsafe.

Figure 8. Comparison to ACI-445F database of proposed equation and ACI [3] simple shear formula
4. CONCLUSIONS
An orthotropic concrete constitutive model was developed that takes into account the interaction of reinforcement ratio on concrete behavior. Compression softening and tension stiffening were also considered in the model. Incorporating equilibrium, compatibility, and the constitutive laws into a nonlinear analysis procedure, a new smeared rotating crack model was developed having the capability of predicting the entire load-deformation response of reinforced concrete elements. Corroboration study using experimental data from test panels showed that the model has a good accuracy in predicting the behavior of reinforced concrete panels throughout the loading history. Using the simplified version of the model and incorporating MCFT concept a new expression for shear capacity of reinforced concrete beams without stirrups was obtained. Limit states of this expression agrees well with the limit solutions of shear failure load for very small and very large sizes; based on plastic and fracture mechanics solutions, respectively. The proposed relation was calibrated by optimum fitting of ACI-445F databank, which resulted in low coefficient of variation. Finally, applying a safety factor, it proposed as shear strength design formula.

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FINITE ELEMENT MODELING AND INVESTIGATION OF THE EFFECTS OF MASONRY INFILLS ON THE BEHAVIOR OF REINFORCED CONCRETE (RC) FRAMES

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ABSTRACT
In this study, a finite element model for masonry-infill of RC frames has been introduced. The micro model incorporates the bricks, bed and head joints, surrounding RC frame and the interface between frame and masonry infill. The results of the analysis have been validated by experimental data reported in the literature. A parametric study has been conducted and the effects of strength, stiffness, aspect ratio of infill panels and the effect of relative stiffness of frame to masonry panel on lateral response of the structure have been investigated. It was found that infill panels increase stiffness and ultimate strength of the frames significantly, and by increasing stiffness and strength of infill, stronger failure mechanisms are activated, and failure of masonry panels shift to failure of concrete frame. It was also observed that aspect ratio plays an important role in determining failure mechanisms not only in the infill but also in the frame.

Keywords: masonry infill, RC frame, finite element modeling, lateral response, failure mechanism.

1. INTRODUCTION
Masonry walls are widely used as infill in steel and concrete structural frames throughout the world. They are usually treated as non-structural elements, and their interaction with the bounding frame is often ignored in design. But experiences of various earthquakes and also recent researches show that masonry infills have very essential effects on the behavior of the structures. Because of high initial stiffness, infills can concentrate a greater part of the earthquake force to the infilled panels. Generally, the behavior of infilled frame is completely different from that of the bare frame because of the interaction between frame and infill [1, 2].

To identify and predict the behavior of infilled frame, several experimental tests have been carried out and a number of different analytical models have been developed. Equivalent diagonal strut is one of the most applicable methods to model infill panels that was presented by Polyakov [3] for the first time. Stafford smith [4] showed that width of the equivalent strut depends on the contact length of the frame and infill. After that Mainstone [5] proposed methods to estimate equivalent strut element characteristics. Saneinejad [6] proposed equations to evaluate the ultimate strength of infill panels. Liauw [7] developed plastic analysis
methods to predict the in-plane limit loads of steel infilled frame. Sophisticated finite element models have also been developed to analyse infilled structures. Mehrabi [8] used interface element for modeling mortar joints. In this research the effects of masonry infills on the behavior of the reinforced concrete frames has been investigated by finite element modeling in ANSYS. The recorded results of Mehrabi's experimental tests [1] have been used to validate the analyses.

2. FAILURE MECHANISMS OF MASONRY INFILL IN RC FRAMES
The main failure mechanisms in masonry infill panels are diagonal cracking, diagonal-sliding cracking, bed joint sliding and corner crushing. Shear failure of the beam and column and short column phenomenon, which will happen if infill panel is not extended to the upper beam of the frame, are the most important failure mechanisms in the frame [1, 2, 5]. Figure 1 shows different failure mechanisms of masonry infills in RC frames. The behavior of infilled frames under lateral load depends on different parameters such as panel aspect ratio, stiffness and strength of the infill and frame, material properties, vertical loads and presence of gap between frame and infill [2, 5]. In this study a two dimensional finite element model has been introduced to study the response of infilled frames under monotonically increasing lateral load.

![Figure 1. Possible failure mechanisms of masonry infills in RC frames](image)

3. EXPERIMENTAL TESTS OF MEHRABI
In 1994, Mehrabi [1] carried out some experimental tests on reinforced concrete infilled frames and investigated different parameters such as monotonic and cyclic loading and different combinations of weak and strong frame and infills. Three specimens of these tests have been used to validate our numerical model. The specimens are 5 and 7, which were masonry-infilled RC frames and Specimen 1, which was a bare frame. Specimens 1 and 5 had weak flexural concrete frame and that of Specimen 7 was strong. The details of weak and strong frames are shown in Figure 2.
Masonry infills of the specimens consisted of 19.2×9.2×9.2 cm solid masonry blocks with head and bed joints of a thickness of 10 mm. Also, infill panels had the height/length ratio of about 1/1.5 and they have completely filled the surrounding frame. These specimens were subjected to a constant vertical compressive load of 294 KN. For the infilled specimens the vertical load was distributed between the column and the beam. For the bare frame specimen the vertical load was applied completely to the columns. Characteristics of the specimens have been summarized in Table 1.

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4. THE PROPOSED NONLINEAR FINITE ELEMENT MODEL

To evaluate both global and local behavior of masonry infill, a numerical micro model considering all bricks and mortars of head and bed joints as well as reinforced concrete frame components has been used. To model bricks, 4-node 2-D plane element (PLAN42 [9]) has been used in both elastic and plastic cases. The plastic case was applied to represent the crushing of bricks at the corner of masonry infill panel. For other parts of infill panels, the bricks have been represented by elastic plane element. It helps to decrease the computation time. To model bricks, cracking has also been disregarded. The head and bed joints of the mortar have
been represented by 2-node nonlinear spring COMBIN39 [9] that is a unidirectional element with nonlinear generalized force-deflection capability. The shear performance of the mortar joints have been represented by inclined spring element with response along shear resistance surface of mortars. The tension and compression behavior of mortar joints have also been represented by straight spring element with response along normal to mortar surfaces.

The concrete frame has been represented by 4-node 2-D plane element (PLAN42) and an elastic-perfect plastic formulation (Drucker–Prager) has been adopted [9]. This Drucker–Prager model is a constitutive model for the behavior of brittle material such as concrete. Reinforcing bars have been also modeled with 2-node elastic-hardening plastic bar element LINK8 [9]. They were connected to the 4-node concrete element at the two external nodes. The contact surfaces of infill panel and bounding frame have been represented by TARGE169 and CONTA172 [9]. These elements are surface to surface contact elements which have sufficient parameters for modeling every kind of contact surfaces. A schematic representation of an infilled frame model consisting of all element types has been shown in Figure 3.

![Figure 3. Infilled frame model](image)

According to the results of material tests [1], parameters of the analyses have been estimated. The value of E modulus for mortars and bricks has been estimated based on that of the three-course masonry prisms tests and the equation existent in literature [10]. Mentioned equation depends on the ratio of brick to mortar thickness. Shear behavior of mortar joints has been evaluated based on the E modulus and mohr-coulomb formulation with parameters C (cohesion) and µ (friction coefficient) that has been obtained from direct shear tests.

The nonlinear behavior of concrete of the frame has been defined by the Drucker-Prager model and parameter C (cohesion) and Ø (friction angle). In the plane stress case, Drucker-Prager's constitutive law reduces to a single continuous yield surface, whose equation reads:
\[
\sqrt{\frac{1}{3} (\sigma_x^2 + \sigma_y^2 - \sigma_x \sigma_y) + \alpha (\sigma_x + \sigma_y) - k \leq 0}
\]  
(1)

Parameters \(k\) and \(\alpha\) are related to \(C\) and \(\varnothing\) of the considered material:

\[
\alpha = \frac{2 \sin \varphi}{\sqrt{3} (3 - \sin \varphi)}; \quad k = \frac{6c \cos \varphi}{\sqrt{3} (3 - \sin \varphi)}.
\]  
(2)

These parameters determine the yield stresses in uniaxial tension and compression \(\sigma_t\) and \(\sigma_c\):

\[
\sigma_t = \frac{k}{\sqrt{3} + \alpha}; \quad \sigma_c = \frac{k}{\sqrt{3} - \alpha}.
\]  
(3)

The complete set of analysis parameters have been described in Table 2. The specifications of the concrete frame reinforcing bars such as yield and ultimate stress that have been obtained from tension test results [1] have been shown in Table 3.

### Table 2: Specifications of numerical models

<table>
<thead>
<tr>
<th>Specimens number</th>
<th>Bricks</th>
<th>Mortar Joints</th>
<th>Frame Concrete</th>
<th>Drucker–Prager parameter</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specimens number</td>
<td>E Modulus (MPa)</td>
<td>Compressive Strength (MPa)</td>
<td>E Modulus (MPa)</td>
<td>Compressive strength (MPa)</td>
</tr>
<tr>
<td>1</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>5</td>
<td>11000</td>
<td>15.59</td>
<td>3500</td>
<td>13.38</td>
</tr>
<tr>
<td>7</td>
<td>11000</td>
<td>15.59</td>
<td>3500</td>
<td>15.52</td>
</tr>
</tbody>
</table>

### Table 3: Specifications of numerical models

<table>
<thead>
<tr>
<th>Reinforcing Bars</th>
<th>Bar Size (mm)</th>
<th>E Modulus (MPa)</th>
<th>Yield Stress (MPa)</th>
<th>Ultimate stress (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>No.2</td>
<td>6.35</td>
<td>2.1E5</td>
<td>370</td>
<td>450</td>
</tr>
<tr>
<td>No.4</td>
<td>12.7</td>
<td>2.1E5</td>
<td>410</td>
<td>660</td>
</tr>
<tr>
<td>No.5</td>
<td>15.9</td>
<td>2.1E5</td>
<td>410</td>
<td>660</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Contact surfaces of frame &amp; infill</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specimens number</td>
</tr>
<tr>
<td>------------------</td>
</tr>
<tr>
<td>5</td>
</tr>
<tr>
<td>7</td>
</tr>
</tbody>
</table>
According to the experimental specimens, infilled frames have been modeled on the 315×90×46 cm reinforced concrete base. The lateral and vertical loading have been applied to the model in the case of displacement and force control, respectively. To prevent the occurrence of tension in the concrete beam, the lateral displacement of the first and last nodes of it has been constrained. Also, the weight of different components of frame and infill panel has been ignored in modeling.

5. COMPARISON OF GLOBAL AND LOCAL BEHAVIOR WITH TESTS
Lateral displacement at the mid-height node of the top beam in the frame is compared to the test results. Also, the local behavior has been evaluated by considering the crack paths that have occurred in different places, sliding in the bed joints, crushing in the corners of the infill panel and the occurrence of the shear failure in the concrete frame component. Figure 4 shows the lateral load-lateral displacement curves obtained from the numerical models and tests for specimens 1, 5 and 7.

The Figures show that the model has been relatively successful in predicting the global behavior over small deformation range and to some extent the large deformation response. It should be noted that the experimental curves of specimens...
5 and 7 are envelopes of hysteresis curves of cyclic tests. The deformed shapes obtained from numerical analysis for specimens 5 and 7 have been presented in Figure 5. The numerical results indicate that the prevailing mechanism at the ultimate strength of specimen 5 is shear failure in the column of concrete frame. In this model, the first important event in the process of increasing lateral load is creation of crack and tearing at the common border of the frame and infill panel and in the tension corners of the infill panel. As loading continues, the cracks form in the bed joints of mortar at the mid height of the masonry panel and propagate in the direction of compression diameter. Increasing of the lateral load, eventually leads to shear failure at the upper end of the windward column. All of these agree with experimental evidence.

The simulated model of Specimen 7 indicates that after creation of crack and tearing at the common border of the frame and infill panel, the diagonal-sliding cracks appear. The prevailing mechanism at the ultimate strength for mentioned specimen is crushing at the compressive corner of the infill panel. It should be noted that no shear failure is observed in the components of the RC frame. All of these also agree with experimental evidence.

![Figure 5. Deformed shapes obtained from numerical analysis](image)

By comparing the results of the proposed model with the similar recorded experimental results and confirming the validation of the analysis, parametric studies will be carried out. Numerical models of the experimental specimens 5 and 7, have been named WF-SI (Weak frame and strong infill panel) and SF-SI (Strong frame and strong infill panel), respectively.

**6. PARAMETRIC STUDY OF MASONRY-INFILLED RC FRAMES**

In the parametric study, the effect of strength of frame and infill, dimensions and aspect ratios of panels have been investigated.

**6.1. Strength of Frame and Infill**

Different combinations of weak and strong frame and infill have been analyzed.
Specifications of the infilled frames with strong masonry infills have already been mentioned in the preceding sections. Specifications of the weak masonry infills [11] are presented in Table 4.

<table>
<thead>
<tr>
<th>Bricks</th>
<th>Mortar Joints</th>
</tr>
</thead>
<tbody>
<tr>
<td>E Modulus (MPa)</td>
<td>Compressive Strength (MPa)</td>
</tr>
<tr>
<td>5000</td>
<td>13.5</td>
</tr>
</tbody>
</table>

Various models such as SF-SI, WF-SI, SF-WI (Strong frame and weak infill panel) and WF-WI (Weak frame and weak infill panel) have been analyzed. Comparison between infilled frame and the respective bare frame is presented in Figure 6. It is seen that both stiffness and strength of the infilled frame are significantly higher than those of the bare frame, but the maximum lateral displacement of infilled frame is less than that of bare frame. The maximum lateral resistance of SF-SI is 38% higher than that of SF-WI. Also, the Figureshows that lateral strength of SF-SI and SF-WI is 82% and 42% higher than that of WF-SI and WF-WI, respectively. This indicates activation of stronger failure mechanisms when strong frame and infill is used.

![Figure 6. Lateral load-lateral displacement of weak and strong frame and infill](image)

The numerical results for SF-WI indicate that after occurrence of diagonal-sliding cracks, large slips occur along the bed joints at the ultimate strength. Shear failure in the windward column is the prevailing mechanism at the ultimate strength for WF-WI. The deformed shapes obtained from numerical analysis for SF-WI and WF-WI have been presented in Figure 7.

6.2. The Effect of Aspect Ratio

In order to investigate the mentioned parameter, three height/length ratios 1/1, 1/1.5 and 1/2 for panels, which have been named AR-1/1, AR-1/1.5 and AR-1/2,
respectively, have been analyzed. The model SF-SI of preceding section is compared to model AR-1/1.5 in this study. The models in these aspect ratios have similar height and different length. Comparison of the models has been presented in Figure 8. The results show that lateral strength of AR-1/2 is 26% and 9% higher than that of AR-1/1 and AR-1/1.5, respectively.

The numerical results for AR-1/1 indicate that after occurrence of diagonal cracks, the maximum lateral resistance is reached when crushing occur at the corners of the infill panel. For AR-1/2 the numerical results indicate that after occurrence of diagonal-sliding cracks and corner crushing in the infill panel, shear failure in the column of RC frame is the prevailing mechanism at the ultimate strength. These results show that different panel aspect ratios can activate different failure mechanisms in masonry infilled RC frames. The deformed shapes obtained from numerical analysis for AR-1/1 and AR-1/2 have been presented in Figure 7.

![Deformed shapes obtained from numerical analysis for SF-WI and WF-WI](image)

![Lateral load-lateral displacement for numerical modeling of different aspect ratios](image)
The values of the initial stiffness, secant stiffness, ultimate strength, ductility and damping for different types of mentioned infilled frame and their bare frames are summarized in Table 4.

Table 5: Stiffness, strength, ductility and damping for all analyzed models

<table>
<thead>
<tr>
<th></th>
<th>Weak Frame</th>
<th>Strong Frame</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Bare frame</td>
<td>WF-SI</td>
<td>WF-WI</td>
</tr>
<tr>
<td>Initial stiffness (KN/cm)</td>
<td>78.4</td>
<td>1698</td>
<td>1286</td>
</tr>
<tr>
<td>Secant stiffness* (KN/cm)</td>
<td>65.18</td>
<td>1231.7</td>
<td>878.5</td>
</tr>
<tr>
<td>Ultimate strength (KN)</td>
<td>80.3</td>
<td>259.2</td>
<td>233.2</td>
</tr>
<tr>
<td>Ductility</td>
<td>6.67</td>
<td>5.45</td>
<td>5.83</td>
</tr>
<tr>
<td>Damping (%)</td>
<td>19.16</td>
<td>19.37</td>
<td>19.71</td>
</tr>
</tbody>
</table>

* From the idealized force-displacement curve [12].

As can be observed from Table 5, infill panels can improve the performance of RC frames. The strong frame and strong infill panels have a better performance than other combinations of strong and weak frame and infill in terms of load resistance and energy dissipation capability.

7. CONCLUSIONS
In this study, a numerical micro-model has been introduced to investigate the behavior of masonry infills in RC frames. The model incorporates the effects of bricks, bed and head joints of mortar and takes into account nonlinear characteristics such as cracking, sliding of mortar joints and crushing of masonry.
infills and shear failure in the frame components. Validation of the analyses was done by appropriate test results.

The effects of several parameters such as strength of both frame and infills and panel aspect ratio were investigated. The study shows that stiffness and strength of the infilled frame is significantly higher than that of a bare frame. The maximum lateral resistance of a strong frame and strong infill is significantly higher than that of weak frame and infill. It was found that masonry infills can increase energy-dissipation capacity of RC frames when they are used with strong frames. Also using masonry infills with weak frames can lead to occurrence of brittle shear failure at the columns of RC frames. The results of parametric studies indicate that changing the aspect ratios of panels can not only change the ultimate strength of the infilled frames, but can also activate different failure mechanisms. For a panel with height/length ratio of 1/2, addition of infill panels can lead to brittle shear failure at the columns, whereas it does not occur in aspect ratios of 1/1 and 1/1.5.

REFERENCES
3. Polyakov, S.V., "On the interaction between masonry filler walls and enclosing frame when loaded in the plane of the wall", Translations in Earthquake Engineering, Earthquake Engineering Research Institute, Oakland, California, 1960, pp.36-42.
REINFORCED CONCRETE SLABS DESIGN BASED UPON
CONCRETE CODE OF IRAN (CCI) AND BRITISH STANDARD
(BS) PROVISIONS

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³Associate Professor
Civil Engineering Department, Shahid Bahonar University of Kerman, Kerman, Iran

ABSTRACT
One of the common methods for analyzing and designing two-way reinforced concrete slabs is the Moment Coefficients Method. In this paper, the Moment Coefficients Method for designing two-way slabs stated in the Concrete Code of Iran (CCI) is considered and compared with the same method stated in the British Standard (BS). For this purpose, the provisions recommended in the above mentioned codes are compared first, and then the differences are discussed. The effect of different provisions on determining the amount of the required steel for the slabs having different edge conditions is considered through a numerical study. In the end, the cases in which the use of a specific code gives conservative or economic results are concluded.

Keywords: two-way slab, concrete code of Iran (CCI), British standard (BS), moment coefficient method, safety factors

1. INTRODUCTION
Analysis of plates and shells to attain internal actions for designing structural elements is one of the fields that scientists and engineers have been working on for years. These attempts have led to some exact and approximate solutions. One of the approximate solutions which is used to analyse rectangular plates under uniform normal loads is the Moment Coefficients Method. This method is mostly used to analyze reinforced concrete slabs and helps one to find internal bending moments and shear forces by applying some coefficients. The amounts of these coefficients depends on slab supports conditions and the slab spans ratios. Moment Coefficients Method is a very restricted method but as it’s rather simple to use, it has been mostly applied in analysis and design of concrete slabs. Each concrete design code has its own provisions for using Moment Coefficients Method. In this paper, the manner of using this method in CCI[1] and BS[2] codes is studied and compared to clarify which code leads to a more economical or conservative design.
2. ANALYSING SLABS BY MOMENT COEFFICIENTS METHOD

The Moment Coefficients Method defines some coefficients to attain internal bending moments and shear forces. The amounts of these coefficients mainly depend on the ratio of the slab dimensions and the slab supports conditions. The larger the amount of the ratio of long span to short span the stiffer the short span becomes, and so it absorbs more energy.

The moments would be calculated by the following relation:

\[ M = C \times w \times l^2 \]  

(1)

in which, \( M \) is the maximum positive or negative internal moment of middle strip of the slab per unit length, \( C \) is the moment coefficient given by the code, \( w \) is uniformly distributed load and \( l \) is the span length.

Based upon the code provisions, \( M \) is the maximum bending moment at the middle and is reduced linearly to one-third of this value at the sides as indicated in Figure 1. Figure 1.a is plotted based on CCI and Figure 1.b is plotted based on BS provisions.

3. PARTIAL SAFETY FACTORS

The partial safety factors which are used in both codes to attain ultimate loads are defined as follows:

In CCI code:
- Ultimate dead load, “D” = 1.25 times service dead load, “d”
- Ultimate live load, “L” = 1.5 times service dead load, “l”

In BS code:
- Ultimate dead load, “D” = 1.4 times service dead load, “d”
- Ultimate live load, “L” = 1.6 times service dead load, “l”

Cited relations indicate that the ultimate loads in BS code are more conservative than the ultimate loads in CCI code.
4. COMPARING THE MOMENT COEFFICIENTS
To study the differences between the two codes, the differences of coefficients are studied first. In Figure 2 the ratio of CCI coefficients of negative moments to BS coefficients of negative moments are plotted versus the ratio of slab spans. The curves are related to the case of 4 continuous edges.

![Figure 2. Comparing coefficients of CCI and BS codes](image)

It can be seen that the value of coefficients for shorter span in CCI code is always more than in BS. But, for longer span CCI code decreases the coefficients values noticeably.

For comparing effects of loads partial safety factors, dead load is assumed constant when the live load is varying (for dead load=5.75 kN/m² and live load varying from 1 to 7 kN/m²) and the ratio of ultimate load in CCI on ultimate load in BS is plotted in Figure 3.

![Figure 3. Effect of live load on ultimate loads](image)
As the variation of the cited ratio due to change of live load is negligible, the numerical study would be just done for a constant value of live load. The amount of long span to short span ratio would be changed to clarify its effect on designing results.

5. NUMERICAL STUDY

A two-way slab having 150mm thickness is given in (Figure 4). The applied live load is supposed to be 2kN/m² and the dead load is taken as equal to 2kN/m² (the slab weight is not taken into account). This slab is designed having constant shorter span by the value of 4m when the longer span is 1.0, 1.1, 1.25, 1.42, 1.66 and 2.0 times the shorter span length, respectively.

![Figure 4. Slab view](image)

For a numerical study, both CCI and BS provisions are used and the analysis results are summarized in Table 1. Then the slab is designed upon both codes. Design results are given in Table 2 and compared in Table 3. This example would be solved by other supports condition.

The values given in Table 1 are the amounts of flexural moments calculated by using Table 15-8-2-4 of CCI and Table 3.14 of BS which give moment coefficients values for analysis. Table 1 indicates that the CCI results, when the spans ratio gets close to 2.0, are almost identical to that of one-way slab. On the other hand, based upon BS provisions, the moment coefficients for shorter span do not change when the spans ratio varies.

As the spans ratio gets closer to 2.0, BS presents a more economical design. Moreover, when the ratio gets closer to 1.0, BS would lead to a more conservative design.

In the following tables, \( M^- \) is the ultimate negative moment on slabs edges calculated by moment coefficients and given for both codes; \( M^+ \) is the ultimate positive moment at the middle of spans and given for both codes; and \( A_s \) is the amount of flexural steel rebar area used in each case. The total \( A_s \) given in Tables 3
and 6 would define the summation of whole rebar value which is used in slabs and is calculated by following relation:

$$\text{Total } A_s = (2A_s \text{ (for } M^- \text{ on } L_a) + A_s \text{ (for } M^+ \text{ on } L_a)) \cdot L_a + (2A_s \text{ (for } M^- \text{ on } L_b) + A_s \text{ (for } M^+ \text{ on } L_b)) \cdot L_b$$

In which $L_A$ is the length of shorter span and $L_B$ is the length of longer one.

### Table 1: Analysis results

<table>
<thead>
<tr>
<th>Spans ratio</th>
<th>CCI $M^-$ Parallel to Y dir (kN.m/m)</th>
<th>CCI $M^+$ Parallel to Y dir (kN.m/m)</th>
<th>CCI $M^-$ Parallel to X dir (kN.m/m)</th>
<th>CCI $M^+$ Parallel to X dir (kN.m/m)</th>
<th>BS $M^-$ Parallel to Y dir (kN.m/m)</th>
<th>BS $M^+$ Parallel to Y dir (kN.m/m)</th>
<th>BS $M^-$ Parallel to X dir (kN.m/m)</th>
<th>BS $M^+$ Parallel to X dir (kN.m/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>7.335</td>
<td>3.366</td>
<td>7.335</td>
<td>3.366</td>
<td>5.679</td>
<td>4.397</td>
<td>5.862</td>
<td>4.397</td>
</tr>
<tr>
<td>1.1</td>
<td>8.965</td>
<td>4.162</td>
<td>7.100</td>
<td>3.226</td>
<td>6.778</td>
<td>5.130</td>
<td>7.094</td>
<td>5.320</td>
</tr>
<tr>
<td>1.42</td>
<td>12.062</td>
<td>5.802</td>
<td>5.587</td>
<td>2.785</td>
<td>9.343</td>
<td>7.090</td>
<td>11.821</td>
<td>8.866</td>
</tr>
</tbody>
</table>

### Table 2: Rebar design results

<table>
<thead>
<tr>
<th>Spans ratio</th>
<th>CCI $M^-$ Parallel to Y dir (A, mm$^2$)</th>
<th>CCI $M^+$ Parallel to Y dir (A, mm$^2$)</th>
<th>CCI $M^-$ Parallel to X dir (A, mm$^2$)</th>
<th>CCI $M^+$ Parallel to X dir (A, mm$^2$)</th>
<th>BS $M^-$ Parallel to Y dir (A, mm$^2$)</th>
<th>BS $M^+$ Parallel to Y dir (A, mm$^2$)</th>
<th>BS $M^-$ Parallel to X dir (A, mm$^2$)</th>
<th>BS $M^+$ Parallel to X dir (A, mm$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>177.331</td>
<td>80.522</td>
<td>177.331</td>
<td>80.522</td>
<td>122.484</td>
<td>94.533</td>
<td>126.491</td>
<td>94.533</td>
</tr>
<tr>
<td>1.1</td>
<td>217.701</td>
<td>99.772</td>
<td>171.547</td>
<td>77.141</td>
<td>146.581</td>
<td>110.483</td>
<td>153.513</td>
<td>114.640</td>
</tr>
<tr>
<td>1.25</td>
<td>258.442</td>
<td>119.104</td>
<td>159.783</td>
<td>77.761</td>
<td>174.862</td>
<td>132.508</td>
<td>199.248</td>
<td>148.595</td>
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<td>1.42</td>
<td>295.436</td>
<td>139.693</td>
<td>134.450</td>
<td>66.514</td>
<td>203.326</td>
<td>153.433</td>
<td>258.852</td>
<td>192.706</td>
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<tr>
<td>1.66</td>
<td>324.427</td>
<td>161.555</td>
<td>107.768</td>
<td>52.312</td>
<td>227.256</td>
<td>169.191</td>
<td>357.723</td>
<td>265.508</td>
</tr>
</tbody>
</table>

### Table 3: Design comparison

<table>
<thead>
<tr>
<th>Spans ratio</th>
<th>CCI/BS Total $A_s$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>0.643</td>
</tr>
<tr>
<td>1.1</td>
<td>0.730</td>
</tr>
<tr>
<td>1.25</td>
<td>0.825</td>
</tr>
<tr>
<td>1.42</td>
<td>0.927</td>
</tr>
<tr>
<td>1.66</td>
<td>1.079</td>
</tr>
<tr>
<td>2.0</td>
<td>1.250</td>
</tr>
</tbody>
</table>
To attain more reliable conclusions, the boundary conditions of the slab are changed. In the new conditions, only the longer spans are continuous. The analysis results are summarized in Table 4. The designing results and comparisons' ratios are given in Tables 5 and 6, respectively.

The same as previous example, by using CCI provisions, obtained amounts show that the results would be close to the condition of one-way slab, when the spans ratio gets closer to 2.0. On the other hand, BS presents constant amount for shorter edge of span which means that shorter edge absorbs a greater amount of energy. As the spans ratio gets close to 2.0, BS presents a more economical design. However, when the ratio gets closer to 1.0, BS would lead to a more conservative design.

Figure 5, indicates that the values of used steel amount ratio obey a uniform configuration when spans ratio increases.

### Table 4: Analysis results

<table>
<thead>
<tr>
<th>Spans ratio</th>
<th>CCI M⁻ Parallel to Y dir (kN.m/m)</th>
<th>CCI M⁺ Parallel to Y dir (kN.m/m)</th>
<th>CCI M⁻ Parallel to X dir (kN.m/m)</th>
<th>CCI M⁺ Parallel to X dir (kN.m/m)</th>
<th>BS M⁻ Parallel to Y dir (kN.m/m)</th>
<th>BS M⁺ Parallel to Y dir (kN.m/m)</th>
<th>BS M⁻ Parallel to X dir (kN.m/m)</th>
<th>BS M⁺ Parallel to X dir (kN.m/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>12.225</td>
<td>4.641</td>
<td>3.366</td>
<td>8.427</td>
<td>6.229</td>
<td>0.000</td>
<td>6.229</td>
<td>0.000</td>
</tr>
<tr>
<td>1.1</td>
<td>13.040</td>
<td>5.111</td>
<td>3.029</td>
<td>9.160</td>
<td>6.962</td>
<td>0.000</td>
<td>7.537</td>
<td>0.000</td>
</tr>
<tr>
<td>1.25</td>
<td>13.692</td>
<td>5.792</td>
<td>2.817</td>
<td>10.168</td>
<td>7.603</td>
<td>0.000</td>
<td>9.733</td>
<td>0.000</td>
</tr>
<tr>
<td>1.42</td>
<td>14.018</td>
<td>6.473</td>
<td>2.224</td>
<td>11.175</td>
<td>8.427</td>
<td>0.000</td>
<td>12.560</td>
<td>0.000</td>
</tr>
<tr>
<td>1.66</td>
<td>14.344</td>
<td>7.087</td>
<td>1.877</td>
<td>11.816</td>
<td>8.885</td>
<td>0.000</td>
<td>17.164</td>
<td>0.000</td>
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<tr>
<td>2.0</td>
<td>14.670</td>
<td>7.701</td>
<td>1.228</td>
<td>12.824</td>
<td>9.710</td>
<td>0.000</td>
<td>24.915</td>
<td>0.000</td>
</tr>
</tbody>
</table>

### Table 5: Rebar design results

<table>
<thead>
<tr>
<th>Spans ratio</th>
<th>CCI M⁻ Parallel to Y dir (Aₘₑ)</th>
<th>CCI M⁺ Parallel to Y dir (Aₘₑ)</th>
<th>CCI M⁻ Parallel to X dir (Aₘₑ)</th>
<th>CCI M⁺ Parallel to X dir (Aₘₑ)</th>
<th>BS M⁻ Parallel to Y dir (Aₘₑ)</th>
<th>BS M⁺ Parallel to Y dir (Aₘₑ)</th>
<th>BS M⁻ Parallel to X dir (Aₘₑ)</th>
<th>BS M⁺ Parallel to X dir (Aₘₑ)</th>
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<td>111.396</td>
<td>80.522</td>
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<td>134.516</td>
<td>0.000</td>
<td>134.516</td>
<td>0.000</td>
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<tr>
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<td>122.830</td>
<td>72.387</td>
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<td>150.610</td>
<td>0.000</td>
<td>163.285</td>
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<tr>
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<td>67.297</td>
<td>221.722</td>
<td>164.741</td>
<td>0.000</td>
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<tr>
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<td>53.047</td>
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<td>0.000</td>
<td>275.548</td>
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<tr>
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<td>171.221</td>
<td>44.718</td>
<td>258.750</td>
<td>193.139</td>
<td>0.000</td>
<td>381.093</td>
<td>0.000</td>
</tr>
<tr>
<td>2.0</td>
<td>361.989</td>
<td>186.364</td>
<td>29.214</td>
<td>281.535</td>
<td>211.492</td>
<td>0.000</td>
<td>565.041</td>
<td>0.000</td>
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</table>
Table 6: Design comparison

<table>
<thead>
<tr>
<th>Spans ratio</th>
<th>CCI/BS (Total steel reinforcement)</th>
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<tr>
<td>1.0</td>
<td>0.710</td>
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<tr>
<td>1.1</td>
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<td>2.0</td>
<td>1.175</td>
</tr>
</tbody>
</table>

Figure 5: Variation of used steel amount ratio

By the spans ratio of about 1.6, it seems that the ratio of CCI/BS total steel reinforcement would be more than one and indicated that BS provisions lead to a more economical design.

6. CONCLUSION

In this paper, the Moment Coefficients Method for designing two-way slabs stated in the Concrete Code of Iran (CCI) is considered and compared with the same method stated in the British Standard (BS).

As the numerical study shows, by changing the value of spans ratio in a two-way slab, the codes give a different result for the amount of steel flexural rebar. For the spans ratio about 1.5-1.6 the codes lead to almost the same amount of steel rebar. For spans ratio more or less than 1.5-1.6 the difference in steel amount increases as it is plotted in Figure 5.

Obtained amounts show that by using CCI provisions, the results would be close to the results of one-way slab, when the spans ratio gets close to 2.0. On the other hand, BS presents constant amount for shorter edge of slab which means that
shorter edge absorbs a greater amount of energy. As the spans ratio gets close to 2.0, BS presents a more economical design. However, when the ratio gets closer to 1.0, BS would lead to a more conservative design. The maximum average variation occurs in span ratio of 1.0, for which CCI gives steel amount about 67 percent of the steel amount of BS. Therefore, it is very important to clarify the fundamental differences between two codes, to show which design is really safer or more economical. To define which code gives better provisions for slabs design the constructions and economical conditions must be considered. As CCI provisions give more economical results for spans ratio between 1.0 and 1.6, and more compatible with construction and economical conditions of Iran, would be more reliable to use.

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CLOSED-FORM SOLUTION OF SEMI-SPHERE CONCRETE SHELL UNDER LATERAL LOAD

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ABSTRACT
Closed-form solution is one of the exact and reliable approaches in analysis of structures. Here, a semi-sphere shell under lateral load (equal to earthquake load), based on theory of shells and plates, has been studied. Internal forces are present in closed-form. In this study, materials are assumed to be isotropic, homogenous and elastic and the shell acts as a membrane. The outcome of this analysis can be used in studying the behavior of revolutionary shells under lateral load.

Keywords: shells of revolution, membrane, earthquake, closed-form solution, dome

1. INTRODUCTION
In this article, the behavior of a spherical shell under lateral load is studied. Emphasis is placed on dome structures. Domes have positive Gaussian curvature. These types of shells are used to cover the roofs of sports halls and large liquid tanks. The containment shield structures of nuclear power plants also have dome-like roofs. Various pressure vessels are either completely composed of a single rotational shell or have shells of rotation as their end caps.
Firstly, the governing membrane equations of shells of revolution will be derived. Then, we shall apply the analysis of a semi sphere membrane shell under seismic equivalent load.

2. GEOMETRICAL DESCRIPTION
We can define two principal radii of curvature at any point on the middle surface of a shell with non-zero Gaussian curvature. Figure 1 shows two principal sections containing the normal shell at point P. These sections create two plane curves with two local principal radii of curvature, $r_1$ and $r_2$, as shown in Figure 1. One of these sections is called meridional curve while the projection of another section on a plane perpendicular to the axis of revolution creates the parallel circles on the shell surface.
The middle surface of a shell of revolution with non-zero positive Gaussian quadrature can be described rather like the earth. Thus, through any point we may take two sections, one perpendicular to the axis of revolution, and the other containing the axis. The first cuts the shell in a parallel curve (circle of latitude) and the other in a meridian (plane of longitude). At any point, the radius of
curvature of the meridian is called $r_1$, and the radius of parallel circle, $r$, is the projected value of another principle radius of curvature which has been denoted by $r_2$.[2]

![Figure 1. A partial perspective view of a surface of revolution showing the principal sections at a point P, the principal radii of curvature, the meridians and the parallel circles.](image)

Parallel circles form the perimeter of the base of a cone, the apex of which is the center of curvature for $r_2$. Due to rotational symmetry, the center of curvature of $r_2$ always lies on the axis of revolution. However, the center of curvature of $r_1$ does not have to lie on the axis.

The angle between the normal to the surface at P with the axis of revolution has been denoted by $\phi$. We have also denoted the horizontal angular position of P, from some arbitrary origin, by the angle $\theta$. The direction of the axis of revolution is assumed to coincide with the z axis. [2]

![Figure 2. A meridional section of rotational showing the geometrical parameters of shell surface.](image)
Referring to Figure 2, the radius of parallel circle, \( r \), at point \( P \) can be written as

\[
\nu = r \sin(\varphi)
\]

Also referring again to Figure 2, the following relations exist among the shell geometrical parameters:[2]

\[
\begin{align*}
 ds &= r_1 \, d\varphi \\
 dr &= ds \cos(\varphi) \\
 dz &= ds \sin(\varphi) \\
 \frac{dr}{d\varphi} &= r_1 \cos(\varphi) \\
 \frac{dz}{d\varphi} &= r_1 \sin(\varphi)
\end{align*}
\]  

(1) \hspace{2cm} (2) \hspace{2cm} (3)

Combining the above relations, we obtain the following inter-relation between the surface parameters \( r_1 \), \( r_2 \), and \( \varphi \).

\[
\frac{1}{r} \frac{dr}{d\varphi} = \frac{r_1}{r_2} \cot(\varphi)
\]

(4)

3. GOVERNING MEMBRANE EQUATION

To derive the membrane equilibrium equations for shells of revolution, we consider the free body diagram of an element of the shell in Figure 3. The element shown in Figure 3 is taken out from the shell by two pairs of infinitesimally adjacent sections. The first pair of sections is meridians while the second pair contains the normal at the corner points. Since these two intersections are principal sections, they are mutually orthogonal to each other.

Figure 3. An infinitesimal element of a rotational surface
The free body diagram of Figure 4 shows the internal membrane forces, $N_\theta, N_\phi, N_\delta$, and their differential variations, $N_{\theta\phi}$, designates the meridional force, $N_\theta$, the hoop force, and $N_\delta$, the membrane shear force; the quantities $P_r, P_\theta, P_\phi$, represent the intensity of external distributed applied loading, in the $r, \theta, \phi$ directions, respectively. [3]

![Figure 4. Free body diagram of a rotational shell element](image)

We write the equations of equilibrium in the $r, \theta, \phi$ directions. Because of the double curvature, the membrane forces have projections in all three directions and thus contribute to all three equilibrium equations. Figure 5, shows the contributions of $N_\theta$ and $N_\delta$ in various directions. [3]

The equilibrium equation in the meridional direction is:

$$\frac{\partial N_{\theta\phi}}{\partial \theta} + r_1 \frac{d}{d\phi} \left( \frac{d(rN_\delta)}{d\theta} \right) - N_\theta - r_1 \frac{d}{d\phi} \left( \frac{dN_\delta}{d\theta} \right) \cos \phi + P_\phi r_1 \frac{d}{d\phi} \left( \frac{dN_\delta}{d\theta} \right) = 0 \tag{5}$$

If we divide both sides of this equation by $d\theta \, d\phi$, we obtain:[3]

$$\frac{\partial (rN_\delta)}{\partial \theta} + r_1 \frac{\partial N_\delta}{\partial \phi} - r_2 N_\theta \cos \phi + P_\phi r_1 = 0 \tag{6}$$

We derive the equilibrium equation in the hoop direction in a similar fashion:[3]

$$\frac{\partial (rN_\phi)}{\partial \phi} + r_1 \frac{\partial N_\phi}{\partial \theta} + r_2 N_{\theta\phi} \cos \phi + P_\phi r_2 = 0 \tag{7}$$

The third equilibrium equation is obtained by projecting all the forces in the direction normal to the shell, i.e., in the $r$ direction. By doing so, we obtain:[3]
\[ N_g \tau_1 \sin \phi + N_\theta (r - \tau_2 r_2) = 0 \]

Which, upon division by \((r \, r_2)\) yields:[3]

\[ \frac{N_g}{\tau_1} + \frac{N_\theta}{\tau_2} = P_\tau \]  

(8)

Equations 6, 7 and 8 constitute the governing equilibrium equations of the membrane theory for shells of revolution. These relations yield \(N_g, N_\theta, N_{\theta\theta}\), i.e. the membrane force field in the shell.

Note that the meridional and hoop forces \(N_g, N_\theta\) appear in all three equations. This indicates that a doubly curved shell is a complex and efficient structure; all three forces \(N_g, N_\theta, N_{\theta\theta}\) contribute to carrying the load in any direction. The spatial interaction of internal forces, manifested in their presence in all equilibrium equations, is indicative of an efficient and profound behavior of doubly curved shells. This spatial collaboration is very rare in framed structures.

4. SHELLS OF REVOLUTION WITH NONAXISYMMETRIC LOADING

Shells structures can be subjected to loadings which are not axisymmetric. Examples of nonaxisymmetric loadings are: wind forces, earthquake effects, soil pressure on buried pipes, and temperature gradients in composite and/or metallic shells.

To perform a membrane analysis of rotationally symmetric shells under arbitrary loading, we must use all three coupled simultaneous partial differential equations (6, 7, 8). If we eliminate \(N_\theta\) from these equations, we obtain the following relations:[1]

\[ \tau_2 \frac{\partial N_g}{\partial \phi} \sin \phi + (\tau_1 + \tau_2) N_\theta \cos \phi + r_1 \frac{\partial N_{\theta\theta}}{\partial \theta} = -r_2 \tau_2 \left( P_\theta \sin \phi + P_\tau \cos \phi \right) \]  

(9a)

\[ \tau_2 \frac{\partial N_{\theta\theta}}{\partial \theta} \sin \phi + 2r_2 N_{\theta\theta} \cos \phi - \tau_2 \frac{\partial N_g}{\partial \phi} = -r_1 \tau_2 \left( P_\theta \sin \phi + \frac{\partial P_\theta}{\partial \theta} \right) \]  

(9b)

For a distributed loading we can expand the loading functions, \(P_\theta, P_\tau, P_\phi\), in terms of Fourier series. These expansions have the following forms:[1]

\[ P_\theta = \sum_{n=0}^{\infty} P_{\theta n} \cos n\theta + \sum_{m=1}^{\infty} q_{\theta m} \sin n\theta \]

\[ P_\tau = \sum_{n=0}^{\infty} P_{\tau n} \cos n\theta + \sum_{m=1}^{\infty} q_{\tau m} \sin n\theta \]

\[ P_\phi = \sum_{n=1}^{\infty} P_{\phi n} \sin n\theta + \sum_{m=1}^{\infty} q_{\phi m} \cos n\theta \]  

(10)
For known loadings, the so-called "Fourier coefficients" \( P_{\theta n}, P_{\phi n}, Q_{\theta n}, \ldots \) can be determined using Fourier series analysis. Equations (9) have solutions which are separable in \( \theta \) and \( \phi \). For each value of \( n \) there are two different solutions: One in which \( P_{\theta n}, N_{\theta n} \) are functions of \( \phi \) multiplied by \( \cos n\phi \), while \( P_{\phi n}, N_{\phi n} \) are functions of \( \phi \) multiplied by \( \sin n\phi \); and the other, in which the roles of \( \theta \) and \( \phi \) are interchanged. Both solutions are found in the same way and for the first we write ;[1]

\[
\begin{align*}
P_{\theta} &= P_{\theta n} \cos n\phi \\
P_{\phi} &= P_{\phi n} \sin n\phi \\
P_{r} &= P_{r n} \cos n\phi \\
N_{\theta} &= N_{\theta n} \cos n\phi \\
N_{\phi} &= N_{\phi n} \sin n\phi \\
N_{r} &= N_{r n} \cos n\phi
\end{align*}
\]

Where \( N_{\theta n}, N_{\phi n}, N_{r n}, P_{\theta n}, P_{\phi n}, P_{r n} \) are, in general, functions of \( \phi \). Substituting these expressions into (13) and canceling the common factor of \( \cos n\phi \) in (9a), \( \sin n\phi \) in (9b) we find:

\[
\begin{align*}
\frac{\partial N_{\theta n}}{\partial \phi} + \left(1 + \frac{r_1}{r_2}\right) N_{\theta n} \cot \phi + n \frac{N_{\phi n}}{\sin \phi} \frac{r_1}{r_2} &= r_2 \left(-P_{\theta n} + P_{r n} \cot \phi \right) \quad (13a) \\
\frac{\partial N_{\phi n}}{\partial \phi} + 2 \frac{r_2}{r_1} N_{\phi n} \cot \phi + n \frac{N_{\theta n}}{\sin \phi} &= r_1 \left(-P_{\phi n} + \frac{n}{\sin \phi} P_{r n} \right) \quad (13b)
\end{align*}
\]

These ordinary differential equations can be solved analytically on numerically. Since equations (9) are linear we may superimpose any of these solutions to obtain other solutions; typical shell analyses and designs are based on just one or two terms.

### 4.1. Stresses in Domes under Seismic Load

For a simple model of earthquake force, acting on the shells of revolution, we assume the following distribution,

\[
\begin{align*}
P_{r} &= -P \cos \theta \sin \phi \\
P_{\theta} &= -P \cos \theta \cos \phi \\
P_{\phi} &= -P \sin \theta \sin \phi
\end{align*}
\]

For a hemispherical dome of radius \( a \) subjected to this load, equations (13) become:
In terms of the new variables:

\[ U = N_{\theta_{\text{in}}} + N_{\phi_{\text{in}}} \]  
\[ V = N_{\phi_{\text{in}}} - N_{\theta_{\text{in}}} \]  

(16)

The equations become:

\[ \frac{\partial U}{\partial \phi} + \left(2 \cot \phi + \frac{n}{\sin \phi}\right) U = a \left(-p_{\theta_{\text{in}}} - p_{\phi_{\text{in}}} + \frac{n + \cos \phi}{\sin \phi} p_{\text{m}}\right) \]  
\[ \frac{\partial V}{\partial \phi} - \left(2 \cot \phi - \frac{n}{\sin \phi}\right) V = a \left(p_{\phi_{\text{in}}} - p_{\theta_{\text{in}}} - \frac{n - \cos \phi}{\sin \phi} p_{\text{m}}\right) \]  

(17)

Each of these first order differential equations has the form:

\[ \frac{\partial U}{\partial \phi} + p(\phi) U + q(\phi) = 0 \]  

(18)

The general solution to this equation is:

\[ U = \left[ C - \int p(\phi) \, d\phi \right] e^{-\int p(\phi) \, d\phi} \]  

(19)

With the help of relations (16) and (18) we find:

\[ U = \left[ A - 2p_{\theta_{\text{in}}} \left( -\cos \phi + \frac{\cos^2 \phi}{2} \right) \right] \left( \frac{1 + \cos \phi}{\sin^2 \phi} \right) \]  
\[ V = \left[ B + 2p_{\phi_{\text{in}}} \left( -\cos \phi - \frac{\cos^2 \phi}{2} \right) \right] \left( \frac{1 - \cos \phi}{\sin^2 \phi} \right) \]  

(20)

Returning to the relations (16) and multiplying the resulting expressions, for the actual field variables \( N_{\theta_{\text{in}}}, N_{\phi_{\text{in}}} \), by \( \cos \Theta \) and \( \sin \Theta \), respectively, we obtain:

\[ N_{\theta_{\text{in}}} = \frac{\cos \Theta}{\sin^3 \phi} \left[ \frac{A + B}{2} + \frac{A - B}{2} \cos \phi + Pa \cos^2 \phi \right] \]  
\[ N_{\phi_{\text{in}}} = \frac{\sin \Theta}{\sin^3 \phi} \left[ \frac{A - B}{2} + \frac{A + B}{2} \cos \phi + 2Pa \cos \phi - Pa \cos^2 \phi \right] \]  

(21)
The integration constants $A$ and $B$ can be determined by imposing the physical condition that $N_\phi$ and $N_{\theta\phi}$ must be finite at $\phi = 0$. Hence after some algebraic manipulations, we obtain the following final solution to the problem:

\[
N_\phi = \frac{Pa(1 - \cos \phi)}{\sin \phi (1 + \cos \phi)} \cos \phi
\]

\[
N_{\theta\phi} = \frac{-Pa(1 - \cos \phi)(\cos \phi + 2)}{\sin \phi (1 + \cos \phi)} \sin \phi
\]

\[
N_\theta = \frac{-pa(1 - \cos \phi)(2 + 2\cos \phi + \cos^2 \phi)}{\sin \phi (1 + \cos \phi)} \cos \phi
\]

Figure 6. Variation of internal membrane forces in a hemispherical dome subjected to seismic load

5. CONCLUSION

In this study, a closed form solution for a hemisphere membrane shell of revolution under lateral seismic load is presented. The results are sketched in Figure 6. The curves indicate that all three forces, i.e., hoop, meridional, and shear components reach their maximum at the support, vary non-linearly over the height, and reduce to zero at apex. The outcome of this work can be used for analysis of dome structures under seismic inertial load.

REFERENCES

OPTIMUM DESIGN OF 2-D REINFORCED CONCRETE FRAMES USING A GENETIC ALGORITHM

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ABSTRACT
Construction of concrete structures involves at least three different materials: concrete, steel and formwork. A large number of parameters, therefore, have to be dealt with in proportioning a reinforced concrete element, including width, depth, number and diameter of rebar. Consequently, together with experience, trial and adjustment are necessary in the choice of concrete sections. A trial section has to be chosen for each critical location in a structural system. The trial section has to be analyzed to determine if its nominal resisting strength is adequate to carry out the applied factored loads. Since more than one trial is often necessary to arrive at the required section, this process is time consuming. Also, the final design of a practiced designer is different from that of a beginner and it is never known whether the result is an optimum design.

The objective of this research is to design optimally reinforced concrete frames that satisfy the limitations and specifications of the American Concrete Institute (ACI) Building Code and Commentary using a Genetic Algorithm (GA). The GA used in this study has an adaptive penalty function. New options are added to the GA, including tournament selection with specified conditions or repairing operator that acts on beams and columns to accelerate convergence of the program. Design results show that the algorithm presented here compares advantageously with classic methods or other GA algorithms used previously for optimum design of concrete frames.

Keywords: concrete frame, genetic algorithm, optimization, design, reinforced concrete

1. INTRODUCTION
In the design of RC frames in proportioning a reinforced concrete element, members’ width and depth and the number and diameter of bars have to be dealt with. Consequently, trial and adjustment are necessary in the choice of concrete sections.

The objective of this paper is to design optimally reinforced concrete frames that comply with the limitations and specifications of the American Concrete Institute (ACI) Building Code and Commentary using a Genetic Algorithm (GA). The optimization of the reinforced concrete members is more challenging than the optimization of members made of isotropic materials, such as steel. The problem
has been considered by several researchers. Krishnamoorty and Mosi (1981) presented cost optimization of two-dimensional frames with rectangular cross-sections using sequential unconstrained minimization technique (SUMT). They considered nonlinear constitutive relationships but had no actual design code. Their cost function includes only the material costs of concrete, steel reinforcement and formwork. Moharrami and Grierson (1993) carried out minimum cost design of RC building frames subjected to vertical and lateral loading, based on the ACI code ("Building" 1989) using the Optimality Criteria approach. The columns had rectangular cross-sections and the beams were considered rectangular, L or T shapes. Their design variables were the width, depth and longitudinal steel reinforcement of the beams and columns. Their cost function included the material costs of the concrete, reinforcement and the formwork. Fadaee and Grierson (1996) presented minimum cost design of three-dimensional RC frames with members subjected to biaxial moments and shear forces based on the ACI code ("Building" 1995). Beams and columns were assumed to have rectangular sections. The cost function included the material costs of concrete, steel and the formwork. Later, Camp, Pezeshk and Hanson (2003) discussed optimum flexural design of two-dimensional reinforced concrete frames using a genetic algorithm (GA). The frames were subjected to vertical and lateral loads and their beams and columns had rectangular sections. They applied a modified version of GA to achieve a low-cost design according to the ACI code ("Building" 1999). The design variables used were depth and width of the sections and the number and diameter of the reinforcement bars. Their cost function included the material costs of the concrete, reinforcement and the formwork. Lee and Ahn (2003) also minimized the cost of two-dimensional reinforced concrete frames subjected to gravity and lateral loading based on the ACI code ("Building" 1999) and the UBC (1997) using GA. Beams and columns had rectangular sections and their design variables were depth and width of the sections and area of the reinforcement. Their cost function included the material costs of the concrete, reinforcement and the formwork. Chan and Wang (2006) also carried out optimum nonlinear stiffness design of two-dimensional tall reinforced concrete buildings under service loads. Beams and columns had rectangular sections and area of the reinforcement was assumed as constant. The cost function also included only the concrete cost.

2. FORMULATION OF THE COST FUNCTION
The first step in an optimization is determination of the objective function. In this research, this function includes costs of the concrete, steel and formwork for beams and columns, where formwork cost includes labor cost. Beams and columns have rectangular sections. The design variables are depth and width of the sections and number and diameter of the reinforcement bars, hence the reinforcement topology can be determined.

Description of the cost function for beams is as follows,

Minimize $F$ :
\[
F = C_c l_b b_h h_b + C_s (l_m A_{abl} + l_m A_{sbn} + l_m A_{shr}) + C_f l_b (2h_b + b_h)
\]  

Subjected to \( c_{1b} \leq 0 \quad c_{2b} \leq 0 \quad \ldots \quad c_{nb} \leq 0 \)

Where, \( C_c \) is cost of the concrete per cubic foot; \( C_s \) is cost of the steel per cubic foot; \( C_f \) is cost of the formwork per square foot (including labor); \( l_b \) is length of the beam; \( b_h \) is width of the beam and \( h_b \) is height of the beam. In this research, beams are subdivided into three segments: the left segment, the middle segment and the right segment. According to this subdivision, \( l_{1d} \) is length of the left segment of beam; also \( A_{afl} \) is area of the reinforcement for the left segment.

Similarly \( l_{bm} \), \( l_{br} \), \( A_{sbn} \) and \( A_{shr} \) are length of the middle segment, length of the right segment, area of the reinforcement for the middle segment and area of the reinforcement for the right segment, respectively. \( c_{1b} \), \( c_{2b} \), \ldots, \( c_{nb} \) are beams constraint functions according to specifications and limitations of the ACI-318-05 code and commentary (ACI-318R-05).

Description of the cost function for columns is as follows,

Minimize \( F \) :
\[
F = C_c l_c b_c h_c + C_s l_c A_{nc} + 2C_f l_c (h_c + b_c)
\]  

Subjected to \( c_{1c} \leq 0 \quad c_{2c} \leq 0 \quad \ldots \quad c_{nc} \leq 0 \)

Where, \( l_c \) is length of the column; \( b_c \) is width of the column; \( h_c \) is height of the column and \( A_{nc} \) is area of the reinforcement. \( c_{1c} \), \( c_{2c} \), \ldots, \( c_{nc} \) are columns constraint functions according to specifications and limitations of the ACI-318-05 code and commentary (ACI-318R-05).

3. PENALTY FUNCTION

All engineering optimization problems have constraints to satisfy, whereas GA is basically introduced for unconstrained optimization. To overcome this problem and optimize engineering problems with GA, we can apply penalty functions which are proposed for constrained problems to convert them into unconstrained problems. Several different ideas have been proposed to improve penalty function methods for engineering constrained optimization problems. In this research, the method introduced by Bean and Hadj-Alouane (1992) is employed. Their penalty function is revised, based on the feasibility or infeasibility of the best penalized solution during recent generations. Their penalty function allows either an increase or a decrease in the imposed penalty during evolution as shown below. This involves the selection of two constants, \( \beta_1 \) and \( \beta_2 \) (\( \beta_1 > \beta_2 > 1 \)), to adaptively update the penalty function multiplier, and the evaluation of the feasibility of the best solution over successive intervals of \( N_f \) generations. As the search progresses, in every \( N_f \)
generation the penalty function multiplier is updated, based on whether the best solution was feasible during that interval. Specifically, the penalty function is,

\[ f_p(X,k) = f(X) + \sum_{i=1}^{m} \lambda_i d_i^k \]  

(3)

\[ \begin{cases} \lambda_k \beta_1 & \text{If previous } N_f \text{ generations have infeasible best solution} \\ \lambda_k / \beta_2 & \text{If previous } N_f \text{ generations have feasible best solution} \\ \lambda_k & \text{Otherwise} \end{cases} \]

\[ \beta_1 > \beta_2 > 1 \]

It is recommended that \( \beta_1 = 5 \) and \( \beta_2 = 3 \).

The total cost function can then be determined as,

\[ F_T = F + \sum_{i=1}^{m} \lambda_k d_i^k \]  

(4)

After analyzing each frame, the feasibility of the frame can be assessed. If the solution is infeasible, it is penalized; otherwise the penalty term in the cost function is set at zero.

4. FRAME ELEMENTS SPECIFICATIONS

4.1. Beams

Flexural moment varies along a beam in a frame. At supports, the negative moments govern, whereas, in the middle of the beam it is the positive moment that governs. For this reason, beams are subdivided into three segments, left, middle and right as shown in Figure 1,

![Figure 1. Beams subdivision along their lengths.](image)

Each segment can be designed according to its maximum moment, thereby individual longitudinal reinforcement for each segment can be found. Notice that width and depth of the section are constant along the beam.

Beams are assumed to have rectangular sections and the longitudinal reinforcements of the beams are arranged in only one layer for tensile and compression steel. Number and size of the bars in different rows are not the same but all bars in a row are of the same size. Beams section specifications are shown
in Figure 2,

Figure 2. Beams section specifications.

Moreover, shear reinforcement will be calculated for beams. Indeed, at first, beams dimensions and longitudinal reinforcements will be produced by GA and then the shear reinforcement will be calculated for the given specifications. GA does not produce shear reinforcement; it produces only section dimensions and number and size of the longitudinal bars. Therefore, for each beam there exist fourteen variables, two for width and depth of the section for the whole beam and four for number and size of the longitudinal bars in each segment.

4.2. Columns
Columns are considered to have a uniform section along their height. In other words, their section dimensions and number and size of the longitudinal bars are constant along their length. Similar to beams, columns have rectangular sections and their longitudinal reinforcements are arranged in only one layer for tensile and compression steel. But unlike beams, number and size of the bars in different rows are of the same size. According to the ACI, minimum number of longitudinal bars in columns is assumed to be four, two for each row.

Production of section dimensions, number of longitudinal bars and their sizes and finally calculation of shear reinforcement is the same as that of the beams. Consequently four variables participate in columns design, two for width and height of the section and two for number and size of the longitudinal bars.

5. REINFORCED CONCRETE FRAMES AND RELATED CONSTRAINTS
All engineering structures have to be resistant under the applied loads. They must carry loads safely, not deform excessively. ACI-318-05 code, used in this research, outlines relations needed for design of concrete structures. These relations form constraints which are applied to beams, columns and concrete frame.

5.1. Beams Constraints
1. Moment constraint: As mentioned before, flexural moment varies along the beams. Accordingly, this constraint is calculated for the left, the middle and the right segment of beams.
2. Maximum spacing for crack control: According to the Portland Cement Association (PCA) notes on ACI, the spacing of reinforcement (Grade 60 bars) closest to a surface in tension shall not exceed that given in tables 9A-1 and
3. Maximum deflection constraint: According to the ACI code, computed deflection of a beam, not supporting or attached to nonstructural elements likely to be damaged by large deflections, shall not exceed $\frac{1}{360}$.

4. Minimum width constraint: Minimum clear spacing between parallel bars in a layer shall be $d_{ls}$, but not less than 1 inch.

5. In this research, widths of the beams are restricted to widths of their associated columns.

6. Another restriction applied to the beams is that their widths are limited to their depths.

7. Maximum depth constraint: "It is also common practice in design of reinforced concrete beams to fix the maximum ratio of the depth to the width of the beam. Typically, $h_{\text{max}} / b$ varies from 2 to 3", [3]. In this research this ratio is set at 2.5.

8. Minimum shear reinforcement constraint: Shear reinforcement designed for specified section characteristics shall not be less than minimum shear reinforcement specified by ACI-318-05. The constraint $m_8$ defined for this case is calculated for the left and the right support shear forces separately.

9. Maximum shear reinforcement constraint: Shear reinforcement must satisfy the relation below,

$$V_s \leq 4V_c$$  

(5)

5.2. Columns Constraints

1. Minimum longitudinal reinforcement ratio constraint: Longitudinal reinforcement ratio of the columns shall not be less than 0.01.

2. Maximum longitudinal reinforcement ratio constraint: Longitudinal reinforcement ratio of the columns shall not be greater than 0.08.

3. Minimum width constraint: In tied reinforced compression members, clear distance between longitudinal bars shall be not less than $1.5d_s$ nor less than 1.5 inches.

4. Constraint related to column interaction diagram: Columns in structural systems are rarely subjected to pure axial force; rather a combination of axial force and flexural moment is exerted to columns. This matter affects columns strength, and interaction of axial force and flexural moment has essential role in calculation of columns capacity. For design purpose, column load-moment strength interaction is used. If the factored axial force and bending moment lies inside the design strength diagram, the capacity of the column is satisfactory.

5. Since the frame under study is a two-dimensional frame, widths of the columns are limited to their depth.

6. Minimum shear reinforcement constraint: Shear forces are not the major criterion in columns design. Indeed columns are designed for axial force and bending moments, then they are checked for shear forces. Similar to beams,
shear reinforcement of the column shall not be less than the minimum permitted shear reinforcement specified by ACI-318-05.

7. Maximum shear reinforcement constraint: Shear reinforcement must satisfy the relation below,

\[ V_s \leq 4V_c \]  \hspace{1cm} (6)

5.3. Frame Constraints
Besides beams and columns constraints, stability of the frame is considered as a whole frame constraint. If the frame is not stable laterally, the related constraint is applied to the frame.

6. GENETIC ALGORITHM
The genetic algorithm used in this research has an adaptive penalty function that has been already explained. The crossover operator has the probability of 0.8, and three types of operators; one-point, two-point and uniform operators have been used. To determine which type of crossover is used, a two-bit binary string is produced. The string "00" refers to one-point crossover, the strings "01" and "10" refer to two-point crossover and finally the string "11" refers to a uniform crossover. One- and two-point crossovers are the least disruptive to the population, while uniform crossover is the most disruptive operator. The mutation rate varies between 0.008 and 0.02. If previous generation has feasible best solution, the rate is 0.008, otherwise the rate is 0.02. Selection operator is a binary tournament selection with specific conditions. Sometimes in design procedure, the designer comes across sections that violate constraints, but minor changes in section depth, section width, number of bars or diameter of bars makes it a convenient section. This problem has been dealt with adding a new operator to GA called repairing.

7. DESIGN EXAMPLES
7.1. Uniaxial Short-Tied Columns
The first example is a problem presented by Zielinski et al. (1995). The aim is to design three uniaxial short-tied columns, each column is subjected to a factored axial force, \(P_f\), and a factored bending moment, \(M_f\). Loadings and material properties for each column is listed in Table 1.

<table>
<thead>
<tr>
<th>Design example</th>
<th>(d') (in.)</th>
<th>(f_s) (psi)</th>
<th>(f'_c) (psi)</th>
<th>(P_f) (lb)</th>
<th>(M_f) (ft-lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.56</td>
<td>58,015</td>
<td>3,626</td>
<td>553,030</td>
<td>326,740</td>
</tr>
<tr>
<td>2</td>
<td>2.76</td>
<td>58,015</td>
<td>4,351</td>
<td>400,160</td>
<td>266,997</td>
</tr>
<tr>
<td>3</td>
<td>2.95</td>
<td>58,015</td>
<td>4,351</td>
<td>449,618</td>
<td>414,510</td>
</tr>
</tbody>
</table>
Values for the column dimensions in inches range from $7 \leq b \leq 30$ and $7 \leq h \leq 30$. Camp, Pezeshk and Hanson (2003) designed this problem according to ACI-318-99, using a modified version of GA. In this research the designed columns have been checked by drawing their exact interaction diagrams. These diagrams show that the columns have acceptable design and their axial forces and bending moments lie inside the design diagrams. Comparison between the exact interaction diagrams and the interaction diagrams drawn in this research, shows that the latter has good accuracy. Figure 3 shows both diagrams and the point related to loading for the first column.

Table 2 lists the design results. It should be noted that the costs in the table are for 1 foot height of columns. Columns "Result (Ave)", "Result (Min)" and "Result (Max)" show decrease or increase percents in cost for average, minimum and maximum results attained in this research versus result attained by Camp, Pezeshk and Hanson (2003), respectively. All costs are in terms of US dollar.

### Table 2: Design results for uniaxial short-tied columns

<table>
<thead>
<tr>
<th>Column</th>
<th>Population Size</th>
<th>Number of Generations</th>
<th>Ave-Cost ($)</th>
<th>Min-Cost ($)</th>
<th>Max-Cost ($)</th>
<th>Min-Size (in.)</th>
<th>Max-Size (in.)</th>
<th>Ave (sec)</th>
<th>Min (sec)</th>
<th>Result (Ave)</th>
<th>Result (Min)</th>
<th>Result (Max)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Camp, Pezeshk, Hanson</td>
<td>200</td>
<td>50</td>
<td>39.01</td>
<td>39.38</td>
<td>41.42</td>
<td>12 * 30</td>
<td>16.93%</td>
<td>14.78%</td>
<td>19.85%</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>This research</td>
<td>200</td>
<td>50</td>
<td>36.98</td>
<td>38.04</td>
<td>39.65</td>
<td>12 * 29.5</td>
<td>4.00%</td>
<td>2.78%</td>
<td>4.31%</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### 7.2. Two-Bay Six-Story Frame

Figure 4 shows a two-bay six-story reinforced concrete frame designed by Rajeev...
and Krishnamoorthy (1998) based on Indian Standard Code of Practice for Reinforced Concrete (IS 1978) design code. The frame was also designed by Camp, Pezeshk and Hanson (2003) based on the ACI-318-99 code using a modified version of GA.

Figure 4. Two-bay six-story frame: (a) geometry and loading; and (b) beam and column group numbering [3].

The dimensions of the frame are: \( h = 4m(13.12\text{ ft}) \), \( L_1 = 6m(19.69\text{ ft}) \), and \( L_2 = 4m (13.12\text{ ft}) \). A factored uniformly distributed vertical load of \( w = 30KN/m (2056/lb/\text{ ft}) \) is applied to every beam in the frame. In addition, a lateral load of \( P = 10KN(2248/lb) \) is applied to each story. The cost of concrete, steel and formwork is estimated as \( \$735/m^2(\$20.81/\text{ ft}^2), \$7.1/kg (\$1.7/\text{ kg}) \), and \( \$54/m^2(\$5.02/\text{ ft}^2) \), respectively (Rajeev and Krishnamoorthy 1998). The unit weight of concrete and steel is approximately \( 145(lbs/\text{ ft}^3) \) and \( 490(lbs/\text{ ft}^3) \). The strength of concrete, \( f_c = 3000 (\text{ psi}) \), and the yield strength of steel, \( f_y = 60000(\text{ psi}) \).

Rajeev and Krishnamoorthy (1998) did not consider the shear capacity of the beam sections, while Camp, Pezeshk and Hanson (2003) designed the frame with and without shear capacity of the beam. Table 3 shows design results attained by Camp, Pezeshk and Hanson (2003) when the moment and shear capacities of the beams and the load-moment interaction in the columns with moment magnification due to frame stability and column slenderness have been considered, but they did not design shear reinforcements. Also longitudinal reinforcement was considered constant along the beam.
In this research, there are not any groupings for beams and columns. Each beam and column has its particular section, defined in the previous sections. The moment and shear capacities of the beams and columns have been considered and longitudinal and shear reinforcements have been designed for each element. Figure 5 shows numbering of the frame elements.

This frame has 240 variables, 168 variables for beams and 72 variables for columns. Design time for 200 generations with a population size of 150 is about 80 min. The best solution has the cost of US$30910. Table 4 lists section dimensions, longitudinal reinforcements designed for some elements of the best solution. In this table:

- \( b \): Width of the section, \( \text{in} \).
- \( h \): Height of the section, \( \text{in} \).
- \( \text{nb}_b(l/m/r) \): Number of bottom bars in the left/mid/right segment.
- \( \text{nb}_u(l/m/r) \): Number of upper bars in the left/mid/right segment.
- \( \text{n_bar}_b(l/m/r) \): Bottom bars size for the left/mid/right segment.
- \( \text{n_bar}_u(l/m/r) \): Upper bars size for the left/mid/right segment.
Table 4: Section dimensions, longitudinal reinforcements designed for some elements of the best solution.

<table>
<thead>
<tr>
<th>No</th>
<th>b (m)</th>
<th>h (m)</th>
<th>n_b_b</th>
<th>n_b_m</th>
<th>n_u_b</th>
<th>n_u_m</th>
<th>n_bar_b</th>
<th>n_bar_m</th>
<th>n_bar_u</th>
<th>n_bar_m_u</th>
<th>n_bar_u_m</th>
<th>n_bar_u_r</th>
<th>n_bar_m_u_r</th>
</tr>
</thead>
<tbody>
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<td>4</td>
<td>4</td>
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<tr>
<td>7</td>
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<td>22</td>
<td>3</td>
<td>4</td>
<td>2</td>
<td>2</td>
<td>6</td>
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<td>6</td>
<td>4</td>
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<td>5</td>
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</tr>
<tr>
<td>15</td>
<td>10</td>
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<td>2</td>
<td>2</td>
<td>2</td>
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<td>6</td>
<td>6</td>
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<td>6</td>
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<td>6</td>
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<tr>
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<td>16</td>
<td>3</td>
<td>3</td>
<td>2</td>
<td>2</td>
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<td>5</td>
<td>5</td>
<td>5</td>
<td>4</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>30</td>
<td>9</td>
<td>10</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>4</td>
<td>4</td>
<td>4</td>
<td>4</td>
<td>4</td>
<td>4</td>
<td>4</td>
</tr>
</tbody>
</table>

Table 5 lists shear reinforcements designed for some elements of the best solution.

Table 5. Shear reinforcements designed for some elements of the best solution.

<table>
<thead>
<tr>
<th>No</th>
<th>( Av_s (l/r) )</th>
<th>( Av_s _min (l/r) )</th>
<th>( x_s_5v_c (l/r) )</th>
<th>( x_s_c (l/r) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.0132</td>
<td>0.0132</td>
<td>0.0132</td>
<td>0.0132</td>
</tr>
<tr>
<td>7</td>
<td>0.01</td>
<td>0.0165</td>
<td>0.01</td>
<td>0.01</td>
</tr>
<tr>
<td>10</td>
<td>0.0080</td>
<td>0.0166</td>
<td>0.0075</td>
<td>0.0075</td>
</tr>
<tr>
<td>15</td>
<td>0.0083</td>
<td>0.0083</td>
<td>0.0083</td>
<td>0.0083</td>
</tr>
<tr>
<td>24</td>
<td>0.0155</td>
<td>0.0075</td>
<td>0.0075</td>
<td>0.0075</td>
</tr>
<tr>
<td>30</td>
<td>0.0075</td>
<td>0.0075</td>
<td>0.0075</td>
<td>0.0075</td>
</tr>
</tbody>
</table>

In this table:
- \( Av_s (l/r) \): Shear reinforcement area in unit length for the left/right half of the elements, \( in^2/in \).
- \( Av_s \_min (l/r) \): Minimum permitted shear reinforcement area in unit length for the left/right half of the elements, \( in^2/in \).
- \( x\_s\_5v_c (l/r) \): Distance between point with shear force of \( 5v_c \) (\( v_c \) is shear resistance provided by concrete) and point with shear force of zero for the left/right half of the elements, \( in \).
- \( x\_s\_c (l/r) \): Distance between point with shear force of \( v_c \) (\( v_c \) is shear resistance provided by concrete) and point with shear force of zero for the left/right half of the elements, \( in \).

In this research, design time has been assumed as a determining parameter, here the selection operator with specified conditions and now the operator repairing has been used to accelerate convergence of the program. In fact, the selection used in this research causes fewer violated constraints to get in a population as the generation number grows, and then these selected populations are repaired by using the repairing operator. As a result, design time for the two-bay six-story frame with 240 variables is about 80 min., whereas the program used by Camp, Pezeshk and Hanson (2003) need 13 hours to design the same frame with only 36 variables.

8. CONCLUSIONS

In this research, optimal design of reinforced concrete frames regarding cost was considered within the limitations and specifications of the ACI code. To design these frames a GA based optimization program was introduced. This program is shown to be applicable and effective especially for problems with large numbers of constraints. Tournament selection with specified conditions and a new repairing operator was used to decrease the design time. The design time in this research is
much shorter than the design time needed for the GA implemented by Camp, Pezeshk and Hanson (2003). Design time for the modified version of GA implemented by the latter researchers was about 9.75 times the design time needed for the GA introduced in this research, while the frame variables used in this research are about 6.67 times the variables used for the frame designed by Camp, Pezeshk and Hanson (2003).

9. UNIT CONVERSIONS
1 in. = 25.4 mm, 1 kip = 4,450 N, 1 k in. = 113 N mm, 1 ksi = 6.9 MPa

REFERENCES
A DIRECT APPROACH FOR DUCTILITY COMPUTATION OF FLEXURAL RC MEMBERS

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ABSTRACT

In seismic areas, ductility is an important factor in design of concrete members under flexure; it is due to the increase in capacity of plastic displacement. As a result, the inertial forces imposed on the structures can be decreased. The effective factors on ductility are; concrete compression strength \( f'c \), the percentage of tension and compression steel, \( \rho \) and \( \rho' \), the amount of stirrups confinement for concrete \( \rho_c \), the stirrups spacing, brittle effect of concrete strength, yield stress of longitudinal bars \( f_y \) and the effect of width to the depth of the section \( b/h \). Perhaps the most simple and general definition for section ductility of members is defined, as the ratio of curvatures at ultimate load to curvatures at yield load (\( \mu = \phi_u / \phi_y \)). In this paper, a proposed method was considered to calculate the flexural curvature ductility ratio of reinforced concrete (RC) sections. Based on the proposed method, computer software was produced to calculate the curvature ductility in confined RC beams. The method is based on actual characteristics of a concrete flexural section by considering almost all effective ductility parameters such as available experimental concrete compression diagrams. By the developed software, the ductility factor of 250 beams under efficient circumstances were investigated completely. The nonlinear multiple regression analyses was also performed for these 250 beams and a direct equation is introduced to determine the ductility factor. Based on the obtained experimental results a comparison was made between the proposed direct method and experimental results, and it was shown that a good agreement is available.

Keywords: ductility, flexural member, RC, nonlinear multiple regression

1. INTRODUCTION

In seismic areas, ductility is an important factor in design of concrete members under flexure; it is due to the increase in capacity of plastic displacement. As a result, the inertial forces imposed on the structures can be decreased [1-2]. The effective factors on ductility are; concrete compression strength \( f'c \), the percentage of tension and compression steel, \( \rho \) and \( \rho' \), the amount of stirrups confinement for concrete \( \rho_c \), the stirrups spacing, brittle effect of concrete strength, yield stress of longitudinal bars \( f_y \) and the effect of width to the depth of the section \( b/h \) [3-10].

Beams ductility can be presented based on behaviour of members section or the entire members’ behaviour. Prevalent criterion of beams ductility calculation according to entire members’ behaviour are the ratio of ultimate displacement to yield displacement \( \mu = \Delta_u/\Delta_y \), ratio of ultimate rotation to yield rotation \( \mu = \theta_u/\theta_y \) and the value of structure absorbed energy. Perhaps the most simple and general definition for section ductility of members is defined, ratio of curvatures at ultimate load to curvatures at yield load \( \mu = \phi_u/\phi_y \). The entire members’ behaviour reveals the actual behaviour of the structure but calculation of member section behaviour is simpler. However, the experimental results show that the difference between curvature and displacement value of ductility are quite small [6, 8] and hence, the curvature ductility is used generally to investigate the member behaviour.

The effect of concrete confinement with ties on flexure ductility was studied by many researchers [4, 9, 11-14]. But in this research, ductility is calculated based on the actual characteristics of a RC flexural section (Experimental strain-stress curves for confined and unconfined concrete) and act as a separator proposed curves, which divide the zone into; effective confined concrete core, unconfined concrete core and unconfined concrete cover. The method is also based on actual characteristics of a RC flexural section by considering almost all effective ductility parameters such as available experimental concrete compression diagrams.

The calculations of the accurate values of curvature ductility of members are usually complicated particularly in confined concrete beams and therefore, by the use of simplified formula can be much easier [15]. Lee and Pan presented an algorithm and simplified formulas for estimating the relationship between only the tension reinforcement and ductility of reinforced concrete beams. They considered the effects of concrete confinement and spilling of the concrete cover. Calculating of ductility based on Lee and Pan’s method is time consuming and difficult.

Based on a proposed method in this research, computer software was produced to calculate the curvature ductility in confined beams. By the developed software, the ductility factor of 250 RC beams under efficient circumstances which are mentioned above were investigated completely. The nonlinear multiple regression analysing was also performed for these 250 RC beams data and a direct equation is introduced to determine the ductility factor.

To investigate the performance and accuracy of the proposed equation, fifteen HSC beams were cast and tested under bending and also the available results of nine HSC beams were selected from research by Tsong et al. [16].

2. CURVATURE DUCTILITY CALCULATION BASED ON PROPOSED METHOD

Perhaps the most simple and general definition for section ductility of members is defined as ratio of curvature at ultimate load to curvature at yield load \( \mu = \phi_u/\phi_y \) shown in Figure 1 where \( M_u \), corresponds to the moment at the beginning of the yielding flat plateau in the moment-curvature curve and \( M_y \), is the moment when the ultimate load was reached during testing.
3. EFFECTIVE CONFINED CONCRETE CORE

3.1. Separator Curves at Transverse Level

There have been many attempts to describe the separator curves of confined concrete. Sheikh and Uzumeri, Sheikh and Yeh and Hwang and Yun made analytical and experimental studies on the mechanism of concrete confinement by considering the various parameters [9, 13-14]. They introduced the concept of the effectively confined concrete area. Fafities and Shah and Woods et al. experimentally investigated the confinement effects of HSC columns [4, 17]. Effective confined concrete area that causes an increase in both member strength and ductility is less than core nominal area which is placed centre to centre of two adjacent transverse bars [13]. Effective confined area of concrete is calculated according to parameters, such as shape of transverse and distance between longitudinal bars. At the section of transverse level with regard to the distance between the longitudinal bars, some of compressive concrete core area is unconfined when it is under bending. The unconfined concrete in core area is hatched and is shown in Figure 2. Longitudinal bars confined concrete effectively in its vicinity. It is assumed that unconfined concrete stress for hatched area out of inside concrete core is uniform.

In this research, a confinement model is proposed to divide concrete inside tie into effective confined concrete core and unconfined concrete core. Therefore, the compression concrete zone of rectangular section under bending is divided by the separator proposed curves.

The relation for separator curves of confined and unconfined concrete areas is as:

$$Y = aX^n$$  \hspace{1cm} (1)

where, terms $a$ and $n$ are the experimental constants.

It is possible to obtain the values of $a$ and $n$ by considering the coordinate of the stirrup corner (point M) in Figure 2, and the separator curves which is lying
between a triangular and elliptical shape, in which the values of A (the hatched area) and \( \theta \) are respectively equal to \( C^2/5.5 \) and 45\(^\circ\) as Sheikh and Uzumeri suggested [13].

Hence;

\[
a = \frac{l}{1.75} \left( \frac{2}{C} \right)^{0.75} \quad \text{and} \quad n = 1.75
\]

Having the values of \( a \) and \( n \), and substituting in Eq. 1, the following equation is obtained:

\[
Y = \frac{1}{1.75} \left( \frac{2}{C} \right)^{0.75} \times X^{1.75} \quad \text{or} \quad Y = \frac{1}{1.75} \left( \frac{2}{C} \right)^{0.75} |X|^{1.75}
\]

where \( C > 0 \), \( -\frac{C}{2} \leq X \leq \frac{C}{2} \)

Now, the Eq. 4 can be written by considering the known values of \( a \) and \( n \) and the coordinate of point M.

\[
h = \frac{1}{1.75} \times \left( \frac{2}{C} \right)^{0.75} \times \left( \frac{C}{2} \right)^{1.75} \Rightarrow h = \frac{C}{3.5}
\]

3.2. Separator Proposed Curves at Midway Between Ties (i.e., critical section)

It is clear that, confinement of concrete is improved if transverse (stirrup) reinforced layers are placed relatively close together along the longitudinal axis of the beam. There will be some critical spacing of transverse reinforcement layers above which the section midway between the transverse sets will be ineffectively confined, and therefore, it seems the available equation will be inappropriate.

The concrete confinement between the stirrups sets (ie, the spacing between two adjacent stirrups) is affecting on buckled place between stirrups sets. The minimum effective confinement lies between two stirrups. This is clearly illustrated in Figure 3. The maximum value of \( Y \) is located at a section midway between stirrups sets. Here, such a section is called a critical section. A suggested value of \( 0.25S \tan\theta \) is reported by Sheikh and Uzumeri [13], where \( \theta = 45^\circ \). Hence, for analysing purpose, the critical section can be calculated by considering of actual effective concrete confinement area at transverse level which is equal to area that 0.25S at sides of width and height of the section.

Now, Figure 4, illustrates a proposed critical section along the beam’s length and it is possible to define the equations for the proposed separator curves in such a section, for both confined concrete and unconfined effective concrete core.

To obtain, the distance \( O' \) from axis \( X \) and \( Y \), the following operations are treated;

\[
MG = d' + d'' - d'', \quad d''' = 0.25 \times S, \quad MG = d' + 0.25 \times S - d''
\]

\[
C_1 = b - 2 \times MG, \quad H_1 = \frac{C_1}{3.5}
\]
\[ C_2 = h - 2MG, \quad H_2 = \frac{C_2}{3.5}, \quad C_3 = C_2, \quad H_3 = H_2 \quad (5) \]

![Figure 2. Unconfined concrete in core and separator curve](image)

\[ C_1, H_1, C_2, H_2, C_3 \text{ and } H_3 \] are the base and height of the separator curves 1, 2 and 3 respectively. Therefore, based on Eq. 3, it is possible to derive the separator curves 1, 2 and 3 for each local coordinate curve (Figure 4).

The equation for separator curve 1:

\[ Y_1 = \frac{1}{1.75} \left( \frac{2}{C_1} \right)^{0.75} \left| X_1 \right|^{1.75}, \quad Y = MG + H_1 - Y_1, \quad X = \frac{b}{2} + X_1 \quad (6) \]
The equation for separator curve 2:

\[ Y_2 = \frac{1}{1.75} \left( \frac{2}{C_2} \right)^{0.75} \left( X_2 \right)^{0.75}, \quad Y = MG + H_2 - Y_2, \quad X = \frac{b}{2} + X_2 \]  

(7)

The equation for separator curve 3:

\[ Y_3 = \frac{1}{1.75} \left( \frac{2}{C_3} \right)^{0.75} \left( X_3 \right)^{0.75}, \quad Y = MG + H_3 - Y_3, \quad X = \frac{b}{2} + X_3 \]  

(8)

3.3. Flowchart to Determine the Moment-Curvature

By using separator curves; experimental stress-strain data of confined, unconfined concrete; and bars stress-strain relationship; a computer program was developed to calculate the moment-curvature curves for confined reinforced concrete beams (i.e., for both high strength and normal strength concrete (HSC and NSC)), and therefore, the curvature ductility is obtained based on moment-curvature curve. The proposed algorithm is demonstrated by the flowchart shown in Figure 5.

---

**Figure 5. Flowchart to obtain moment-curvature diagram**
By use of computer software which is based on proposed method, 250 beams with various assumed properties (shown in Table 1) were studied to determine the curvature ductility ($\mu_\phi$).

The electrical strain gauges were fixed on the surfaces of concrete cylindrical specimens and tested under compression and the stress-strain diagrams under the load were plotted. Two diagrams of total are shown in Figure 6. Behaviour of Steel was applied perfectly, where equation for steel was given in reference [18].

<table>
<thead>
<tr>
<th>$f'_c$</th>
<th>$\rho$</th>
<th>$\rho'$</th>
<th>$\rho_0$</th>
<th>$s/d$</th>
<th>$\mu_\phi$</th>
<th>$f'_c$</th>
<th>$\rho$</th>
<th>$\rho'$</th>
<th>$\rho_0$</th>
<th>$s/d$</th>
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<td>0.0087</td>
<td>0.01633</td>
<td>0.50</td>
<td>12.7</td>
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<td>0.0293</td>
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<td>41.3</td>
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<td>0.01633</td>
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<td>8.0</td>
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<td>0.0122</td>
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</tr>
</tbody>
</table>

$f_y=420$ MPa and $b/h=0.65$

4. ANALYZING THE DATA BY NONLINEAR REGRESSION METHOD

Any regression analysis should be preceded by a great deal of thought devoted to what variables should be included in the analysis, how these variables might influence the dependent variable, the correlation among the independent variables and ease of using a predictive model based on the selected independent variables. Therefore the first step in the regression analysis should be the development of the form of the predictive model based on a rational analysis of the problem. Regression analysis can then be used to develop the parameters of the model, test the importance of the variables included and develop confidence intervals for the predictions. A nonlinear model is defined as an equation that is nonlinear in the coefficients or a combination of linear and nonlinear in the coefficients. For example Gaussians, ratios of polynomials and power functions are all nonlinear. In matrix form, nonlinear models are given by the formula:

$$ y = f(x, \beta) + \varepsilon $$

(9)
Where \( y \) is an \( n \)-by-\( 1 \) vector of observations, \( f \) is a function of \( \beta \) and \( X \), \( \beta \) is a \( m \)-by-\( 1 \) vector of unknown parameters, \( x \) is the \( n \)-by-\( m \) matrix made up of \( n \) observations on each of \( m \) independent variables and \( \epsilon \) is an \( n \)-by-\( 1 \) vector of errors.

Nonlinear models are more difficult to fit than linear models because the coefficients cannot be estimated using simple matrix techniques. Instead, an iterative approach is required that follows these steps: a) Start with an initial estimate for each coefficient. For some nonlinear models, a heuristic approach is provided that produces reasonable starting values. For other models, random values on the interval \([0,1]\) are provided. b) Produce the fitted curve for the current set of coefficients. The fitted response value \( \hat{y} \) is given by:

\[
\hat{y} = f(x, \beta)
\]

and involves the calculation of the Jacobian of \( f(x,b) \), which is defined as a matrix of partial derivatives taken with respect to the coefficients. c) Adjust the coefficients and determine whether the fit improves. The direction and magnitude of the adjustment depend on the fitting method [19]. Many methods can be used to solve these problems such as Gauss-Newton. This method is potentially faster than the other methods, but it assumes that the residuals are close to zero.

The magnitude of ductility is dependent upon the different variables such as; concrete compression strength \( f'_c \), the percentage of tension and compression steel, \( \rho \) and \( \rho' \), the amount of stirrups confinement for concrete \( \rho_c \), the stirrups spacing, brittle effect of concrete strength, yield stress of longitudinal bars \( f_y \), and the effect of ratio of width to the height of the section, \( b/h \). Here, a constant longitudinal yield stress value of 420 MPa is assumed.

### Table 2: Details of testing program of tested beams [5-6] and Comparison of experimental and direct proposed equation results

<table>
<thead>
<tr>
<th>Beam No.</th>
<th>( f'_c ) (MPa)</th>
<th>( d ) (mm)</th>
<th>( d'_c ) (mm)</th>
<th>( \Lambda' )</th>
<th>( \rho ) (%)</th>
<th>( \rho_F )</th>
<th>( \rho'_c ) (%)</th>
<th>( \mu_{exp} )</th>
<th>( \mu )</th>
<th>Error (%)</th>
</tr>
</thead>
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<tr>
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<td>56.31</td>
<td>254</td>
<td>2014</td>
<td>0.61</td>
<td>-</td>
<td>-</td>
<td>( \rho_{min}=0.13 )</td>
<td>2014</td>
<td>0.61</td>
<td>11.15</td>
</tr>
<tr>
<td>B1</td>
<td>69.50</td>
<td>254</td>
<td>-</td>
<td>2014</td>
<td>0.61</td>
<td>-</td>
<td>-</td>
<td>-</td>
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<td>10.46</td>
</tr>
<tr>
<td>BC2</td>
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<td>250</td>
<td>2020</td>
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<td>0.24</td>
<td>2014</td>
<td>0.61</td>
<td>6.84</td>
<td>8.96</td>
<td>23.66</td>
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<tr>
<td>B2</td>
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<td>2020</td>
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<tr>
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<td>B4</td>
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<tr>
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Average error 21.12

\( \rho_c = 0 \) and \( f_y = 400 \)
Table 3: Details of testing program of tested beams [16] and Comparison of experimental and direct proposed equation results

<table>
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<th>Beam No.</th>
<th>$f'_c$ (MPa)</th>
<th>$A_c$</th>
<th>$\rho$ (%)</th>
<th>$\rho'$ (%)</th>
<th>Stirrups</th>
<th>Spacing (cm)</th>
<th>$\rho_c$ (%)</th>
<th>$\mu_{(exp)}$</th>
<th>$\mu_{(eq.11)}$</th>
<th>Error (%)</th>
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<td>3.39</td>
<td>2#4 1.51</td>
<td>4</td>
<td>5.47</td>
<td>25.63</td>
<td>35.05</td>
<td>26.87</td>
<td></td>
</tr>
<tr>
<td>C-2</td>
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<td>2#6</td>
<td>3.39</td>
<td>2#4 1.51</td>
<td>8</td>
<td>2.73</td>
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<td>19.54</td>
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</tr>
<tr>
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<td>2#4 1.51</td>
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<tr>
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<td>3.39</td>
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<td>C-5</td>
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<td></td>
<td></td>
<td></td>
<td>Average error 19.96</td>
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</tr>
</tbody>
</table>

The computer software, based on proposed method was testified for 250 RC beams of mentioned variables. Nonlinear regression method is used to analyze these 250 beams data. The analysis results provide the following direct equation to determine the ductility factor:

\[
\mu = 102748\rho_{c}^{1.24} + 2.33 \left( \frac{b}{h} \right)^{2.13} + 3.04 \left( \frac{f'_c}{E_c} \right)^{0.68} - 7692\rho^{0.49} + 27600\rho^{0.02} + 244755 \left( \frac{f'_c}{E_c} \right)^{0.93} + 8.39 \tag{11}
\]

R-square and average errors of the proposed equation are 0.91 and 13 percent respectively.

5. COMPARISON OF EXPERIMENTAL AND DIRECT PROPOSED EQUATION

To evaluate the accuracy of proposed direct Eq. 11, the experimental results of tests reported by Akbarzadeh and Maghsoudi, and Tsong et al. [5-6, 16] are investigated. Table 2 and 3 present the detailed testing programs. Curvature ductility is defined as the ratio of curvatures at ultimate load to curvatures at yield load ($\mu = \phi_u/\phi_y$). The experimental yielding curvature, $\phi_y$, corresponds to the curvature at the beginning of the yielding flat plateau in the moment-curvature curve. The experimental ultimate curvature, $\phi_u$, is the curvature when the ultimate load was reached during testing. For the tested beams, experimental curvature ductilities and the obtained ductility amount of these beams based on proposed equation are compared and shown in Table 2 and 3. The average error for experimental and proposed direct Eq. (11) is 20 percent, which indicates that a good agreement is available.

6. CONCLUSIONS

In seismic areas, flexural ductility is an important factor in design of concrete members. The calculation of the accurate values of ductility of a member is usually complicated and therefore a direct and accurate approach to obtain such value is

\[
\mu = 102748\rho_{c}^{1.24} + 2.33 \left( \frac{b}{h} \right)^{2.13} + 3.04 \left( \frac{f'_c}{E_c} \right)^{0.68} - 7692\rho^{0.49} + 27600\rho^{0.02} + 244755 \left( \frac{f'_c}{E_c} \right)^{0.93} + 8.39 \tag{11}
\]
necessarily required particularly in seismic regions. The proposed direct Eq. (11) for calculating the curvature ductility satisfies this requirement with an average error as low as 20%.

REFERENCES


NONLINEAR FINITE ELEMENT ANALYSIS OF FLANGED RC SHEAR WALLS BASED ON MULTI-LAYER SHELL ELEMENT

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ABSTRACT
Nonlinear simulations for structures under earthquakes have been widely focused on in recent years. However, precise modeling for the nonlinear behavior of reinforced concrete shear walls, which are the major lateral-force-resistant structural member in high-rise R.C. buildings, still has many items to be investigated. In this paper, based on the principles of composite material mechanics, a multi-layer shell element model is proposed to simulate the in-plane bending and the coupled in-plane bending-shear nonlinear behaviors of RC shear wall. The multi-layer shell element is made up of many layers with different layers of thickness, and different material models (concrete or rebar) are assigned to various layers so that the structural performance of the shear wall can be directly connected with the material constitutive law. Besides the traditional elasto-plastic-fracture constitutive model for concrete, which is efficient but does not give satisfying performance for concrete under complicated stress condition, a novel concrete constitutive model, referred as microplane model, which is originally proposed by Bazant et al., is developed to provide a better simulation for concrete in shear wall under complicated stress conditions and stress histories. Three flanged shear walls under static push-over load were analyzed with the proposed shear wall model for demonstration. The simulation results show that the multi-layer shell elements can correctly simulate the in-plane bending failure for tall flanged walls and the coupled in-plane bending-shear failure for short flanged walls. In addition, with microplane concrete constitutive law, the behavior and the damage accumulation of flanged shear wall can be appropriately modeled, which is very important for the performance-based design of structures under disaster loads.

Keywords: flanged shear wall, nonlinear analysis, microplane, multi-layer shell element

1. INTRODUCTION
Nonlinear simulations for structures under earthquakes have been widely focused on in recent years. However, precise modeling for the nonlinear behavior of reinforced concrete (RC) flanged shear walls, which are the major lateral-force-resistant structural member in medium-rise and high-rise buildings, still has not
been successfully solved. As the cross section of the flanged shear wall member is much bigger than that of the beam and column member, its deformation behavior under the lateral load is more complicated and the research has focused on the nonlinear analysis model for flanged shear wall. In this paper, based on the principles of composite material mechanics, a multi-layer shell element model is proposed to simulate the coupled in-plane/out-plane bending or the coupled in-plane bending-shear nonlinear behaviors of RC flanged shear wall. At the element level, the model uses the shell element that is made up of multiple layers with different thickness and different material models (concrete or rebar) are assigned to various layers. Since the model relates the nonlinear behaviors of the shear wall element to the constitutive relations of concrete and steel directly, it has many advantages in the description of the actual complicated nonlinear behaviors. In the nonlinear analysis for the concrete structures, the constitutive relation of the concrete has great effect on the analysis results. Although the traditional elasto-plastic-fracture constitutive model for concrete is efficient, it does not give satisfying performance for concrete under complicated stress condition. So at the material constitution level, a novel concrete constitutive model, referred to as microplane model, which is originally proposed by Bazant et al. [1, 2, 3 and 4], is developed to provide a better simulation for concrete in shear wall under complicated stress conditions and stress histories. In order to validate the capacity of the proposed shear wall model, three flanged shear walls with different nonlinear behaviors under given load cases were taken as examples. Pushover analysis and static cyclic loading analysis was carried out on these shear walls with the proposed shear wall model to illustrate the capacity of the proposed model.

2. MULTI-LAYER SHELL ELEMENT

The proposed multi-layer shell element is based on the principles of composite material mechanics and it can simulate the coupled in-plane/out-plane bending and the coupled in-plane bending-shear nonlinear behaviors of RC shear wall. Basic principles of multi-layer shell element are illustrated in Figure 1. The shell element is made up of many layers with different thickness. And different material properties are assigned to various layers. This means that the rebars are smeared into one layer or more. During the finite element calculation, the axial strain can be obtained in one element. Then according to the assumption that plane remains plane, the strains and the curvatures of the other layers can be calculated. And then the corresponding stress will be calculated through the constitutive relations of the material assigned to the layer. From the above principles, it is seen that the structural performance of the shear wall can be directly connected with the material constitutive law.

The constitutive model of the rebars is set as the perfect elasto-plastic model. Because the rebars in different directions are smeared into one layer, so if the ratios of the amounts of the distributing rebars to the concrete in the longitudinal direction and transverse direction are the same, the rebar layer can be set as isotropic. But if the ratios in the two directions are different, the rebar layer should...
be set as orthotropic with two principal axes. And in different principal axis, the stiffness is set differently according to the ratio of the amount of rebars to concrete, in order to simulate longitudinal rebars and transverse rebars respectively.

3. NONLINEAR FINITE ELEMENT PROGRAM
The NONLACS2 (NONLinear Analysis of Concrete and Steel Structures) program utilizes the basic structure of the NONLACS program [5] with the same finite element formulation and differs from the previous programs in terms of its versatility, to analyze both normal and high-strength concrete systems, to eliminate the element size effect (mesh size dependency) using both fracture mechanics and strength-based approaches, to utilize different models for concrete in compression and tension, and to determine the ultimate concrete tensile and compressive strain, \( \varepsilon_{tu} \) and \( \varepsilon_{cu} \), respectively. The program can be used to predict the nonlinear behavior of any plain, reinforced or prestressed concrete, steel, or composite concrete-steel structure that is composed of thin plate members with plane stress conditions. This includes beams, slabs (plates), shells, folded plates, box girders, shear walls, or any combination of these structural elements. Time-dependent effects such as creep and shrinkage can also be considered.

3.1. Concrete Properties
As shown in Figure 2(a), the ascending branch of the concrete uniaxial stress-strain curve up to the peak compressive strength is represented by the Saenz equation [6]:

\[
\sigma = \frac{E_0 \varepsilon}{1 + \left( \frac{E_0}{E_{sc}} - 2 \right) \left( \frac{\varepsilon}{\varepsilon_{max}} \right) + \left( \frac{\varepsilon}{\varepsilon_{max}} \right)^2}
\]

where \( E_0 \) is the initial modulus of elasticity of the concrete, \( E_{sc} \) is the secant modulus of the concrete at the peak stress, \( \sigma \) is the stress, \( \varepsilon \) is the strain, and \( \varepsilon_{max} \) is
the strain at peak stress. The descending or the strain-softening branch is idealized by the Smith and Young model [7]:

\[ \sigma = \sigma_c \left( \frac{\varepsilon}{\varepsilon_{\text{max}}} \right) \exp \left( 1 - \frac{\varepsilon}{\varepsilon_{\text{max}}} \right) \]  

(2)

where \( \sigma_c \) is the compressive strength of the concrete. For uniaxially loaded concrete, \( \sigma_c \) is equal to \( f'_{c} \). For high-strength concrete, the compressive stress-strain response is modeled using a modified form of the Popovics' equation [5].

For analysis of most plane stress problems, concrete is assumed to behave as a stress-induced orthotropic material. In this study, the orthotropic constitutive relationship developed by Darwin and Pecknold [8] is used for modelling the concrete using the smeared cracking idealization. The constitutive matrix, \( D \), is given by:

\[
D = \frac{1}{(1-\nu^2)} \begin{bmatrix}
E_1 & \nu \sqrt{E_1 E_2} & 0 \\
\nu \sqrt{E_1 E_2} & E_2 & 0 \\
0 & 0 & \frac{1}{4} (E_1 + E_2 - 2\nu \sqrt{E_1 E_2})
\end{bmatrix}
\]

(3)

in which, \( E_1 \) and \( E_2 \) are the tangent moduli in the directions of the material orthotropy, and \( \nu \) is the Poisson's ratio. The orthotropic material directions coincide with the principal stress directions for the uncracked concrete and these directions are parallel and normal to the cracks for the cracked concrete. The concept of the "equivalent uniaxial strain" developed by Darwin and Pecknold [8] is utilized to
relate the increments of stress and strain in the principal directions. Therefore, stress-strain curves, similar to the uniaxial stress-strain curves, can be used to formulate the required stress-strain curves in each principal direction.

The strength of concrete, $\sigma_c$, and the values of $E_1$, $E_2$ and $\nu$ are functions of the level of stress, and the various stress combinations. The concrete strength, when subjected to biaxial stresses is determined using the failure envelope developed by Kupfer et al. [9]. The values of $E_1$ and $E_2$ for a given stress ratio ($\alpha=\sigma_1/\sigma_2$) are found as the slopes of the $\sigma_1-\varepsilon_1$ and $\sigma_2-\varepsilon_2$ curves, respectively. For the descending branches of both compression and tension stress-strain curves, $E_i$ is set equal to a very small number, 0.0001, to avoid computational problems associated with negative or zero values for $E_i$. The concrete is considered to be crushed, when the equivalent compressive strain in the principal directions exceeds the ultimate compressive strain of the concrete, $\varepsilon_{cu}$. Two models are used for determination of the concrete ultimate compressive strain, $\varepsilon_{cu}$, for high and normal-strength concretes [10] and confined concretes [11]. For elimination of the numerical difficulties after crushing ($\varepsilon>\varepsilon_{cu}$) and cracking of the concrete ($\varepsilon>\varepsilon_{ct}$), a small amount of compressive and tensile strength as a fraction of concrete strength, $\gamma_{cf}f'_c$ and $\gamma_{tf}f'_t$, is assigned (optional) at a high level of stress [Figure 2(a)], where parameters $\gamma_c$ and $\gamma_t$ define the remaining compressive and tensile strength factors, respectively.

Cracking of the concrete is idealized using the smeared cracking model, and is assumed to occur when the principal tensile stress at a point (usually a Gauss integration point) exceeds the tensile strength of the concrete. The stiffness across the crack is assumed to be zero and the principal directions are not allowed to rotate. The aggregate interlock at the cracks and the dowel action between the reinforcing steel and the concrete are considered using the shear retention factor, $\beta$. In reality, the concrete is able to resist tension between the cracks in the direction normal to the crack; this tension-stiffening phenomenon is implemented in the algorithm by assuming the ascending and the descending branches of the tensile stress-strain curve to terminate at $\varepsilon_{cr}$ and $\varepsilon_{tu}$, respectively.

4. DEMONSTRATION CASES

In order to validate the capacity of the proposed shear wall model, three flanged shear walls were selected as the demonstration models. Pushover analysis and static cyclic loading analysis was carried out on these shear walls with the proposed shear wall model. For the shear wall, the lengths in two directions in the wall plane are both much larger than the thickness of the wall. This is much different from the beam and column members, and it will lead to bending deformations as well as shear deformations which can not usually be neglected at the same time when the wall is under lateral load in plane. These shear deformations in the wall plane have an important effect on the failure type of the wall and this complicated behavior causes the nonlinear analysis of the shear wall to become much more difficult than the beam element directly. Since, the shear span ratio of the wall is a main factor which affects the shear deformation behavior,
case 1 and case 2 will simulate the coupled in-plane bending-shear nonlinear behaviors of RC flanged shear wall with different shear span ratios respectively. And case 3 simulates the out-plane bending behaviors of RC flanged shear wall. Figure 3 shows the finite element model in case 1, and the finite element model for case 2 and case 3 are similar to case 1.

4.1. Shear Wall Case 1
The shear span ratio of the shear wall in this case is 2. For pushover analysis, the in-plane lateral load increased by step is only applied at the top of the wall. Besides, static cyclic loading process is also analyzed. In both the two analyses, the vertical load with the axial force ratio of 0.2 is applied in advance at the top of the wall. The load-displacement curve for pushover is plotted in Figure 4. From Figure 4, it can be seen that the utmost loading capacity of the shear wall is about 170kN with the displacement of about 7mm. At this time, quite a lot of concrete elements at the bottom had cracked and most tensile rebars had yielded. After that, as the crack expanded, the compressive area was getting smaller and smaller, which caused the loading capacity to drop. It can be concluded that the failure type of the wall in this case is mainly in-plane bending failure and shear deformation doesn’t play an important part in the response of the shear wall.
Displacement of the shear wall along the height at different stages in pushover is shown in Figure 5, indicating that the shape of lateral displacement is of bending type. So, the deformation behavior of shear wall structure is clearly illustrated here. Load-Displacement curve for cyclic loading is plotted in Figure 6 and the pinch effect is shown in it. This reflects the actual response characteristics of the flanged shear wall under cyclic load clearly. Besides, exterior envelope of the load-displacement curve has entered the softening part, which indicates that the microplane model can simulate the damage accumulation of flanged shear wall during the cyclic loading process precisely. This is very important for the performance-based design of structures under disaster loads.
Figure 4. Load-Displacement curve for pushover

Figure 5. Displacement along the height at different stages in pushover

Figure 6. Load-Displacement curve for cyclic loading
4.2. Shear Wall Case 2
The shear span ratio of the shear wall in this case is 1, therefore this wall belongs to the type of short wall and the in-plane shear failure always occurs in this type of walls. The load-displacement curve is shown in Figure 7 and the curve of the same relation in case 1 is also shown in Figure 7 for a comparison. It can be seen that the stiffness and the loading capacity of the wall in case 2 are much larger than that in case 1 because the shear span ratio has affected the response characteristic of the shear wall under the lateral load. In the state of maximum loading capacity, a compressive column had formed in the diagonal direction of the wall. After that, the concrete of the diagonal compressive column was crushed and quit the loading gradually, which caused the loading capacity of the wall to drop. But this process is more brittle than the descending process in case 1. This can be proved by comparing the descending part of the two curves in Figure 7.

![Figure 7. Load-Displacement curve for pushover](image)

This is a typical in-plane shear failure process of shear wall. Obviously, shear deformation plays an important part in the response of the shear wall in this case and the shear failure process has much more brittleness than the bending failure process.

In Figure 8, the pinch effect can still be seen in the load-displacement curve for cyclic loading. And similarly to Figure 6, exterior envelope of the load-displacement curve has entered the softening part because of the damage accumulation of shear wall during the cyclic loading process. But the exterior envelope of the load-displacement curve in case 2 is steeper than case 1 because the shear failure process has much more brittleness than the bending failure process.
4.3. Shear Wall Case 3

In the actual shear wall structures, the flanged shear walls are laid in both longitudinal and transverse directions. When the lateral load is applied to structure in one direction, the response of the wall with the plane in the same direction will present the coupled in-plane bending-shear nonlinear behavior just as in the above case 1 and case 2. But the wall with the plane perpendicular to the loading direction will bend out of the loading plane and present the out-plane bending behavior. This must be considered in the finite element analysis for shear wall structures.

In case 3, the geometric model is the same as in case 1. To study the out-plane bending behavior of flanged shear wall, the out-plane lateral load increased by step is applied at the top of the wall, and the vertical load with different axial force ratios is applied in advance at the top of the wall.

Figure 9 shows the relation between the lateral load and the lateral displacement at the top of the wall under different axial forces studied. Because the thickness of the shear wall is much smaller than the height, the out-plane bending behavior of the shear wall is very similar to the bending behavior of the 1-D beam element. This can be proved from Figure 9.
5. CONCLUSION

The proposed multi-layer shell element model based on the principles of composite material mechanics relates the nonlinear behaviors of the shear wall element to the constitutive relations of concrete and steel directly, and therefore it has many advantages in the description of the actual complicated nonlinear behaviors. And at the material constitution level, a novel concrete constitutive model, referred to as micro-plane model is introduced to provide a better simulation for concrete in flanged shear wall under complicated stress conditions and stress histories. The simulation results show that the multi-layer shell element model can correctly simulate the coupled in-plane/out-plane bending failure for tall flanged walls and the coupled in-plane bending-shear failure for short flanged walls. And with micro-plane concrete constitutive law, the cycle behavior and the damage accumulation of flanged shear wall can be precisely modeled, which is very important for the performance-based design of structures under earthquake loads.

REFERENCES

THEORETICAL AND EXPERIMENTAL STUDY OF UNBOUNDED POST-TENSIONED CONTINUOUS SLAB DECKS CONSISTING OF HIGH STRENGTH SCC

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ABSTRACT
Self-Consolidating Concrete, SCC is the new generation type of concrete which is not needed to be compacted by vibrator and it will be compacted by its own weight. Since SCC is a new innovation, therefore, understanding the implementation of this type of concrete on the ultimate unbounded tendon stress of post-tensioned self-consolidating concrete of bridge slabs decks (PSCCSD) is critical. For this purpose, the theoretical and experimental investigation of continuous tow span PSCCSD consisting of high strength concrete was performed. The slabs deck (L=7.5 m, b=1 m, h=0.2 m) were simulated by this concrete and the percentage of tensile and compressive steel reinforcement are in accordance with the provision of the ACI-08 for prestressed conventional (vibrating) concrete structures. During the test, the strains on concrete, steel strands and ordinary bar and deflections were measured at different locations along each span. Based on the experimental measurements, the values of experimental ultimate unbounded tendon stress \( f_{ps} \) for two tested post-tensioned SCC, continuous slabs were measured. The theoretical early and up to date works, as well as the codes recommendations for predictions of internal unbounded tendon stress at ultimate (for conventional concrete used in prestressed structures) are reviewed and their relations are used to compare with the only available experimental results of post-tensioned continuous SCC slabs of this study. It was found that the experimental results are higher than the theoretical as well as the codes prediction values suggestion for \( f_{ps} \). However, among the theoretical values suggested by different selected codes of this study, the ACI-08 values are close to the experimental values of this study.

Keywords: post-tensioned, continuous slab decks, SCC, Ultimate unbounded stress

1. INTRODUCTION
The development of SCC started in Japan (Tokyo University) in the mid 80’s with the aim of reducing durability problems in complicated and heavily reinforced concrete structures due to lack of skilled workers and poor communication between designers and construction engineers [1]. Even though conventional (vibrating) concrete previously (and still today) in some applications was cast without any
compaction, this new concrete was deliberately designed to be able to fill every corner of the form and encapsulate all reinforcement with maintained stability only under the influence of gravitational forces.

The amount of pressure on Iranian transportation infrastructure continues to increase as our growing population demands new roads and older roadways need to be replaced. Only a portion of the necessary funds are available for building new bridges and replacing deficient ones. Therefore, research in bridge design is crucial. High quality structures need to be designed and built with increasing efficiency to allow them to serve society better for a longer period of time, while leaving finances for other undertakings. This research document focuses on one specific type of bridge system; post-tensioned self consolidating concrete slabs made continuous over two spans through the use of continuity diaphragms. This type of bridge system was selected because it has many advantages (see Figure 1).

A continuous bridge is one in which two or more simple spans are connected end-to-end with continuity diaphragms. To understand the moments that develop in a continuity diaphragm, consider a simply supported system. The ends of the girder are able to rotate freely throughout the service life of the bridge from the effects of creep, shrinkage, prestress loss, live loads, temperature gradients, and other loading conditions. In a continuous system, no further end rotation is allowed after the continuity diaphragm is poured and the ends of the slabs are fixed. Restraint moments must then develop in the continuity slabs to oppose those moments that would rotate the end of the slab if it were unrestrained.

A continuous bridge has several advantages over a series of simple span structures. First, there is a reduction in mid-span bending moments and deflections. This is economical because the deck cross-section can be reduced, or fewer prestressing strands can be used in cases where the member size is fixed [2]. Secondly, making a bridge continuous will improve serviceability by eliminating joints in the deck. The removal of joints will improve the riding surface of the bridge, and durability will be increased because the water and salts from the deck will not drain onto the
substructure. Many people consider this the most important advantage [3]. In addition, the exclusion of joints in a design will reduce the initial cost of the bridge and also reduce bridge maintenance. Third, a bridge that has been made continuous will redistribute moments if the load capacity is exceeded for a particular girder in the system [2]. To determine the change in force in an unbounded post-tensioned tendon due to load on a structure, the entire structure needs to be analyzed rather than individual sections. The change in tendon strain due to applied loads should be calculated by the structure displacements. To calculate the strain in unbounded tendon the displacements of anchorage are required. The calculation of unbounded tendon strain generally requires an iterative procedure. The procedure becomes more complicated when nonlinearities due to cracking, material stress-strain relations, eccentricity changes with applied loads, and joint opening in segmental construction etc., are included in the analysis. Time dependent effects due to creep and shrinkage of concrete and relaxation of prestressed reinforcements, and the effect due to temperature gradient across the section depth are important in the analysis to predict accurate deflections, strains and stresses in concrete structures at the serviceability limit state. In addition, the contribution of the uncracked concrete to the stiffness of the structure, which is known as the tension stiffening effect, should be considered.

The studies mentioned above were useful only to find the tendon stress at the ultimate limit state. To determine the strain in unbounded tendons due to the entire loading range up to ultimate, the structural analysis has to be performed to find displacements for the given loads. This becomes difficult as the unbounded tendon stresses are not known a priori, thus iterative methods are generally required. Simplified methods are also available to determine the behavior of concrete structures with internal unbounded tendons due to service loads considering cracking. Most of these methods are however limited to beams with symmetrical loads and tendon profiles.

1.1. Theoretical Early and up to Date Work for Predictions of Internal Unbounded Tendon Stress at Ultimate

Baker [4] was among the first to propose an equation to calculate the unbounded prestressing tendon strain, $\varepsilon_{pu}$ at ultimate Eq. (1):

$$
\varepsilon_{pu} = \varepsilon_{pe} k (\Delta \varepsilon_{ps})
$$

Where $\varepsilon_{pe} =$ strain due to effective prestress which is the stress in the tendon after the effects of self weight, short and long term losses; $\Delta \varepsilon_{ps} =$ change in strain calculated for an identical but bonded tendon due to load in excess of the dead load leading to ultimate failure of the structure; $k =$ reduction factor representing the effect of absence of bond between the concrete and tendon. He suggested a value of $k$ equal to 0.1 be used in design to be on the safe side. This value was found to be very conservative [5], especially for beams with smaller span-to-depth ratios.
The ultimate tendon stress, $f_{psu}$, has been calculated by assuming linear material stress strain relation for unbounded prestressing steel, which is usually the case in practice according to many researchers, because the steel stresses generally remain within the elastic range Eq. (2):

$$f_{psu} = E_{ps} \varepsilon_{ps}$$

(2)

Where $E_{ps}$ = modulus of elasticity of prestressing steel. Several other investigators [6, 7] modified the value of $k$ based on experimental results by relating $k$ to the neutral axis depth of the critical section at ultimate. However, the determination of the neutral axis depth needs iteration as $f_{psu}$ and neutral axis depth are interdependent.

The effect of nonprestressed steel and the continuity of the concrete member have not been addressed in the above work. Therefore, more experimental studies were done to determine these effects on the value of $f_{psu}$ at ultimate, using simple and continuous members.

To calculate the stress in the prestressing steel at nominal strength, the following equation was adopted in the latest version of the ACI-Code [8] Eq. (3):

$$f'_{ps} = f_{ceu} + 0.70 + \frac{f'_{c}}{100\rho_{p}}$$

(3)

Where $\rho_{p}$ the prestressing is steel ratio; $f_{ceu}$ is the concrete compressive strength, in psi; and $f'_{ps}$ is the effective stress of the prestressing steel, in $MPa$. Eq. (1) was derived by Mattock et al. [2] Based on evaluation of experimental data of unbounded members tested before 1971. Using the test results of post-tensioned unbounded slabs, supported with an analytical truss model, Mojtahedi and Gamble [9] concluded that increasing the span-to-depth ratio of unbounded members reduces the stress increase in the prestressing steel. Based on their findings, the ACI Building Code, in its 1983 version, limited the use of Eq. (3) to members with span-to-depth ratios of 35.

Two major drawbacks of the ACI Building Code equations are: 1) they neglect the effect of bonded tension reinforcement; and, more importantly, 2) they do not consider the effect of multispans systems or loading pattern in continuous members, which has a major influence on the stress in unbounded tendons. In 1976, Tam and Pannell [10] accounted for the effect of bonded tension reinforcement using the following Eq. (4).

$$f'_{ps} = f_{ceu} + E_{ps} \varepsilon_{cu} \left( \frac{d_{cu} - c}{L} \right)$$

(4)

In which the neutral axis depth $c$ is extrapolated in accordance with Tam and
Pannell’s approach as follows Eq. (5).

\[
\tau = \frac{f_{pe} + 10.5 \varepsilon_{cu} \frac{d_p}{L} A_{pz} + f_{cz} - A_{pz} f_{cz}^t - 0.35 f' c (b - b_w) k_f}{0.05 \varepsilon_{cu} A_{pz} + 10.5 \sigma_{ps} A_{pz} / L}
\]  

(5)

where \( A_t \) and \( A_c \) are the area of bonded tension and compression steel, respectively; \( E_{ps} \) is the modulus of elasticity of the prestressing steel; \( \varepsilon_{cu} \) is the ultimate concrete compression strain; \( L \) is the span length; and \( \psi \) is the ratio of the equivalent plastic hinge length \( L_p \) to the neutral axis depth \( c \). For rectangular sections or flanged sections with rectangular section behavior, \( b_w = b \). Based on their own test results of simply supported members loaded with single concentrated load at midspan, Tam and Pannell [10] recommended the use of a value of \( \psi = L_p / c = 10.5 \). Using a value of \( E_{ps} = 2 \times 10^5 \) MPa, \( \varepsilon_{cu} = 0.003 \) and \( \psi = 10.5 \) leads to a value of \( E_{ps} \varepsilon_{cu} \psi = 6000 \) MPa. Note that Eq. (4) does not take into account the effect of continuous members. In 1991, based on the results of analytical studies [11, 12] and test data of internally unbounded members, Harajli and Kanj [13] proposed the following lower bound design equation taking into consideration the effect of bonded tension reinforcement, member span-to-depth ratio and loading pattern in continuous members Eq. (6):

\[
f_{ps} = f_{pe} + \gamma_c f_{ps} \left[ 1 - \frac{A_{pz} f_{pe} + A_{pz} f_{cz}^t}{b d_p f' c} \right]
\]  

(6)

In which the term \( (A_{pz} f_{pe} + A_{pz} f_{cz}^t) / b d_p f' c \) need not be taken greater than 0.23, and \( \gamma_c \) is a combined member span-to-depth ratio and continuity coefficient given as Eq. (7):

\[
\gamma_c = n_o \frac{0.12 + \frac{2.5}{L / d_p}}{n}
\]  

(7)

where \( n_o \) is the number of loaded spans required to form a collapse mechanism, and \( n \) is the total number of spans between anchorages. In principle, the value of \( n_o \) is equal to unity for evaluating the stress at the section of maximum positive moment, and \( n_o = 2 \) for evaluating the stress at the section of maximum negative moment at an interior support. In its 1998 version, however, the same code introduced a totally new equation that was also adopted by the AASHTO Guide Specification [14] Eq. 8, 9.
For rectangular section behavior, \( \beta_w = b; l_e \) and \( l_t \) are the effective tendon length and length of tendon between anchorages, respectively; and \( N_s \) = number of support hinges required to form a mechanism crossed by the tendon. In its 2005 version of AASHTO LRFD [15] introduced a total Eq. (10):

\[
f_{ps} = f_{pe} + 6300 \left( \frac{d_p - c}{l_e} \right) \leq f_y \quad \text{and} \quad l_e = \left( \frac{2l_t}{2 + N_s} \right)
\]  

(10)

In 1991, Naaman and Alkhairi [16] proposed the following expression Eq. (11):

\[
f_{ps} = f_{pe} + 6300 \left( \frac{d_p - c}{l_e} \right) \leq f_y \quad \text{and} \quad l_e = \left( \frac{2l_t}{2 + N_s} \right)
\]  

(11)

Where \( \Omega_W \) is a bond reduction factor, expressed as \( \Omega_W = k/ (L/d_p) \), and \( L_1/L_2 = \text{the ratio of the length of loaded span(s) in continuous members to the total length of tendon between anchorages (see Figure 2).} \)

\[ \text{Continuity Coefficient: (1+N_s/2)/n; L_1/L_2; n_c/n} \]

The equation given in the Canadian code [17], Eq. (12) to predict unbounded tendon stress at ultimate accounts indirectly for nonprestressed steel area, \( f'c \), and span-to-depth ratio. This equation was derived assuming formations of plastic hinges at ultimate limit state (Loov, 1987):

\[
f_{ps} = f_{pe} + 8000 \left( \frac{d_p - c}{l_e} \right) \leq f_y
\]  

(12)

British code (BS 8110, 1997)

The equation given in the British Code [18], Eq. (13)
\[ f_{pb} = f_{pe} + \frac{7000}{f'_{ck}} \left( 1 - 1.17 \frac{f'_{ck}}{f_{pe} d} \right) \] \[ x = 2.47 \left( \frac{f'_{ck}}{f_{pe} d} \right) \frac{d}{f_{pe}} \]  

where, \( b_0 \): width or effective width of the section or flange in the compression zone, \( f_{pe} \): design effective prestress in the tendons after all losses, \( f'_{ck} \): characteristic strength of concrete, \( f_{pu} \): specified tensile strength of prestressing steel, \( A_{pe} \): area of prestressing tendons in the tension zone, \( f_{pb} \): design tensile stress in the tendons, \( d \): effective depth to the centroid of the steel area, \( x \): depth of the neutral axis.

1.2. Analysis of Initial Effects Due to Prestress and Self-Weight

Initial analysis of the effects due to prestress and self-weight are considered in two parts. In the first part, the effective tendon forces are applied to the member as an external load effect. Section strains at any location along the member are determined from section equilibrium. In the second part, the self-weight is applied to the structure, also as a load effect with strains again determined at discrete sections. Initial strain from the analysis of the tendon forces alone are considered in the equilibrium calculations (see Figure 3).

![Figure 3. Initial strains at three sections along continues slab with prestress forces acting alone](image)

1.3. Application of External Loads to the Member

The first application of external loads occurs in the analysis with the application of the cracking load. It is assumed that the member first cracks at the maximum moment section when the tensile capacity of concrete is exceeded. The corresponding cracking moment defines the magnitude of the bending moment distribution and hence the load level. Initially the structure is assumed to remain uncracked with the cracking load applied to attain a snapshot of the beam behavior just prior to cracking is obtained (Figure 4). It is reminded that in all above reports the conventional (vibrated) concrete were used to find the ultimate unbounded tendon stress, \( f_{pu} \), and therefore argent research is needed to find out the value of \( f_{pu} \) while using SCC (non vibrating concrete) in post-tensioned concrete structures. Although, the use of SCC in simple post-tensioned concrete slab is reported but no work was observed for this type of concrete while using in continuous post-tensioned deck slabs constructions.
1.4. Experimental Program

Two continuous unbounded post-tensioned self-consolidating concrete slab tests, PSCC slab1 and PSCC slab2 were conducted. As yet there is no any standard for SCC used in prestressed concrete, the post-tensioned slabs were designed according to ACI 318-08 [8] for conventional (vibrating) concrete. The tendons used were strand, supplied by Bridon Ltd. The overall dimensions of the continuous slabs were identical with an overall length of 7500 mm, each span length of 3500 mm, a width of 1000 mm, and a depth of 200 mm. PSCC slab1 and PSCC slab2 had four longitudinal parabolic tendons with a nominal diameter of 11.11 mm and area of 74.54 mm$^2$ (see latter in Figure 6). Each tendon was supported on seven steel chairs to maintain the variable designed eccentricity of the tendons. At the mid-span and central support of the slabs the height of eccentricity was 30 mm. The heights of the chairs were interpolated considering the tendon curvature as a second degree parabola. Each tendon passes through the dead and live anchorages that were identical and supplied by CCL Ltd. The general layout of the unbounded post-tensioned one-way SCC slabs is shown in Figure 5. The post-tensioning was carried out by the first author by elongating the strands, to a specific load, from the live anchor using a hydraulic jack after the SCC gained a minimum strength of 420 MPa. The live end was locked using wedges with the load transferred to the concrete. The non-jacked end of the strand was prelocked using wedges at the dead anchor. Bursting reinforcement was designed, according to ACI 318-08 [8], to resist tensile bursting forces around individual anchorages. Seven transverse, 8 mm diameter, ordinary reinforcement steel bars having a nominal yield stress of 500 MPa were positioned at each end of the slab adjacent to the dead and live anchorages. The bars were tied using ten 12 mm diameter closed links. The results of fresh and hardened SCC used are presented in Table 1. However, further details of the tested specimens are given in reference [19].
Table 1: Fresh and hardened concrete characteristics specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Average compressive cube strength at transfer, $f_{ci}$ (Mpa)</th>
<th>Average compressive cube strength at 28 days, $f_{cu}$ (Mpa)</th>
<th>L-box</th>
<th>J-ring</th>
<th>V-funnel</th>
<th>Slump flow</th>
</tr>
</thead>
<tbody>
<tr>
<td>SCC slab1</td>
<td>59.0</td>
<td>67.1</td>
<td>0.36</td>
<td>0.83</td>
<td>76</td>
<td>68</td>
</tr>
<tr>
<td>SCC slab2</td>
<td>61.0</td>
<td>68.0</td>
<td>0.37</td>
<td>0.89</td>
<td>77</td>
<td>74</td>
</tr>
</tbody>
</table>

1.5. Material Properties
Tensile tests were carried out to determine the material properties of the ordinary bars and the tendon. The nominal ultimate tensile strength of the tendon, was 2005 MPa, corresponding to a nominal breaking load of 150 kN. The nominal yield load was specified as 110 kN. The strain gauges were fixed onto the strands and the reinforcement bars (by the first author) before concrete casting. Four displacement transducers LVDTs were used to measure the deflection at the mid span of the slabs and under load. A data logger system was used to record the load and strain at regular intervals during the slabs test. The strains in the tendons were recorded during the post-tensioning process.

1.6. Test Set Up and Experimental and Theoretical Comparison
The loading arrangements are shown in Figure 6 for testing the two spans post-tensioned slabs. A 1400kN pushing capacity jack applied the load, which was measured and controlled by different load cells. The slabs were loaded at tow locations using spreader plates 1000×30×30 mm, as shown in Figure 6. A data logger system was used to record the applied load, the strains in the tendons and concrete surface, and the vertical deflections. The load-tendon strains are plotted and shown in Figure 7. Also the values of experimental ultimate unbounded tendon stress, $f_{pu}$ for two tested post-tensioned SCC slabs are measured and compared with the theoretical values suggested by different researchers (Table 2).
2. CONCLUSIONS

The values of experimental ultimate unbounded tendon stress, \( f_{ps} \), for two tested post-tensioned SCC (none vibrating), continuous slabs were measured and compared with the available theoretical values suggested by different codes and researchers for conventional (vibrating) concretes. Although no vibrating is used in SCC, but for two tested specimens the obtained results are indicated that the
experimental values of $f_{pu}$ are higher than the five codes suggested value and close to the ACI-08 value. Also, the obtained experimental values of $f_{pn}$ are higher than the two theoretical values suggested by Tam and Panell, and Harajli and Kanj, and close to the value suggested by Haragli and Kanj. However, comparing the theoretical and five selected codes value, it was found that the theoretical suggested value by Haragli and Kanj are close to the code value provided by ACI-08. It was also found that, no change in $f_{pu}$ values are obtained for either of the slabs tested while compared to the theoretical and codes values, except for the values obtained by Haragli and Kanj which are similar to the experimental values and are different for either of the slabs. This is because in this method, the amount of ordinary tensile and compression reinforcement used are taken into consideration.

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THE FINITE ELEMENT ANALYSIS OF RC JOINTS STRENGTHENED WITH EXTERNAL FRP COMPOSITES

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2M.Sc Student, Faculty of Civil Engineering, Semnan University, Semnan, Iran

ABSTRACT
Externally bonded fiber-reinforced-polymer (FRP) sheets have been successfully used for strengthening of damaged or deficient reinforced concrete members. Despite of a lot of research conducted and tests on application of these new sheets during the last decade, further research is still required to consolidate recent developments and expand the scope of application of FRPs for structural applications. Nonlinear finite element analysis combined with laboratory testing constitutes an efficient approach for pursuing this objective.

The objective of this paper is exploring and illustrating the contribution of a refined three-dimensional (3D) constitutive FE model for investigating the nonlinear response of concrete joint, reinforced with steel rebars and strengthened with external FRP sheets. The analyses were carried out by using finite element software having different capacities. Different parameters such as application of FRP sheets with different patterns, different loading conditions and different strengthened areas have been considered to show the results. Several results regarding increasing ultimate values in the strengthened model in comparison with the reference specimen, ductility of the strengthened model, and evaluation of ductile against non-ductile joint have been presented in this paper.

Keywords: FE model, nonlinear analysis, RC joint members, FRP sheets, strengthening

1. INTRODUCTION
Existing reinforced concrete (RC) structures that were designed according to pre-1970’s codes often have inadequate reinforcement detailing, which not only results in deficient lateral load resistance, but also in insufficient energy dissipation, rapid strength deterioration and improper hinging mechanisms during earthquakes, leading to excessive drifts and ultimately to structural collapse. Non-ductile detailing is generally manifested through deficient joint shear resistance, deficient column shear capacity, deficient column’s main reinforcement lap splices, deficient anchorage of beam positive reinforcement at the beam-column joint, and deficient beam shear resistance. In particular, recent earthquakes have demonstrated that RC beam-column joints that have been constructed based on pre-1970’s design codes may initiate and cause total collapse of structures. For instance, Figure 1-a [1] shows a RC structure that collapsed during the 1999 Kocaeli Earthquake in Turkey
in which joints failures appear to be the major contributor to such collapse, while Figure 1-b shows a close-up of a non-ductile failure of a beam-column joint during the same earthquake. Beam-column joint deficiencies combined with the weak column/strong beam glitch contradict failure hierarchy of the design capacity concept. A failure in the beam is usually less critical than that in the column, and the latter is less critical than a failure in the joint. Hinging in the joint, being at the point of intersection of the beam and column, allows excessive rotations both in the beam and column in conjunction with a loss of load carrying capacity of the column. Such dangerous failure mechanism is unacceptable and must be prevented in design.

![Figure 1](image)

**Figure 1. Damages to moment resisting frames during the Kocaeli 1999 earthquake:**
(a) joint induced structural collapse; (b) beam-column joint failure [1]

There is a perceived void in the current literature for studies that focus on the behavior of reinforced concrete beam–column joints under cyclic loading. In fact, most reported research in the literature is mainly on cyclic behavior of connections [3] in newly designed steel structures and also concrete connections retrofitted by traditional rehabilitation techniques [2]. Moreover; most of the recent researches involved in finite element modeling of RC connections; are concerned with exterior beam-column joints.

This study intends to investigate the effect of various combinations of FRP wrapping patterns on the performance of interior reinforced concrete beam–column joints, i.e. ductility, under combined axial and lateral cyclic loads. A three-dimensional finite element analysis model of FRP wrapped beam–column joints, which exhibit material and geometric nonlinearities that are due to large displacements, confinement effect, and concrete nonlinear behavior, are developed. The FEA model is validated through leveraging an experimental study on a FRP-wrapped beam–column joint.

2. EXPERIMENTAL TEST ON INTERIOR RC BEAM-COLUMN JOINTS

There are abundant experimental tests regarding RC beam-column joints, and most
of them have been carried out on exterior beam-column joints. In addition; limited 
researches have been investigated on finite element modeling of interior beam-
column joints. A. Mukherjee et al [5] have performed a fully detailed, 
comprehensive experimental test on interior RC beam-column joints strengthened 
with FRP laminates. The test scheme and specimen which have been investigated 
in the test are firstly introduced. Eventually, the FE model calibrated regarding the 
experimental study is presented and results are analyzed.

2.1. Specimens Details
Two different types of RC joints have been cast for experimental verification [5]. 
One set of joints has adequate steel reinforcements with proper detailing of 
reinforcements at the critical sections (Figure 2). In the other set of specimens the 
beam reinforcements have deficient bond lengths at the junctions with the columns 
(Figure 3). When the beam was transversely loaded the first set was characterized 
by a long plastic zone (ductile) while the second set failed in reinforcement pull out 
and exhibited sudden failure (non-ductile).

Figure 2. Specimen with ductile joint reinforcement [5]

Figure 3. Specimen with non-ductile joint reinforcement [5]

The specimens in Figures (2) and (3) were strengthened using carbon and glass 
FRP materials. Prior to the application of the FRP, the concrete substrate was
smoothed by grinding. Figures (4) and (5) present schematic arrangement for two typical systems; L-overlays and precured carbon plates which were utilized respectively, in the aforementioned experiment. In Type A; GFRP/CFRP sheets have been applied in L shape to upgrade the joints. These sheets have been applied in several layers. FRP has been applied on the top and bottom surface of concrete surfaces, so the fibers were along the axes of the members (Figure 4a). Then, FRP wraps were provided over the inner layers (Figure 4b), the direction of fibers in wraps was perpendicular to the axis of the members. Figure 5a shows glass fiber sheets (80mm wide and 250mm long) on either side of the joint. Only one layer is provided on one side. Two layers of FRP have been provided on the other side to evaluate its efficacy.

![Figure 4](image1.png)

**Figure 4. Type a strengthening system-use of composite overlays [5]**

Both the column and the beam are then wrapped by unidirectional glass fibers with 100mm lap length. Same configuration is repeated using carbon fiber sheet using 1 and 2 layers of overlays and single wrap with 100mm overlap. Both adequate and deficient joints were reinforced using this configuration. Furthermore, procured carbon plate (25mm wide and 1.2mm thick) have been used in the beams in Type B to improve bending stiffness. To achieve a good bond between the plate and the concrete, a groove (25mm wide and 25mm deep) has been created inside the joint. The plates have been inserted into the joint as shown in Figure 5, and then the groove has been filled by injecting epoxy resin, and the
plates have been inserted in the groove as shown in step 1 of Figure 5. The beams and columns have then been wrapped using a single wrap of carbon sheet.

2.2. Test Program
The experimental setup which has been utilized by A. Mukherjee et al [5] is shown in Figure 6. The column was fixed at its ends on a loading frame. It was subjected to a constant axial load of 100KN which is 50% of ultimate load carrying capacity of the column. Cyclic load was applied using a hydraulic actuator with load cycle based on increasing displacement control. Three cycles were repeated at each level of displacement. Vertical deflection of the tip of the beam was recorded directly by the linear variable displacement transducer (LVDT). The compressive strength of concrete used in this experiment has been 30N/mm² and also properties of other material used are shown in Table (1). The same values have also been used for FE modeling of aforementioned specimens. Totally, 12 specimens at two categories of ductile and non-ductile reinforcement, including as-built and strengthened specimens with different patterns have been tested in this experimental research by [5].

![Figure 6. Experimental setup [5]](image-url)

The elaborate test matrixes for adequate and deficient specimens which have been investigated through this experiment are presented in Tables (2) and (3) respectively.

**Table 1: Properties of materials [5]**

<table>
<thead>
<tr>
<th>Material</th>
<th>Effective thickness (mm)</th>
<th>Ultimate strength (MPa)</th>
<th>Tensile modulus (GPa)</th>
<th>Ultimate strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>Glass-G (fiber)</td>
<td>0.36</td>
<td>2250</td>
<td>70</td>
<td>0.0239</td>
</tr>
<tr>
<td>Carbon-C (fiber)</td>
<td>0.11</td>
<td>3500</td>
<td>230</td>
<td>0.0117</td>
</tr>
<tr>
<td>Carbon plate-CP (composite)</td>
<td>1.2</td>
<td>2800</td>
<td>165</td>
<td>0.017</td>
</tr>
<tr>
<td>Mild steel longitudinal</td>
<td>6 mm dia</td>
<td>275</td>
<td>198</td>
<td>0.045</td>
</tr>
<tr>
<td>reinforcement</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mild steel transverse</td>
<td>3 mm dia</td>
<td>555.13</td>
<td>193</td>
<td>0.043</td>
</tr>
<tr>
<td>reinforcement</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Table 2: Test matrix for ductile specimen [5]

<table>
<thead>
<tr>
<th>S. no</th>
<th>Specimen name</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>D-1</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>G1L-D</td>
<td>Type A with single L of GFRP at top and bottom</td>
</tr>
<tr>
<td>3</td>
<td>G2L-D</td>
<td>Type A with two L of GFRP at top and bottom</td>
</tr>
<tr>
<td>4</td>
<td>C1L-D</td>
<td>Type A with single L of CFRP at top and bottom</td>
</tr>
<tr>
<td>5</td>
<td>C2L-D</td>
<td>Type A with two L of CFRP at top and bottom</td>
</tr>
<tr>
<td>6</td>
<td>CP1-D</td>
<td>Type B with CFRP plate at top and bottom</td>
</tr>
</tbody>
</table>

Table 3: Test matrix for non-ductile specimen [5]

<table>
<thead>
<tr>
<th>S. no</th>
<th>Specimen name</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>ND-1</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>G1L-ND</td>
<td>Type A with single L of GFRP at top and bottom</td>
</tr>
<tr>
<td>3</td>
<td>G2L-ND</td>
<td>Type A with two L of GFRP at top and bottom</td>
</tr>
<tr>
<td>4</td>
<td>C1L-ND</td>
<td>Type A with single L of CFRP at top and bottom</td>
</tr>
<tr>
<td>5</td>
<td>C2L-ND</td>
<td>Type A with two L of CFRP at top and bottom</td>
</tr>
<tr>
<td>6</td>
<td>CP1-ND</td>
<td>Type B with CFRP plate at top and bottom</td>
</tr>
</tbody>
</table>

3. FINITE ELEMENT MODELING OF INTERIOR RC BEAM-COLUMN JOINTS

All specimens which have been investigated in the experimental study conducted by Mukherjee [5] are modeled in this section by using non-linear finite element ANSYS ver 11.

3.1. Material Models

The constitutive relationships employed to describe the mechanical behavior of materials as well as the interaction between steel bars and concrete are basically those proposed in CEB-FIP Model Code 1990 [4], with some slight modifications. In compression, the behavior of the concrete is that proposed by the same code and, in tension, a linear elastic behavior is assumed up to the strength of concrete in tension ($f_{ct}$). For the sake of comparison, a second model that indirectly incorporates the tension-stiffening effect [6] is also implemented. In such a model, the progressive loss of rigidity after cracking is quantified indirectly through an adaptation of the tension behavior introducing a softening branch, which is calibrated using the $\alpha$ and $\varepsilon_m$ parameters. Both the curves are illustrated in Figure 7. The aforementioned parameters are usually set at $0.5 \leq \alpha \leq 0.7$ and $\varepsilon_m = 0.0020$. In this case, fracture mechanics could be used to establish these values, based on energy criteria [8].

The perfect plasticity model of the behavior of the longitudinal reinforcement bars and also the interaction between reinforcement bars and concrete are shown in...
Figures (8) and (9), respectively. The parameters shown in Figure 9 depend on the bond conditions and confinement of concrete, as established in CEB-FIP Model Code 1990 [4].

Solid 65, Solid46 and Link8 are the element used in ANSYS to develop these FE models. The Solid65 and Link8 elements were used to model the concrete and reinforcement, and also layered solid elements, Solid46, were used to model the FRP composites.

3.2. Bond Model
The correct simulation of the bond between concrete and reinforcement bars plays
a significant role in the proper modeling of beam-column connections. When the bond forces tend to zero it is apparent that the majority of the shear force will be transferred across the joint core by a diagonal compression strut mechanism and hence severe diagonal tension cracking is less likely if bond deterioration occurs at an early stage of loading [9]. A complex interaction between flexural response of the adjacent beam element and the joint shear transfer mechanism occurs also due to the stress penetration into the panel zone from the beam bars, combined with a fixed-end rotation in the beam due to progressive bond degradation and pull out mechanism. The discrete bond model implemented in ANSYS consists of a one-dimensional (1D) finite element with a realistic bond-slip relationship as shown in Figure 10. Additional information on the discrete bond model can be found in [7].

For plain round bars with a diameter of 12 mm, the total bond strength was approximately $\tau_m + \tau_f = 1\text{ MPa}$ ($\tau_m = \text{mechanical bond}$; $\tau_f = \text{frictional bond}$) for a slip of $s_1 = 0.03\text{ mm}$ [10]. During cycling, the bond degradation valid for deformed bars is principally due to the shear failure of concrete between the ribs of the bar. In the case of smooth bars, it is reasonable to assume that friction is the only source of bond mechanism at the steel-concrete interface and that it is scarcely influenced by the cycling.

3.3. Analytical Modeling of Specimens
Two different finite element models for each of the specimen in two different categories of ductile and non-ductile were analyzed. Albeit, neither of the experimental specimen had been investigated for monotonic loading, but in order to acquire the ductility and load-displacement curves for specimens; all the FE models in this analyses went under monotonic loading for non-linear analysis. A relatively fine discretization was employed for monotonic loading as Figure 11a. On the other hand, for saving computation time, cyclic analyses were carried out using a relatively coarse discretization shown in Figure 11b. The FE model of steel bars is shown in Figure 11c.
The experimental control specimen of ductile category (D-1) was firstly modeled and then went under both monotonic and cyclic loading to assess and validate the accuracy of FE model. Figure 12 compares the applied force versus free tip of the beam drift curves for monotonic loading of the fine and coarse models with the envelope curve from the cyclic experiments for ductile specimen. It can be seen that the numerical results agree reasonably well with the experimental results. The coarse model, however, slightly overestimated the peak resistance and exhibited slightly more brittle response. For both models the failure mode was diagonal shear failure of the joint. This point confirms the accuracy of FE model; therefore FE model is extended to acquire further results.

Non-linear finite element analyses for the entire models including ductile and non-ductile; has been implemented by utilization of the two different patterns of strengthening Type A & B.

The usage of two different types of strengthening which has been performed for both ductile and non-ductile FE models is illustrated in Figure 13.
3.4. Results for FE Models with Ductile Joint Reinforcement

The displacement levels of the first few cycles do not generate any nonlinear deformation in the model. The onset of stiffness degradation is identified by simultaneous appearance of tension cracks at the root of the cantilever beam. The analyses show that at this point the steel started to yield and it was not capable of taking any further load. The additional load from this point was carried out by the FRP. At this point, linearity of the ascending and the descending paths is lost. This phenomenon is yield point.

The post yield behavior is signified by monotonic degradation of stiffness. Ability of the structure to survive an earthquake depends to a large extent, on its ability to dissipate the input energy. Forms of energy dissipation include kinetic energy, viscous damping and hysteretic damping, etc. An estimate of the hysteretic damping can be found by the area enclosed in the load–displacement hysteresis loops. Yield points for ductile specimen are provided in Table (4). Columns 2 & 3 of Table (4) summarize the percentage increase in the yield load. The CP1-D exhibited the highest increase in the yield load followed by the C2L-D, G2L-D, C1L-D and G1L-D specimens. It may be noted that the forces at the tensile face of the beam are shared by the steel and FRP in proportion of their relative stiffnesses. The stiffness of carbon is considerably higher than that of glass. Therefore, for the same tip load, the tensile force in steel is lower in the carbon reinforced FE model than in the glass reinforced models. As a result, the steel in the carbon reinforced models yield at higher tip loads. The CP1-D models are anchored at the joint through a groove. Therefore, they exhibit higher stiffness than other sheet models. The models with two-layer reinforcement had higher yield loads than the models with one layer reinforcements. Due to FRP reinforcements the displacement at yield increased to a much lesser extent than the load (Comparison of column 2 (or 3) with 4 (or 5), Table 4). Another interesting point is that the glass reinforced models had much higher displacement at yield than the carbon reinforced models. This is due to the higher stiffness of carbon than glass. There is satisfactory agreement between FE model and experimental test results. The initial stiffness and the ultimate displacements are also summarized in Table (5).

It is worthwhile to mention that almost all the values of increase and promotion for
FE models are higher in comparison with experimental results. This is due to the reduction of degrees of freedom in analytical finite element model in comparison with the real specimen. FE models are inherently stiffer than real specimens. Figure 14 reveals the ratio of ductility for strengthened specimen versus control specimen. Apparently ductility for all strengthened FE ductile models has increased between 25 to 78%. The joint shear crack, which ultimately caused the model to fail, was similar to the shear cracks observed in the experiments.

Table 4: Yield points of ductile specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>% increase</th>
<th>% increase</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>experiment</td>
<td>FE model</td>
</tr>
<tr>
<td>Control-D</td>
<td></td>
<td></td>
</tr>
<tr>
<td>G1L-D</td>
<td>21.32</td>
<td>23.48</td>
</tr>
<tr>
<td>G2L-D</td>
<td>48.42</td>
<td>51.12</td>
</tr>
<tr>
<td>C2L-D</td>
<td>57.89</td>
<td>59.60</td>
</tr>
<tr>
<td>CP1-D</td>
<td>116.18</td>
<td>112.10</td>
</tr>
</tbody>
</table>

Table 5: Ultimate points in ductile specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>% increase</th>
<th>% increase</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>experiment</td>
<td>FE model</td>
</tr>
<tr>
<td>Control-D</td>
<td></td>
<td></td>
</tr>
<tr>
<td>G1L-D</td>
<td>17.14</td>
<td>18.25</td>
</tr>
<tr>
<td>G2L-D</td>
<td>75.00</td>
<td>73.28</td>
</tr>
<tr>
<td>C2L-D</td>
<td>140.94</td>
<td>138.25</td>
</tr>
<tr>
<td>CP1-D</td>
<td>41.37</td>
<td>45.95</td>
</tr>
</tbody>
</table>

Unfortunately, a direct comparison of the sequence of cracking (flexural to shear) was not possible since monotonic loading of the test specimens was not performed. The finite element model for monotonic loading (fine mesh) was also used to investigate: 1) the influence of the bond strength (Figure 15a) and 2) the influence of the normal column force (Figure 15b). Figure 15 shows the influence of the ultimate bond strength ($\tau_m + \tau_f$) on the response of the joint. It can be seen that with higher bond strength the resistance is higher and the failure more brittle. Figure 15b shows the influence of the axial column force on the applied force versus free tip of beam drift curve. It can be seen that with higher compressive force the joint shear (thus overall subassembly) resistance increases.

The time that analyses finished and elements turned to fail, pondering through strain distribution in the control model revealed that the beam has failed at the joint through the formation of a hinge. The hinge has formed between the two shear links of the beam. It seems concrete has spalled in such a fashion that two semicircular surfaces have been created. The FRP reinforced models, on the other
hand, did not have the semicircular failure planes. The failure planes were approximately vertical. It could be concluded that the difference in the failure mode is due to the presence of the FRP wraps.

![Graph showing ductility for strengthened ductile models](image)

**Figure 14.** Ductility for Strengthened ductile models

![Comparison of numerical and experimental results for monotonic loading](image)

**Figure 15.** Comparison of numerical and experimental results for monotonic loading:
(a) effect of variation of bond strength; (b) effect of variation of axial load

3.5. Results for FE Models with Non-Ductile Joint Reinforcement

Non-linear analyses show that to some extent the extracted results from non-ductile FE models are close to ductile FE model. However due to the presence of continuous steel bars in the joint area, ductile model have a higher load bearing capacity, stiffness and energy dissipation capability. In Tables (6) & (7) the yield and ultimate points for non-ductile models are given. The G2L-ND exhibited the highest increase in the yield load followed by the C2L-ND, CP1-ND and C1L-ND. The models with two-layer reinforcement had higher yield loads than the models with one layer. Due to FRP usage the displacement at yield increased to a much lesser extent than the load (Comparison of column 2 (or 3) with 4 (or 5), Table 6).
Table 6: Yield points of non-ductile specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Yield load</th>
<th>Deflection at yield load</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>% increase</td>
<td>% increase</td>
</tr>
<tr>
<td>Control-ND</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>G2L-ND</td>
<td>103.75</td>
<td>42.56</td>
</tr>
<tr>
<td>C1L-ND</td>
<td>12.60</td>
<td>13.25</td>
</tr>
<tr>
<td>C2L-ND</td>
<td>79.36</td>
<td>36.45</td>
</tr>
<tr>
<td>CP1-ND</td>
<td>68.36</td>
<td>-45.25</td>
</tr>
</tbody>
</table>

Table 7: Ultimate points in non-ductile specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Initial stiffness</th>
<th>Ultimate deflection</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>% increase</td>
<td>% increase</td>
</tr>
<tr>
<td>Control-ND</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>G1L-ND</td>
<td>41.46</td>
<td>42.56</td>
</tr>
<tr>
<td>G2L-ND</td>
<td>9.52</td>
<td>13.25</td>
</tr>
<tr>
<td>C2L-ND</td>
<td>32.49</td>
<td>36.45</td>
</tr>
<tr>
<td>CP1-ND</td>
<td>-41.17</td>
<td>-45.25</td>
</tr>
</tbody>
</table>

The load-displacement envelopes for ductile and non-ductile FE models are plotted in Figures (16-17). The envelopes let us compare the relative performance of the models. All the FRP reinforced models have higher peak loads than the control model. For ductile joints the CP1 model has the highest peak load followed by the C2L, G2L and G1L. For non-ductile joints the G2L-ND model has the largest envelope area followed by the C2L-ND, CP1-ND and C1L-ND. Comparing these Figures reveal the superior performance of ductile joints.

Ductility promotion for non-ductile joints is approximately between 16 to 51% (Figure 18).
4. CONCLUSION

With application of FE models (validated based on experiment) for two different types of RC joints including ductile and non-ductile reinforcement details the promotional effect of both glass and carbon composite has been investigated. These two composites could be efficiently used for seismic retrofitting of RC joints regardless of reinforcement details. Obviously due to presence of continuous steel bars in the joint area for ductile joints, they exhibit a more superior, ductile behavior rather than non-ductile joints. The main cause of superior performance of the FRP reinforced joints is the continuous confinement provided by the FRP wraps which impede the creation of hinge through the spalling of concrete. FE models confirm the advantage of carbon reinforcements over glass reinforcement in case of ductile joints. But for non-ductile joints, glass reinforcing is preferable. Utilization of FRP sheets have a promotional efficiency regarding to yield load, performance and initial stiffness of joints. Commonly CFRP strengthened joints reveal stiffer behavior than GFRP strengthened joints regardless of reinforcing details.

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3D FINITE ELEMENT MODELLING OF BOND-SLIP BETWEEN REBAR AND CONCRETE IN PULL-OUT TEST

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ABSTRACT
A reinforced concrete material is a composite material made up of two components with unequal mechanical behaviour and physical features. In general, the external load is already applied to concrete and the reinforcing bars receive its part of the load only from the surrounding concrete by bond. In composite structures, the bond between different components of reinforced concrete member has a primordial role and its negligence conducted to poor structural response. Therefore, for modeling of reinforced concrete structures one needs a simple and realistic bond-slip model. There are various finite element models for bond-slip relationship between reinforcement and concrete. In this paper, modeling of the transition region between steel and concrete as a cohesion layer in the finite element program (Ansys) is discussed. A 3D finite element model to represent this layer has been introduced. The layer involves modeling the ribs and effects of slip and bond stress of the bar. The accuracy of the models is assessed by comparison of the finite element numerical response with experimental data from pullout test.

Keywords: pull-out test, finite element, bond-slip relationship

1. INTRODUCTION
A reinforced concrete (RC) structure is a composite structure made up of two materials with different characteristics, namely, concrete and steel. In general, the external load is already applied to concrete and the reinforcing bars receive its part of the load only from the surrounding concrete by bond. “Bond stress” is the name assigned to the shear stress at the bar-concrete interface which, by transferring load between the bar and the surrounding concrete, modifies the steel stresses. This bond, when efficiently developed, enables the two materials to form a composite structure. In composite structures, the bond between different components of reinforced concrete member has a primordial role and its negligence is conducted to poor structural response. These complex phenomena have led engineers in the past to rely heavily on empirical formulas for the design of concrete structures, which were derived from numerous experiments. For these reasons, the incorporation of bond is carried out considerably in recent works. The properties of this interaction depend on several factors, such as friction, mechanical interaction and chemical adhesion [1, 2].
In the past, a number of experimental investigations have been carried out in order to clarify and understand the behaviour of deformed bars pulled out from a concrete block under monotonic and cyclic loading conditions. These experimental results are well documented in the specific literature [9]. Based only on the experimental results it is difficult to filter out the influences of material and geometrical parameters on the bond behaviour. Therefore, to better understand the bond behaviour, a reliable bond model (simulation of the transmission of forces in the bond zone, see Figure 1a) that can be employed in a three-dimensional finite element, an analysis is needed. The numerical modelling of the bond behaviour is principally possible at two different levels: (1) detailed modelling (see Figure 1b) in which the geometry of the bar and the concrete is modelled by three-dimensional elements and (2) phenomenological modelling (see Figure 1c) based on a smeared or discrete formulation of the bar-concrete interface [3].

![Figure 1. Schematic simulation of the idealized bond zone [3]](image)

In the phenomenological modelling of bond the concrete and the reinforcement are discretised by two- or three-dimensional finite elements. The link between the bar and the concrete can be realized by a discontinuous approach where bond is defined by discrete, zero-thickness elements (springs) whose behaviour is controlled by the bond stress-slip relationship. This approach is able to realistically predict the bond behaviour for different geometries and for different boundary conditions only if a realistic constitutive model for the surrounding concrete is used. However, the model is not able to automatically predict the bond behaviour of a given bar geometry. Consequently, the influence of these parameters must be stored in advance in the basic parameters of the bond model. Thus one has the possibility to realistically simulate the behavior of reinforced concrete structures with relatively low effort in modelling and computing time. By the use of detailed modelling, such as both modelling of the ribs of the reinforcement and the concrete lugs (see Figure 1b) between the ribs of the reinforcement a quite fine finite element mesh has to be generated. This leads again to a high effort in modelling work and also to a really long computing time in particular while carrying out a finite element analysis on complex reinforced concrete structures [3, 4].

2. REINFORCEMENT FINITE ELEMENT MODELS

In finite element modelling of reinforced concrete structures, there are three
different alternative representations of reinforcement: smeared, embedded and discrete reinforcement models. The first one is rarely used and therefore it depends on the nature of used structure. The discrete and embedded representations are formulated and introduced in the developed program [4, 5].

2.1. Discrete Reinforcement Representation
The discrete modelling of steel reinforcement is the first approach used in finite element analysis of reinforced concrete structures [10]. The discrete representation of reinforcement uses one dimensional truss elements and it is the only way for accounting for bond slip and dowel action effects, Figure 2.

![Figure 2. Discrete representation of steel bars [4]](image)

A significant advantage of discrete representation is that it can account for possible displacement of the reinforcement with respect to the surrounding concrete. The bond effects are usually related with this representation and the bond-link or cohesive models can be used to connect the steel and concrete nodes in order to consider this effect. The main disadvantage is that the finite element mesh patterns are only restricted by the location of reinforcement and consequently the increase of the number of concrete elements and the degrees of freedom. In this way, Lagrange or Serindipity isoparametric concrete elements are used and a line three node truss elements is used to represent the steel and the compatibility between concrete and steel must be guaranteed [4].

2.2. Embedded Reinforcement Representation
In this representation, the reinforcement bar is considered as an axial member incorporated in the concrete element such that its displacements are consistent with membranous concrete elements and bond loss can be considered, Figure 3. In this scope, many works have presented different formulations for this model. Embedded models allow for an independent choice of concrete mesh. So, the same number of nodes and degrees of freedom are used for both concrete and steel. The disadvantage of this procedure is that additional degrees of freedom increase the computational and numerical treatment [4].
3. FINITE ELEMENT MODELS FOR BOND

Two different elements have been typically proposed to include the bond-slip effect in the finite element analysis of RC structures. One is bond link element and the other is bond zone element as also known as contact element. These elements are associated with the discrete reinforcement model, which has the advantage of representing different material properties more precisely. Afterwards other bond conditions at different nodes can be easily represented [5]. To describe the bond behaviour between concrete and steel, the vertical and horizontal relative displacement between concrete and steel in the local coordinates can be considered. The same type of isoparametric elements and it has, at the unloaded stage, no physical dimension in the transverse direction. It uses linear, quadratic or cubic interpolation functions corresponding to the number of nodes per element. In linear analysis, the vertical relative displacements are too small compared to the horizontal displacement [4].

3.1. Analysis With Bond Link Element

Bond-link element consists of two orthogonal springs which connect and transmit shear and normal forces between a reinforcing bar node and an adjacent concrete node (Figure 4). Since the link has no physical dimensions, the two connected nodes originally occupy the same location in the finite element of undeformed structure [5]. The bond element is a two-node finite element. The element displacement field is a slip which is defined as a relative movement between the reinforcing bar and concrete in the direction parallel to the axis of the reinforcing bar.

The bond effect is assumed as an interaction between reinforcing bars and surrounding concrete. When the change of stresses in concrete and steel occurs, the effect of bond begins and becomes more pronounced at the end anchorages of reinforcing bars and in the vicinity of cracks.
3.2. Analysis With Contact Element
The behaviour of the concrete-steel interface must be described from stress-strain laws. Many constitutive relationships are presented in the literature. In this element, the contact surface between the steel bar and the concrete in the immediate vicinity of the steel bar is modeled by a bond stress-slip law which considers the special properties of the bond zone. The most important differences are that contact element has the dimension along the steel-concrete interface (it does not have physical dimension in other two directions) and it provides a continuous contact surface between - steel bar and concrete [5].

3.3. Analysis Without Bond
In this case, the stiffness matrices of the steel elements are computed in local axis at the nodes of non bond. The concrete element stiffness matrices are calculated in the global axis and they are transformed steel local axes at common nodes. In y-direction, concrete and steel have the same degree of freedom but have different degree of freedom in x-direction at common nodes [4].

4. LOCAL BOND SLIP RELATIONSHIP
4.1. Differential Equation Governing The Slip
In Figure 5 a steel reinforcement embedded in a concrete mass is shown. Over a small piece of the bar, dx, the change in the relative displacement of the steel to concrete, $d\Delta$, is equal to change in steel deformation, $\delta_s$, minus the change in concrete deformation, $\delta_c$. That is [7]:

$$d\Delta = \delta_s - \delta_c$$  \hspace{1cm} (6)

The magnitudes of differential deformation for the reinforcement and concrete, if we assume an elastic state, are given in equation 7 and 8 respectively as follows:

$$\delta_s = \left(\frac{\sigma}{E_s}\right)dx$$  \hspace{1cm} (7)
\[ \delta_c = \left( \frac{\sigma_s}{E_s} \right) dx \]  

(8)

Figure 5. Bond consideration for steel reinforcement in concrete.

where the sub-scripts “s” and “c” refer to steel and concrete respectively. The terms used in equation 1 are general (independent of the type of reinforcement) and apply to local level (vary with location and stage of the test) [7].

In practice, the value of \( \delta_c \) is negligible relative to \( \delta_s \) because the concrete section is usually much larger than the steel section and the normal stress in concrete is much lower. Therefore, the second term in equation 6 is neglected and whole differential slip at local level is attributed to the steel deformation. It follows that equation 6 reduce to [7]:

\[ d \Delta \sim \delta_s \]  

(9)

Substituting from equation 7 into equation 9 and re-arranging, we can write:

\[ \frac{d \Delta}{dx} = \frac{\sigma_s}{E_s} \]  

(10)

If we differentiate both sides of the above equation with respect to \( dx \), the following equation will be obtained:

\[ \frac{d^2 \Delta}{d^2 x} = \left( \frac{1}{E_s} \right) \frac{d \sigma_s}{dx} \]  

(11)

On the other hand, the bond stress and steel stress (over segment \( dx \)) are inter-related from the condition of equilibrium that states (Figure 5):

\[ (\sigma_s + d\sigma_s)A_s = \sigma_s A_s + \tau \times dx \times \pi \times d_b \]

Simplifying:
\[
\frac{d \sigma_s}{dx} = \tau \times \left( \frac{\pi d_p}{A_s} \right)
\]  

(12)

If we substitute from equation 12 into equation 11, the following equation will be attained:

\[
\frac{d^2 \Delta}{dx^2} = \tau(s(x)) \times \left( \frac{\pi d_p}{A_s E_s} \right)
\]  

(13)

Where \(d_p\) is the diameter, \(A_s\) is the cross-sectional area, \(E_s\) is the Young’s modulus of the reinforcing bars and \(s(x)\) is the slip between concrete and steel abscissa \(x\) [7].

Equation 13 is known as the fundamental differential equation for the bond between a steel reinforcement and concrete. This equation has been drawn in the same form as shown above or in other forms (but with the same concept) by various authors.

It is assumed that the bond characteristics of reinforcing bar are analytically described by a local relationship of bond \(\tau = \tau(s)\), in which \(\tau\) is the shear stress acting on the contact surface between bars and concrete and \(s\) is the slip; that is the relative displacement between those of the steel bar and concrete.

4.2. Analytical Expressions for Bond-Slip Relationship

The experimental evidence indicates that the load transfer between reinforcement and concrete is mainly accomplished through bearing of the reinforcing bar lugs on the surrounding concrete and through friction at large slip values (Figure 1a). The adhesion is negligible. This behavior can be described using so-called bond stress-slip relationships.

The simple bi-linear bond stress-slip model is selected and the parameters of the model are derived from the experiment data corresponding to material features of each specimen. The bond stress-slip relationship which is used in the model and the corresponding components are shown in Figure 6.

\[
\tau_1(s) = E_b s \quad s \leq s_1
\]  

(14)
\[ \tau_2(s) = \tau_1 + E_b \Delta u(s - s_1) \quad s_1 \leq s \leq s_2 \]  

\( s_1 = 2 \text{ mm}, \ s_2 = 10.5 \text{ mm}, \ \tau_f = 10.55 \text{ MPa} \quad \text{and} \quad \tau_1 = 13.50 \text{ MPa} \)

5. NUMERICAL EXAMPLES

5.1. Finite Element Modeling of Pullout Test

To study the bond behavior of steel reinforcement in a concrete matrix, we use pull-out tests of a steel bar (Ø12mm) with ribs (see Figure 1) which was performed by Eligehausen (2003) [9].

To investigate the performance of the cohesion layer, numerical investigations on pullout specimens have been carried out. The specimen is an anchor of a reinforcing bar \( d_b = 12 \text{ mm} \) in a well confined cylinder of concrete of 150mm height and 60mm diameter which corresponds to anchorage length of 5 bar diameters (embedment length \( l_E = 5 \ d_b = 60 \text{ mm} \)).

For the numerical investigations the finite element software (Ansys) has been used and a detailed FE model in 3D mode with and without bond-slip effect as cohesion layer to simulate bond have been employed. Since rib of reinforcement are being simulated, the mesh size close to the rib in steel bar, concrete and cohesion layer should be small enough to accurately describe the deformation and stress gradients. However, for the remaining regions coarse mesh can be used in order to reduce the computational costs. The results of these numerical investigations are compared with the results of the experimental investigations [8]. The test specimen used in the finite element model is shown in Figure 7.

Table 1 shows the summary of the basic material variable used in the experimental and numerical investigations.
Table 1: Summary of the material parameters

<table>
<thead>
<tr>
<th>Material properties</th>
<th>Values (kg/cm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete compressive strength</td>
<td>300</td>
</tr>
<tr>
<td>Concrete tensile strength</td>
<td>30</td>
</tr>
<tr>
<td>Concrete E modulus</td>
<td>273664</td>
</tr>
<tr>
<td>Concrete Poisson’s coefficient</td>
<td>0.2</td>
</tr>
<tr>
<td>Steel E modulus</td>
<td>2100000</td>
</tr>
<tr>
<td>Steel yield stress</td>
<td>3000</td>
</tr>
<tr>
<td>Steel Poisson’s coefficient</td>
<td>0.3</td>
</tr>
</tbody>
</table>

A displacement control load being applied to the end of the reinforcement in pull-out test Figure 8.

In this paper, the concrete and the reinforcement bar was modeled by eight-node Serendipity axisymmetric elements (Plane 82, Axisymmetric) with 2×2 Gauss integration points. To display the bond slip effect between concrete and steel, two distinct models are selected, such as: (1) full perfect, (2) bi-linear model. These models are introduced in finite element program (Ansys) and the collected results are analysed and discussed in the next section.

![Figure 8. Displacement control load applied to the end of the reinforcement](image)

Figure 9. Stress distribution in reinforced concrete with bi-linear law of bond, (a) σₓₓ, (b) σᵧᵧ, and (c) τₓᵧ.
The bond stress-slip relation obtained in the finite element calculation when the three dimensional modelling of the reinforcement is used are substantially corresponding to the curves of the experimental investigations.

According to the normal and shear stress curves (Figures 9-10), it is possible to appreciate how the connection influences the transmission of the efforts from steel bar towards the concrete and vice versa.

### 6. CONCLUSION

In this paper the methods of modeling of reinforcing bars and bond-slip models between steel rebar and concrete in the finite element program is described. Then one analytical expression of bond-slip relationship is selected and the pull-out test with slip and without slip modeled by finite element software (Ansys) in 3d mode and then the obtained results are presented and compared with experimental data from pullout test. It was found that stress distribution in the steel bar and concrete of pull-out tests may principally be influenced by the properties of the interface.

1. In the improvement of finite element models of composite material, it is necessary to use not only the constitutive laws of concrete and steel but also one of the interface.
2. The stress distribution in the steel bar of pull-out tests may principally be
influenced by the properties of the interface.
3. The finite element studies of pullout tests with a short embedment length (local bond conditions) show relatively good agreement between experimental and numerical results.
4. The cohesion layer is able to predict transfer of bond stresses from reinforcement into concrete realistically.
5. The proposed approach predicts the stress field in the concrete and along the steel bars (local behaviour)

REFERENCES
A NONLINEAR DYNAMIC BASED REDUNDANCY INDEX FOR REINFORCED CONCRETE FRAMES

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ABSTRACT

In almost all codes of practice for seismic resistant design of buildings, a behavior factor is used to reduce design base shear. The behavior factor is affected by several parameters such as ductility, overstrength and redundancy reduction factors. There are two common approaches to assess the effects of redundancy on the strength of a structural system, which are as follows: Static Pushover Analysis and Incremental Dynamic Analysis. The two indices: redundancy strength coefficient and redundancy variation coefficient have been introduced to measure these effects. Simplified methods are developed and presented to calculate these parameters. In this paper the redundancy strength and the redundancy variation parameters are evaluated for the reinforced concrete plane frames with different number of stories, bays and ductility capacities. The investigations indicate that these two parameters are mainly the results of redundancy reduction factors.

Keywords: redundancy, behavior factor, ductility, concrete frames, redundancy strength index, redundancy variation index

1. INTRODUCTION

Although behavior factor (R) has been an important subject in structural engineering studies in recent years but most important studies in this field get back to the last two decade. Among the researchers in this field, Freeman is the one who calculated behavior factor with using the capacity spectrum method. According to this method, to compute the quantity of R by an analytical method, it can be formulated as follows:

\[ R = R_A \times R_B \times R_C \times \ldots \times R_N \]  

(1)

Where Rx are parameters such as arrangement of frames, type of structural system, composition of loads, degree of uncertainty, damping, characteristics of nonlinear behavior in structure, characteristics of materials, ratio of building dimensions, failure mechanism and other effective parameters. The range of effective factors in determining R is such that it would almost be impossible to find two buildings with identical behavior factors. In other words, each building has its own unique
features. Therefore, instead of adding all effective factors, as mentioned in the behavior factor relation, usually only the factors having more determinant role in the behavior factor are studied. In this paper, two main coefficients namely the structural capacity and the force resulting from earthquake are primarily considered, and the factors that help increase the capacity and reduce the seismic forces are determined in the following steps.

In 1991, separate researches on behavior factor, also known as Uang plasticity coefficient method were accomplished by Uang [1]. In 1997, Pandey and Barai [2] studied the structural sensitivity response to uncertainty. They assumed that for every structure subjected to a given loading, the general uncertainty is proportional to the reverse structural sensitivity response; thus, the structural response sensitivity reduces with increasing uncertainty.

In 1999, Bertero and Bertero studied uncertainty in the seismic resistant design. In this study, they explained the main concepts of seismic uncertainty and defined the probabilistic effect of uncertainty on structural failure.

In 2003, Wen, and Song [3], studied the reliability of structural behavior under earthquakes. They believed that when more elements are involved in resistance against lateral load, the probability for collapse of all elements, at the same time, is lower than the case when le elements with equal resistance are involved.

In 2004, Hosain and Tsopelas tried to determine structural uncertainty in reinforced concrete buildings. In that, they studied $r_u$ (uncertainty resistance coefficient) and $r_v$ (uncertainty variation coefficient) and their relation with the component’s plastic rotation ductility factor ($\mu_\theta$). The effects of number of stories and bays, the length of bays and story height were studied as well. They then studied the effect of uncertainty on behavior factor ($R_B$). Here, the effect of number of stories and bays, bays’ length, story height and also the effect of gravity loads on uncertainty coefficient are studied. Even the effect of number of frames present at each lateral load direction has been considered, and finally, the procedure to compute uncertainty coefficient using uncertainty resistance and uncertainty change coefficients were studied.

2. REDUNDANCY

The redundancy concept has been considered by engineers, especially after Kobe, Northridge and Turkey earthquakes, during which many buildings with low redundancy degree were damaged. Therefore, the redundancy topic was introduced seriously, and the degree of redundancy in structural systems was considered for seismic design.

There is some information about the useful effects of redundancy in structural resistance, but the efficient methods measurement methods are not available as yet. The effects of three parameters are usually considered to measure redundancy degree, which include:

1. Static redundancy degree of system
2. The ratio of probability in system failure to parts failure
3. Involvement of additional capacity which was not necessary for design
Some researchers studied the effects of redundancy degree with deterministic method; that is, use of nonlinear static analyses. There are few studies in which probabilistic method is applied to determine the effects of redundancy degree using structural reliability.

Seismic redundancy degree (n) for a structural system is actually the number of critical areas (plastic hinges) in a structural system which continue to yield until the structure exceeds the allowable limit leading to emergency disasters like plastic displacement or complete collapse. In engineering problems of earthquake, it is assumed that if all critical points (plastic hinges) yield simultaneously, the structure would fail under earthquake shaking. The redundancy degree is defined using the parallel and serial structural system reliability theory, determining the probability of failure in serial systems by weakest connection model and setting the probability of failure in parallel systems through secure decay model [4].

In 1999, Bertero et al studied the effect of redundancy and redistribution of internal forces in seismic design and stated that a part of behavior factor is originated from redundancy degree and can not be determined independent from overstrength and ductility. They also assumed that when the structure can not withstand gravity loads under the effect of earthquake forces, it would collapse. About structural resistance against displacement due to increasing lateral load, the resistance in the first yielding point is considered, and the maximum resistance is predicted using the reliability of displacement capacity.

A structure takes advantage of the positive effects of redundancy degree when:

a) Change coefficient in “structural demand” reduces in comparison to change coefficient in “structural capacities”

b) Addition resistance increase

c) Curvature capacity increases in plastic hinges

d) A minimum rotation capacity is ensured in all elements of structural system

According to much uncertainty in structural capacity and demand, one of the methods in studying the redundancy of structural systems under seismic loads, is to use the reliability concept. In one kind of structural system without change in materials and configuration, the redundancy degree factor can only influence the reliability on structural stability against earthquake induced lateral loads and the structure behavior factor, seriously. It should be considered that the redundancy degree is different in similar frames. If the size of an element, its reinforcement and implementation details change, the failure mechanism may naturally change, but even for two completely similar frames, redundancy degree will be different for various lateral load models [5].

Behavior factor used in codes, which reduces the level of elastic forces in the design process and in its primary formulation, is defined in terms of ductility coefficient ($R_\mu$) and the additional resistance coefficient ($R_s$)[6]. Ductility coefficient is computed considering nonlinear response of structural system. The relationships for computing functional ductility coefficient is formulated by some researchers by involving the natural period of structure and its ductility capacity, which are commonly based on nonlinear response change of a multi-story building relative to nonlinear response of a system with single degree of freedom [7].
The overstrength capacity shows the actual lateral resistance in comparison to modeling resistance. Overstrength may be divided into two general parts. The first part is related to the overstrength resistance modeling until the first hinge yields in a structure, and the second part is related to the formation of the first hinge until a mechanism for the total failure of a structure is developed [8].

In ATC-19, the formulation of behavior factor (R) is introduced [8]. This coefficient includes an additional factor (\(R_R\)) used to account for the effect of redundancy degree in a structure. These effects include probability effects and others related to structural system geometry either in a plan or at a point in height. Therefore, behavior factor (R) is equal to:

\[
R = R_{\mu} \cdot R_S \cdot R_R
\]

Some effective parameters in redundancy and structural system reliability are the ratio of demand to the capacity of structural systems, the kind of failure mechanism formed, building high, the number of stories, the length and the number of bays. This study computes the probabilistic and deterministic effects of redundancy through obtaining two redundancy resistance index (\(r_s\)) and redundancy variation index (\(r_v\)). These two indexes are used in measuring the resistance reduction coefficient from \(R_R\) redundancy for structural frames with two-dimensional reinforced concrete.

3. REDUNDANCY INDEXES

Redundancy resistance index \(r_s\) represent the ability of a structural system in redistributing forces while failure and the capability of a structure in transferring the forces of elements yielded to the elements with higher resistance. This index is a function of static redundancy, ductility, strain hardening and the average resistance of elements in a structural system. Second index having probabilistic nature is an \(r_v\), redundancy variations index. This index measures the probability effect of elements resistance on structural system resistance. It is also a function of static redundancy in a structural system, and on the other hand is a function of statistical nature in ductility and structural elements resistance. Following variables are used in computing above indexes:

- Base shear in the beginning of yielding system.
- Ultimate base shear.
- The number of local failure or the number of plastic hinges caused during ultimate failure of structure.
- The access of elements curvature to ultimate curvature.

4. REDUNDANCY RESISTANCE INDEX

Redundancy resistance index \(r_s\) are defined as the ratio of average ultimate resistance (\(\overline{S_u}\)) to yielding resistance (\(\overline{S_y}\)). In which \(\overline{S_y}\) is the average system resistance non-redundant system.
In a method suggested for this paper in studying the effects of redundancy using nonlinear dynamic analysis with increased acceleration, the base shear during failure and yielding is considered. In previous studies, this method is applied for studying the effects of overstrength [10]. In this study, the system failure standards that will be considered in nonlinear static and dynamic analyses with increased acceleration are as follows:

- Limitations related to storey drift which according to code [11] for buildings which period lower than 0.7 second are limited to 2.5% and for structures with period more than 0.7 second are limited to 2%.
- The index of structure stability which in a structure with high ductility is limited to 0.125 and in a structure with low ductility is limited to 0.25.
- The formation of failure mechanism in a structure and collapsing structure.
- The access of structure failure index to a number one according to park-Ang criterion [12].

In pushover static analyses performed in this study, it is assumed that lateral loads with reverse triangular distribution are inserted into a structure which is proportional to Iran 2800 standard earthquake force. In nonlinear dynamic analysis with increased acceleration, the maximum acceleration of any record is coordinated to a primary number (here it is considered to be 0.02g) and in one stage in increased to 0.02g and the structure is analyzed in every step until when one of the four above-mentioned criteria’s is occurred. In this stage, the analysis is stopped.
and base shear is used during yielding and maximum base shear is used for measuring redundancy resistance index.

5. REDUNDANCY VARIATION INDEX

The relation between resistance of a structural system and the resistance of its composing elements is obtained using plastic analysis of structure. In this relation, the selection of failure mechanism is important because it can result in non-actual estimates from redundancy variation index. For simplify computations, one sway mechanism according to Figure (2) is considered. This mechanism is based on the "strong column" and "weak beam" assumption which column resistance is at least 20% more than the resistance of beams.

![Figure 2. Sway type failure mode of a generic plane frame](image)

The frame strength (base shear strength) for any failure mode could be represented by the following expression:

\[
S = \sum_{i=1}^{n} C_i M_i
\]

(4)

Where \(S\) = frame strength (base shear); \(n\) = number of plastic hinges in the frame resulting from the particular failure mode or collapse mechanism considered; \(M_i\) = yield moment of the structural element where plastic hinge "i" is formed; and \(C_i\) = coefficient with units radians length that is a function of the plastic rotation and geometry of the structure. Eq. (4) is of the form of the strength equation of a parallel system type.

The mean value of the frame strength can be derived from the following expression:
\[ \bar{S} = \sum_{i=1}^{n} C_i M_i \]  

(5)

Where \( M_i \) = mean value of the strength of the structural element where plastic hinge "i" is formed.

Accordingly the standard deviation of the frame strength \( \sigma_f \) can be obtained from:

\[ \sigma_f = \sqrt{\sum_{i=1}^{n} \sum_{j=1}^{n} C_i C_j \rho_{ij} \sigma_{M_i} \sigma_{M_j}} \]  

(6)

Where \( \rho_{ij} = \) correlation coefficient between the strengths \( M_i \) and \( M_j \) and \( \sigma_{M_i} = \) standard deviation of the yield moment \( M_i \). also \( \rho_{ij} = 1 \) for \( i = j \). To further simplify the deviation, a regular multistory multi-bay frame with the following properties is considered:

1. The frame is composed of elements with identical normally distributed strengths:

\[ M_i = M_j = M_e \]  

(7)

\[ \sigma_{M_i} = \sigma_{M_j} = \sigma_e \]  

(8)

2. The correlation coefficient between the strength of any two pairs of elements is the same.

\[ \rho_{ij} = \rho_e \]  

(9)

3. The bays of the frame have identical spans and the stories identical high which result in:

\[ C_i = C_j = C \]  

(10)

Eq.(5) and (6) now become:

\[ \bar{S} = nC \bar{M}_e \]  

(11)

\[ \sigma_f = C \sigma_e \sqrt{n + n(n-1)\rho_e} \]  

(12)

The following relationship between the coefficient of variation (COV) of the frame strength \( \nu_f \) and the COV of the element strength \( \nu_e \) is calculated by dividing Eq. (12) to Eq. (11):
The redundancy variation index $r_v$ is defined as the ratio between $v_f$ and $v_e$:

$$ r_v = \frac{v_f}{v_e} = \sqrt{\frac{1 + (n - 1)\rho_e}{n}} \quad (14) $$

For a parallel system with unequally correlated elements, $\rho_e$ could be substituted with the average correlation coefficient $\bar{\rho}$ defined as:

$$ \bar{\rho} = \frac{1}{n(n-1)} \sum_{i\neq j}^{n} \rho_{ij} \quad (15) $$

Therefore, Eq. (14) Can be modified using the average correlation coefficient of the strengths of the plastic hinges as fallows:

$$ r_v = \sqrt{\frac{1 + (n - 1)\bar{\rho}}{n}} \quad (16) $$

Hence the redundancy variation index $r_v$ is a function of the number of plastic hinges “n” and their average correlation coefficient between their strengths, and represents a measure of the probabilistic effects of redundancy on the system strength, its values range between 0 and 1.

For a building structure where a single plastic hinge causes collapse ($n=1$), $r_v = 1$ and the structure under consideration in non-redundant. The other extreme value $r_v = 0$ indicates an infinitely redundant structural system and is reached either when an infinite number of plastic hinges are required to cause collapse (practically “n” attains large values) or when element strengths in a structure are uncorrelated (the average correlation coefficient in Eq.(16) is zero).

Using Eq.(16) $r_v$ can be estimated from a pushover or dynamic analysis and for a particular value of the average correlation coefficient of the structural member strength.

6. REDUNDANCY FACTOR "R_R"

The overall effects of redundancy on the structural strength may be completely described by the ratio of the ultimate strength of a structural system to the ultimate strength of non-redundant structure. Thus:
Where $S_u =$ structural system strength which includes all the effects of redundancy; and $S_{nr} =$ the same strength but for non-redundant structural system. Assuming that the strength of a structure is distributed normally, the characteristic or design strength of a structural system, its standard deviation, the coefficient $k$ is formed. Therefore, both $S_u$ and $S_{nr}$ may be written as follows:

$$ S_u = \bar{S}_u - k\sigma_f $$  \hspace{1cm} (18)

$$ S_{nr} = \bar{S}_{nr} - k\sigma_{nr} $$  \hspace{1cm} (19)

Where $\sigma_f =$ standard deviation of the frame strength; $\sigma_{nr} =$ standard deviation of the non-redundant frame strength; $\bar{S}_u =$ average of the ultimate frame strength and $\bar{S}_{nr} =$ average of the non-redundant frame strength.

An expression for $\sigma_f$ could be obtained as follows:

$$ r_v = \frac{\sigma_f}{\sqrt{r_s}} \Rightarrow \sigma_f = r_v r_s \bar{S}_{nr} $$  \hspace{1cm} (20)

By virtue of $\bar{S}_u = r_v \bar{S}_{nr}$; Eq. (19) results into:

$$ S_u = r_s \bar{S}_{nr} - k r_v r_s \bar{S}_{nr} = r_s (1 - k r_v \bar{S}_{nr}) $$  \hspace{1cm} (21)

Where $r_v =$ redundancy variation index; $r_s =$ redundancy strength index and $v_c =$ COV of the strength of the structural system elements.

Using Eq. (19) and (21), Eq.(17) becomes:

$$ R_R = \frac{r_s (1 - k r_v \bar{S}_{nr})}{\bar{S}_{nr} - k \sigma_{nr}} = r_s \left( \frac{1 - k r_v \bar{S}_{nr}}{1 - k v_{nr}} \right) $$  \hspace{1cm} (22)

Where $v_{nr}$ is the COV (coefficient of variation) for non-redundant frame strength.

A non-redundant frame structure could be modeled as a parallel system consisting of ideal elastic-brittle elements. Such a system behaves like a series system, where failure of one element results in the system collapse, and that the safety index of the system is equal to that of the element. For a non-redundant system, $(n=1)$ $v_{nr}=v_c$. Therefore, the redundancy factor ($R_R$) can be expressed as follows:

$$ R_R = r_s \left( \frac{1 - k v_c r_v}{1 - k v_c} \right) $$  \hspace{1cm} (23)
7. CASE STUDY ABOUT THE EFFECTS OF REDUNDANCY ON TWO-DIMENSIONAL CONCRETE FRAMES

In order to compute redundancy indices, 16 frame samples from 2 bay to 5 bay and with two, four, six and ten stories were designated. SAP2000 software [13] and the IDARC software [14] are used for nonlinear dynamic and nonlinear static analyses. For nonlinear static analysis, 16 frame samples with high ductility and 16 frame samples with low ductility are selected (Figure 3).

The lateral load pattern applied to the structure is reverse triangular, which is
approximately in accordance with lateral force criteria of the earthquake standard 2800 of Iran. Four different cases of design and analysis are considered for comparison. In the first case, the bay length is 4 meters and the story height is 3 meters and in the second case, the story height is increased from 3 to 4 meters. In the third case, the bay length is increased to 5 meters and finally in the forth case, the gravity loading intensity is increased to 30%. Therefore, in static nonlinear analysis, one hundred twenty eight frames are designed with SAP2000 and then analyzed by the IDARC. Response curves are computed in terms of displacement at the top of structure ($\Delta_{tar}$) with respect to base shear divided by structure weight ($C_b$). Two values, base shear coefficient during yielding and also maximum base shear coefficient are important over curve. The $r_s$ index is obtained by dividing maximum base shear coefficient to the base shear coefficient when yielding.

Using maximum number of plastic hinges formed in nonlinear static analysis, one can obtain $r_v$ index. As a result of having these two indices, resistance reduction coefficient can be obtained from redundancy according to relations in the third part.

In order to carry out nonlinear dynamic analysis with increased acceleration by the IDARC software, begins to analyze with a primary PGA value in any stage, and it continues the operation with 0.02g increase relative to the previous measure, until one of the failure conditions is reached. In this case, the value of base shear coefficient is applied for computing $r_s$ and also for computing $r_v$ index. The number of plastic hinges formed while failure is used to compute 16 frames with high ductility and 16 frames with low ductility. Eight seismic records are applied, equally, for both linear and nonlinear static analysis methods, for 4 different cases. Finally, 1024 frames were analyzed with different cases and the values of base shear coefficient while forming the first plastic hinge. Maximum base shear coefficient and the number of plastic hinges when failing are used as parameters required for computing $r_s$, $r_v$ and $R_R$ indices. It is necessary to note that the average values obtained from eight records is the basis for computing above indices.

8. CONCLUSION

Comparing the responses obtained from Static Pushover Analysis method (SPO) with Incremental Dynamic Analysis (IDA), it is concluded that in most conditions, $R_R$ coefficients obtained from static method are larger than dynamic method, but this difference is maximally 10%. According to result from Figure(4), we can conclude that the results obtained from nonlinear static method are in good agreement with results obtained by nonlinear time history method and may be used as a reliable method.

Finally, it should be emphasized that these results are only for frames modeled in this study and might not hold true for all other structural models.
REFRENCES


Figure 4. Average of redundancy modification factor with number of story
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EVALUATION OF REDUCTION FACTOR FOR REINFORCED CONCRETE BUILDINGS RETROFITTED WITH CFRP JACKETS

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ABSTRACT
Reduction factor shows the efficiency of lateral load resistance systems in dissipation of seismic energy through inelastic behavior. This parameter is broadly used in guidelines to determine elastic resistance of the structure. Since these seismic guidelines mainly put their emphasis on common lateral load resistance systems, it may not be appropriate to use the published reduction factor values in designing composite or strengthened lateral load resistant systems. The main objective of this research is to examine the quantitative impacts of confined concrete columns with CFRP jackets on reduction factor. Therefore, three models of 4, 7 and 10-story buildings, in a very high seismic zone were selected. Pushover analyses were performed by means of the software SAP 2000 for three-dimensional models. Finally, the reduction factor of reinforced concrete (RC) buildings that were retrofitted with CFRP jackets was found to be 9.9. This result indicates an enhancement in the seismic resistance and specially, in ductility of the buildings.

Keywords: concrete buildings, reduction factor, confinement, carbon fiber reinforced polymer (cfrp) jackets, pushover analysis

1. INTRODUCTION
Reduction factor shows the efficiency of lateral load resistance system in dissipation of seismic energy through inelastic behavior. This parameter is broadly used in guidelines to determine elastic resistance of the structure. By taking many parameters and effects into consideration, depending on type of the lateral load resistant system, different seismic design guidelines reduce the calculated values for earthquake loads. These parameters and effects are namely structural system ductility, structural indeterminacy degree, structural overstrength and dissipation of seismic energy. For the first time, in the first decade of the twentieth century, following obtained experiences and study results from real earthquakes, researchers proposed the vertical seismic shear force to be a ratio of total weight of the building. In the following years, it was found that for higher buildings, the stiffness reduces and the period of vibration increases as the height increases. As a result, earthquake imposes lower accelerations to higher buildings. With finding out this
phenomenon, further development of structural dynamics knowledge and better understanding of structures behavior, it was understood that reduction factor is related to the number of building stories. Accurate determination of the reduction factor of a building would improve exactness of the calculation of its seismic resistance, evidently. This factor depends on various parameters such as type of the lateral load resistant system, fundamental vibration period of the building, force-deformation model for materials, ductility capacity, overstrength factor and design safety factors.

In the present study, firstly, different definitions were explained and then possible effects of abovementioned parameters on reduction factor were investigated. Since seismic retrofitting of buildings is a new concept in Iran, quantitative effects of confinement of RC sections with CFRP jackets on reduction factor of RC buildings were subsequently studied. Three models of 4, 7 and 10-story buildings, in a very high seismic zone were selected. Pushover analyses were performed by means of the software SAP 2000 for three-dimensional models.

2. MODELING APPROACH
Some of the most important force-deformation models are bilinear, trilinear and those with reduction of stiffness and resistance in each cycle. The force-deformation relationship should be based on experimental documents or those, which are stated in [1-3]. For a pushover analysis, it is possible to utilize the general force-deformation relationship that is illustrated in Figure 1 or any other proper curves, which describe the performance under constant increase of displacement. For nonlinear dynamic methods, force-deformation relationships should describe the performance under both constant increases of displacement or under numbers of displacement cycles.

![Figure 1. Force-deformation relationship for concrete elements [3]](image)

3. DUCTILITY FACTOR
The most significant parameter in determining reduction factor of a structure is the ductility factor. Ductility factor is shown with ($\mu$) and is calculated from below equation:

$$\mu = \frac{\Delta_u}{\Delta_y}$$

(1)
where, $\mu$ is the ductility factor defined as the ratio between the maximum displacement ($\Delta_u$) and the yield displacement ($\Delta_y$). Higher values of ductility factors would mean higher ductility capacities and therefore, higher reduction factors.

Several studies have been conducted for determination of reduction factors. Most of these studies propose that for a specified earthquake record, the reduction factor depends on ductility and fundamental period of vibration of the building [4-6]. Consequently, with a high precision, reduction factor could be expressed in terms of ductility as stated in Eq. (2):

$$ R_\mu = R_{\mu}(T, \mu) $$

An excellent overview has been presented by Miranda and Bertero (1994). In this paper, a bilinear spectrum was used for the reduction factor $R_\mu$ [6]:

$$ R_\mu = (\mu - 1) \frac{T}{T_C} + 1, \quad T < T_C $$

$$ R_\mu = \mu, \quad T \geq T_C $$

where, $\mu$ is the ductility factor as defined above, $T$ is characteristic period of the ground motion and $T$ is the fundamental period of structures.

Fundamental period of structures ($T$) is a major factor in calculation of reduction factor and could be assessed using various experimental methods or by means of computers. Increase of this parameter would increase reduction factor. The fundamental period of a non-retrofitted reinforced concrete building without shear walls could be computed from the below experimental relationship [7]:

$$ T = 0.8 \times 0.07 H^{\frac{1}{4}} = 0.056 H^{\frac{1}{4}} $$

where, $H$ is the building height in meters.

4. OVERSTRENGTH FACTOR

In most of design guidelines, structures are designed so that none of their elements exceed the elastic state. However, since some plastic hinges form after exceeding this limitation, it does not always result in the collapse of the structure. By forming plastic hinges, general stiffness of the structure would decrease but still, it can resist higher loads. This procedure will continue until the formation of plastic hinges cause an instability mechanism and stiffness of the structure become zero. In the case that structure loses its ductility capacity simultaneously, it would collapse. As a result, in mentioned guidelines, the extra resistance of elements after the formation of plastic hinges is neglected. This resistance capacity is defined as the overstrength factor ($R_o$) and is calculated as ratio of the yield base shear to the
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By considering important parameters, reduction factor of a structure, could be determined by [9]:

$$ R_u = Y \times R_\mu \times R_S $$

where, $R_\mu$ is the period-dependent ductility factor, $R_S$ is the period-dependent over strength factor and $Y$ is the safety factor.

7. MODELING AND ANALYSIS

SAP2000 Nonlinear Version 8 has been utilized for analysis and design procedures [10]. This program is capable of performing static and dynamic analyses of structures in three dimensions. This program is compatible with most of the design guidelines.

A Three-dimensional model of each structure was created in SAP2000 to carry out nonlinear static analysis. In addition, a $P$-$\Delta$ analysis has been performed for every model. The ACI 318-99 guideline was employed for design purposes, since this guideline is supported by the SAP2000 program.

8. INTRODUCING MODELS

Three symmetrical moment resistance RC frame buildings are considered in this study. Three structures of 4, 7 and 10-story buildings are modeled in three dimensions with height to width ratio of about 1 to 2. These structures are considered according to Iranian Seismic Design Code (Standard 2800-05) [7] as residential buildings with a medium importance factor ($I$=1.0). Based on this code,
structures are assumed located in a very high seismic zone with a design ground acceleration of 0.35g and the soil type is assumed class II. For all of the three models, span lengths equal to 4 m in both directions. The 4-story model consists of a three-bay frame, 7-story consists of four bays and the 10-story consists of five bays (Figure 2). Typical floor-to-floor height is 3.2 m.

![Figure 2. Plan view of 4, 7 and 10 story buildings](image)

Beam and column dimensions and the amount of longitudinal reinforcement were specified as could be seen in Figure 3. Table 1 presents the final FRP jacket thicknesses for all columns.

Concrete properties are assumed to be 210 kg/cm² for the compressive strength, 218800 kg/cm² for modulus of elasticity and 0.2 for Poisson ratio. The strength of both longitudinal and transverse reinforcements is chosen to be 3000 kg/cm² with
modulus of elasticity of $2.1 \times 10^6 \text{ kg/cm}^2$. The CFRP tensile strength and modulus of elasticity ($E_{frp}$) are $42400 \text{ kg/cm}^2$ and $2.32 \times 10^6 \text{ kg/cm}^2$, respectively. The rupture strain of employed CFRP fibers is 0.18 and their thickness is 0.1375 mm/layer.

| Table 1: Initial and final thicknesses of the FRP jackets for all columns. |
|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|
| Member group    | Member sizes (mm) | Longitudinal reinforcement | FRP thickness (mm) | Initial | Final |
| C1              | 30 30            | 13.5             | 0.000            | 0.412 |
| C2              | 30 30            | 18.0             | 0.000            | 0.678 |
| C3              | 40 40            | 16.0             | 0.000            | 0.678 |
| C4              | 40 40            | 24.0             | 0.000            | 0.963 |
| C5              | 40 40            | 32.0             | 0.000            | 0.963 |
| C6              | 50 50            | 25.0             | 0.000            | 1.513 |
| C7              | 50 50            | 37.5             | 0.000            | 1.788 |
| C8              | 50 50            | 50.0             | 0.000            | 1.788 |
| C9              | 60 60            | 54.0             | 0.000            | 1.513 |
| C10             | 60 60            | 72.0             | 0.000            | 1.788 |

The dead and participating live loads on the stories are $650 \text{ kg/m}^2$ and $200 \text{ kg/m}^2$, respectively. Dead loads, which are exerted by internal partitioning walls, are also participated in abovementioned value of dead loads. Loads that are related to peripheral walls and parapets are assumed to be $700 \text{ kg/m}$ and $250 \text{ kg/m}$. Lateral loads were determined by means of an equivalent static method and are applied in directions as stated in Iranian Seismic Design Code (Standard 2800-05) [7].

9. APPLIED MODEL FOR FRP CONFINED CONCRETE

In the last few years, many studies have been conducted on the stress–strain behavior of FRP-confined concrete and various models have been proposed [11]. However, the stress–strain model for FRP confined rectangular sections, that has been proposed by Teng and Lam appears to be a suitable model for our study as it is simple and it captures the main characteristics of the stress–strain behavior of FRP-confined concrete [12].

Based on this model, the compressive strength and axial rupture strain of FRP-confined concrete in rectangular sections are calculated as described by the following equations [13]:

$$f'_{cc} = f_{cc} \left( 1 + 3.3 k_{st} \frac{f_l}{f_{co}} \right)$$

(9)

$$\frac{\varepsilon'_{cc}}{\varepsilon_{co}} = 1.75 + 12 k_{st} \left( \frac{f_l}{f_{co}} \right) \left( \frac{\varepsilon_{l,sep}}{\varepsilon_{co}} \right)^{0.45}$$

(10)

where, $f_l$ is the equivalent confining pressure, defined as follows:
where $E_{frp}$ is the elastic modulus of the FRP, $B$ and $D$ are dimensions of the rectangular cross-section ($D \geq B$), $\varepsilon_{h,rup}$ is the FRP hoop rupture strain and $t$ is the thickness of FRP fibers.

According to Figure 4, Teng and Lam proposed the following model for determination of the total area of concrete enclosed by the FRP jacket:

$$A_e = \frac{A_c}{1 - \rho_{sc}}$$

where, $\rho_{sc}$ is the cross-sectional area ratio of longitudinal steel and $R_C$ is radius of the rounded corners. These researchers suggested Equations (13) and (14) for the shape factor for strength ($k_{s1}$) and the shape factor for strain ($k_{s2}$):

$$k_{s1} = \left( \frac{B}{D} \right)^2 \frac{A_e}{A_c}$$

$$k_{s2} = \left( \frac{D}{B} \right)^{0.5} \frac{A_e}{A_c}$$

where most of the parameters are the same as previous equations and $A_c$ is the area of the effectively confined concrete and $A_C$ is the total area of concrete enclosed by the FRP jacket.
10. PUSHOVER ANALYSIS

As mentioned before, the models were analyzed using SAP2000 [10], which is a general-purpose structural analysis program for static and dynamic analyses of structures. For nonlinear analysis of initial models, axial force–moment hinges and pure moment hinges are assigned to the ends of beams and columns, respectively. Rupture strains and rotations of plastic hinges could be evaluated using the stated nonlinear static criteria in [1-3].

The moment-rotation relationships of plastic hinges are similar to the moment-rotation relationship of moment hinges except that they are compatible with the moment-axial load interaction curves. The moment-axial load interaction curves of columns Figures could be determined using principal theories. Effect of FRP confinement on increase of strength and ductility of the columns could be concerned in interaction curves. Figure 6 shows an example of the moment-axial load interaction curves for a rectangular column with dimensions of 40×40 cm.

For performing a pushover analysis, two kinds of load distributions are utilized in this study:

a) Distribution type I; distribution is proportional to the lateral loads that have been calculated from a linear spectral dynamic analysis.

b) Distribution type II; distribution is uniform where the lateral loads are proportional to the weight of each story.

For better clarifying the results, which are presented in Table 2, Figure 7 shows the base shear-top displacement relationship for the non-retrofitted frame (labeled as “initial”) and the retrofitted frame (labeled as “final”). It is evident from Figures and tables that confinement of reinforced concrete columns with FRP fibers would increase their strength and ductility by 19 and 38 percents respectively. Therefore, this method could be considered as a major way for enhancing seismic performance of concrete structures.

![Figure 6. The moment-axial load interaction curves for a rectangular FRP-confined columns with dimensions of 40×40 cm for (a) ρ=1% and (b) ρ=2%](image)

In Table (2) effective parameters such as ductility factor \( R_\mu \), overstrength factor \( R_S \) and safety factor \( Y \) were obtained for evaluating the reduction factor. Using these values and by means of Eq. (8) the reduction factor of RC buildings could be
calculated.

Table 2: Different parameters of reduction factor for models

<table>
<thead>
<tr>
<th>Model</th>
<th>Design</th>
<th>Yield</th>
<th>Ultimate</th>
<th>μ</th>
<th>R_μ</th>
<th>R_s</th>
<th>Y</th>
<th>R_w</th>
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<tr>
<td></td>
<td>V_w,</td>
<td>V_v,</td>
<td>Δ_y,</td>
<td></td>
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<td></td>
<td></td>
</tr>
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<td>V_ton</td>
<td>ton</td>
<td>cm</td>
<td>cm</td>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>4S-O-1*</td>
<td>66.7</td>
<td>198.9</td>
<td>7.9</td>
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<td>1.7</td>
<td>1.7</td>
<td>3.0</td>
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</tr>
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<td>43.1</td>
<td>1.7</td>
<td>1.7</td>
<td>3.9</td>
</tr>
</tbody>
</table>

*Note: Model identification is "number of story – non retrofit (O) / retrofit (R) – lateral load type."
In Table 3, reduction factor values for each of the models under both lateral load distributions are represented. As it is apparent, average of reduction factor value for non-retrofitted reinforced concrete buildings is 6.7 and is 9.9 for retrofitted RC buildings.

<table>
<thead>
<tr>
<th>Story</th>
<th>Non-retrofitted Lateral Load Type I</th>
<th>Non-retrofitted Lateral Load Type II</th>
<th>Retrofitted Lateral Load Type I</th>
<th>Retrofitted Lateral Load Type II</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>7.1</td>
<td>7.0</td>
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<td>10</td>
<td>6.5</td>
<td>6.9</td>
<td>9.3</td>
<td>9.2</td>
</tr>
<tr>
<td>Average</td>
<td>6.7</td>
<td></td>
<td>9.9</td>
<td></td>
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</tbody>
</table>

11. CONCLUSIONS
The results derived from the nonlinear static analyses of initial models, which were based on [7], are stated as below:
1. Confinement of reinforced concrete columns would increase their strength and ductility by 19 and 38 percents, respectively.
2. Reduction factor of a non-retrofitted reinforced concrete building without shear walls is evaluated to be 6.7 but with confinement of columns with CFRP jacket, this factor would increase to 9.9.
3. Application of CFRP fibers for confinement of reinforced concrete columns would increase their resistance, ductility and capacity of seismic energy dissipation. Moreover, it would move the failure point from columns to beams.
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CONCRETE SHEARE WALLS STRENGTHENING STRUCTURAL ELEMENTS FOR SEISMIC REHABILITATION OF MASONRY BUILDINGS

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ABSTRACT
This paper aims at investigating the behavior of masonry buildings strengthened with concrete shear walls. To this end, a masonry wall with openings extracted from an individual two-storey masonry building is modeled using ANSYS finite element software. A number of nonlinear static analyses were conducted on the model. The nonlinear behavior of the existing model was compared with that of the model enhanced with concrete shear wall. The results obtained from the numerical analyses confirm that the load-carrying capacity of the unreinforced masonry buildings degrades due to the global failure of the wall. Whereas, the masonry buildings strengthened with concrete shear wall may achieve their target displacement. Moreover, the results reveal that the wall load-carrying capacity may be upgraded, if the added shear wall is connected to both the diaphragm and adjacent masonry wall.

Keywords: rehabilitation, masonry building, concrete shear wall, push-over analysis

1. INTRODUCTION
The masonry buildings have extensively been constructed in Iran for low-rise buildings, because of their cost-effectiveness. Most of them have been designed regardless of the effect of earthquake induced loads. In other words, these kinds of buildings are non-engineered structures which may be susceptible to a significant risk during an earthquake.

Among the points of weakness associated with the masonry buildings, the lack of appropriate tying system, improper masonry elements, large openings, etc. are noteworthy.

Since many of the important buildings such as schools are structured with unreinforced masonry buildings, it is necessary to rehabilitate those using proper solutions.

One of the methods used for seismic rehabilitation of masonry buildings is known as simple rehabilitation method [1]. In this method, the building weaknesses are retrofitted one by one. Another method is coating a reinforced concrete layer on the masonry walls. By using this method, the lateral load-carrying capacity of structure is enhanced. In the above-mentioned methods providing integrity for the
diaphragm is mandatory. Among the other methods used for rehabilitation of masonry buildings, the use of an external lateral resisting system is noteworthy. In this method, the lateral load-carrying capacity of the structure is mainly provided with the added lateral resisting elements. Another method that can be used is structure of an exposed lateral seismic load-carrying system. For this purpose, a concrete shear wall, well known as an effective lateral resisting system, can be used. Providing appropriate integrity among the added shear walls, the existing masonry walls and the diaphragm is a key concern in this method. The mobilization of induced actions among these elements is another concern.

2. CASE STUDY
In this study, an existing two-storey masonry building with an area of 829 m² is investigated (see Figure 1).

![Figure 1. Typical floor plan](image)

In order to assess the structural system of the building, a set of destructive and non-destructive inspection program was prescribed. This inspection program indicated that the building possesses unreinforced masonry system, having some rawbacks, including:

a. Inappropriate tying system
b. Improper bricking
c. Lack of diaphragm integrity

The above drawbacks cause the building to be a seismic vulnerable structural system. Several methods were investigated for seismic rehabilitation of the building [2]. The methods were compared with each other regarding the technical
and economical aspects. Among them, the method of adding concrete shear wall was assessed as the best solution. According to the initial studies, four L-shape shear walls were provided as shown in Figure 2.

![Concrete shear walls position](image)

Figure 2. Concrete shear walls position

In order to study the effect of adding concrete shear wall on lateral behavior of the masonry building, the nonlinear behavior of one of the perimeter axis (Ax1) was evaluated using ANSYS finite element software. The lateral behavior of this axis was studied in its existing situation as well as in presence of concrete shear walls. Since the number of shear walls is limited, the diaphragm rigidity should be provided. The diaphragm rigidity conditions may be satisfied using such details as shown in Figure 3.

![Details for providing diaphragm rigidity](image)

Figure 3. Details for providing diaphragm rigidity
Since the gross mass of the masonry buildings has mainly arisen from the masonry walls, two details were utilized to connect the concrete shear walls to the building: Firstly, connecting the shear walls to both the diaphragm and masonry walls of the building; secondly, connecting the shear walls to the diaphragm, only. In essence, in first detailing, the lateral force is transmitted to the shear walls through the diaphragm while both the diaphragm and masonry walls contribute to the transition of lateral forces to the shear walls. The proposed details for connection of the concrete shear walls to the diaphragms and to the adjacent masonry walls are illustrated in Figures 4 and 5, respectively. Also the details shown in Figure 6 were proposed for transferring the lateral forces from shear wall to the earth.

![Figure 4. Shear wall-to-diaphragm connection](image4)

![Figure 5. Proposed details to connect concrete shear wall to masonry one](image5)
3. FAILURE MECHANISM AND MATERIAL MODELS FOR UNREINFORCED MASONRY STRUCTURES

Masonry wall is a composite structure that consists of brick and mortar. The geometry, axial loading and material properties, play an essential role in the response of the wall and in the mechanism of failure. Figure 7 shows different in-plane failure modes for masonry walls.

Several researchers have proposed failure criteria for masonry material. Page [7] suggested a microscopic finite element model for masonry, considering elastic elements for bricks and link elements for joints. Although this model could present global nonlinear behavior of masonry walls and crack distribution, it failed to express the failure in bricks and the effect of multi-axial stresses on the response. Ganz [8] also presented a failure criterion for masonry under biaxial compressive stresses, neglecting the tensile strength.

William and Warkne [9] developed a constitutive model for tri-axial behavior of brittle materials like concrete. The model considers multi-axial stresses of brittle material and takes into account cracking, crushing and sliding phenomena by reflecting their effects on the stiffness matrix [4, 9].

The general form of William and Warkne's theory has been used here. The above model as has been developed in ANSYS, has also been recommended by [3] for
masonry material. In another study, Kumar and Bhandari adopted a similar model for masonry arches [10].

4. MODELING
As previously mentioned, to study the behavior of masonry building strengthened with concrete shear wall, a typical perimeter axis (Axe 1) is employed. Nonlinear static analysis was carried out using ANSYS finite element software. Nonlinear material was assigned to the masonry and concrete elements. To this end, William-Warkne failure criterion was employed for materials. The element SOLID65 was used to model masonry and concrete walls. This element has capability to idealize brittle materials for cracking and crushing phenomenon. The element SHELL63 was applied for the modeling of horizontal and vertical ties. Figure 8 shows a scheme of the model.

The mechanical properties of the brick and mortar employed in the building were obtained according to the test results. The test results are summarized in Table 1.

<table>
<thead>
<tr>
<th>$f_{cb}$</th>
<th>$f'_{cb}$</th>
<th>$V_t$</th>
<th>$V_s$</th>
</tr>
</thead>
<tbody>
<tr>
<td>MPa</td>
<td>MPa</td>
<td>MPa</td>
<td>MPa</td>
</tr>
<tr>
<td>10</td>
<td>1.2</td>
<td>0.18</td>
<td>0.095</td>
</tr>
</tbody>
</table>

In the above table:
$f_{cb}$: compressive strength of bricks
$f'_{cb}$: compressive strength of masonry walls
$V_t$: allowable tension stress
$V_s$: allowable shear stress

5. ANALYSIS PROCEDURE
In order to identify the nonlinear behavior of the building, a nonlinear static analysis, well known as Push-over analysis, was conducted on the model. Since the height to length ratio of the wall is low, the first deformation pattern which is well
adapted to the first mode of vibration was applied to the model. For this purpose a prescribed displacement was applied to top of the building. The target displacement was obtained according to FEMA 356. In the following, the target displacement is calculated for the model with and without strengthening elements:

a. Existing masonry wall:

\[
T = \frac{2\pi}{\sqrt{k/m}} = \frac{2\pi}{\sqrt{30000/16}} = 0.145 \text{sec}
\]

\[
\delta_i = C_0 C_z C_y S_y \frac{T^2}{4\pi^2} = 1.2 \times 1 \times 1 \times (0.3 \times 2.5) x \frac{0.145^2}{4\pi^2} = 0.005 m
\]

b. Strengthened masonry wall:

\[
T = \frac{2\pi}{\sqrt{k/m}} = \frac{2\pi}{\sqrt{50000/16}} = 0.112 \text{sec}
\]

\[
\delta_i = C_0 C_z C_y S_y \frac{T^2}{4\pi^2} = 1.2 \times 1 \times 1 \times (0.3 \times 2.5) x \frac{0.112^2}{4\pi^2} = 0.003 m
\]

The parameters \(k\) and \(m\) denote the wall stiffness and the wall mass, respectively. The wall stiffness was estimated from the initial linear analysis.

6. ANALYSIS RESULTS

The crack pattern shown in Figure 9 illustrates that under a 5-mm target displacement, a major failure had taken place in the existing masonry wall. While in the strengthened masonry wall, the minor cracks are localized around the openings under its target displacement (Figure 10).

The in-plane shearing stress contour shown in Figure 11 indicates that the applied acts are localized mostly within the concrete shear walls, and the shearing stress is nearly uniform within the masonry elements. The maximum shearing stress applied to the masonry element in the strengthened wall is, approximately, half that in the existing wall.
Figure 10. Crack pattern in the strengthened masonry wall

Figure 11. In-plane shearing stress contour in the existing masonry wall

Figure 12. In-plane shearing stress contour in the strengthened masonry wall
In Figure 13, the base shear is depicted versus the top displacement for all the models. As observed, the strengthened masonry models show more stiffness and strength in comparison with the existing model. Furthermore, as previously noted, the added shear walls are more efficient if connected to both the diaphragms and masonry walls. In essence, as the added shear walls are attached to the diaphragms only, a significant promotion is not achieved. This is due to the considerable mass of the walls in the masonry buildings. In case of connecting shear walls to both the diaphragms and masonry walls, the lateral strength and initial stiffness is increased by 50% and 100%, respectively.

7. CONCLUSION
This paper dealt with one of the efficient methods for seismic rehabilitation of masonry buildings. Through the nonlinear analyses performed using ANSYS finite element software, it was confirmed that the lateral behavior of masonry buildings may be promoted by providing concrete shear walls. According to the numerical results, the added concrete shear walls become much more efficient if connected to both the diaphragms and adjacent masonry walls. The analysis results indicate the load-carrying capacity of the existing masonry walls is thoroughly deteriorated due to the major cracks. While, in the strengthened masonry walls the cracks are minor and localized round the openings. In addition, the concrete shear walls and masonry ones can contribute to the lateral forces, resulting in reduction of shear demand on the masonry walls. Also, in case of connecting shear walls to both the diaphragms and masonry walls the lateral strength and initial stiffness is increased by 50% and 100%, respectively.

REFERENCES
1. FEMA 356 Seismic rehabilitation prestandard, Federal Emergency


F.E MODELING OF NORMAL AND SELF-CONSOLIDATING RC BEAMS AND EXPERIMENTAL BENDING BEHAVIOR OF SCC BEAMS

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ABSTRACT
Development of a new generation of concrete, self-consolidating concrete, SCC is a very desirable achievement in the RC structures for overcoming problems associated with many problems such as congestions of steel reinforcement. This non-vibrating concrete is not affected by the skill of workers, and shape and amount of reinforcing bar arrangement of a structure. Due to high-fluidity and resisting power of reinforcing of SCC, it can be pumped longer distances. In this research, the finite element, F.E modeling of four normal concrete (vibrating concrete), NC and SCC beams in bending is performed and the results are compared together. For the experimental phase, the results of available tested beams of dimensions 20 cm width, 30 cm height and 300 cm length are used. For modeling longitudinal steel bars and concrete, the 2-node and 8-node 3-D elements, are used respectively. The 8-node element has ability to consider cracking in tension and concrete crushing in compression. The deflection, cracking, yield and ultimate loads, as well as beams ductility are compared numerically and experimentally. The comparison of results obtained by two methods indicates that a satisfactory agreement is achieved.

Keywords: self-consolidating and normal concrete, bending behavior, finite element modeling

1. INTRODUCTION
Concrete structural components require the understanding of the responses of those components to a variety of loading. There are a number of methods for modeling the concrete structures through both analytical and numerical approaches. Finite element analysis (FEA) is a numerical one widely applied to the concrete structures based on the use of the nonlinear behavior of materials. FEA provides a tool that can simulate and predict the responses of reinforced concrete members. The use of FEA has increased because of progressing knowledge and capability of computer package and hardware. Any attempts for engineering analyses can be done conveniently and fast using such versatile FEA packages.
Self-consolidating concrete (non vibrating concrete), SCC is a new type of concrete that is able to flow and compact under its own weight and completely fill the formwork even in the presence of dense reinforcement, whilst maintaining
homogeneity and without the need for any additional compaction. This has generated
tremendous interest since the initial development in Japan by Okamura [1] in the
1980s in order to reach durable concrete structures. Since that time, Japanese
contractors have used SCC in different applications. In contrast with Japan, research
in Europe, America and Iran started only recently [2, 3]. The advantages of SCC over
normal concrete (vibrating concrete), is that NC offers many benefits to the
construction practice; the elimination of the compaction work results in reduced costs
of placement, equipment needed on construction, time and improved quality control
[4]. Therefore, a comparison of these two types of concrete, i.e. NC and SCC in
bending can be interesting, especially for practical engineers.

2. MATERIALS PROPERTIES MODELING

2.1. Reinforced Concrete

An eight-node solid element, solid65, was used to model the concrete. The solid
element has eight nodes with three degrees of freedom at each node-translation in
the nodal x, y, and z directions. The element is capable of plastic information,
cracking in three orthogonal directions, and crushing [5]. The geometry and node
locations for this element type are shown in Figure 1.

![Figure 1. Solid65-3-D reinforced concrete solid [6]](image)

2.1.1. Concrete Mechanical Properties

2.1.1.1. Self-Consolidating Concrete

Development of a model for the behaviour of concrete is a challenging task.
Concrete is known as a quasi-brittle material and has a different behaviour in
compression and tension. In this research, the graph of nonlinear-isotropic stress-
strain of SCC is obtained by the first author from results of compression test of
concrete specimens in the laboratory with the help of embedded sensor (Figure 2).

![Figure 2. Self-consolidating concrete stress-strain diagram](image)
2.1.1.2. Normal Concrete

For NC, Saenz-Smith relationship has been used to introduce stress-strain diagram, science convergence plays the main role. Therefore, ascending branch of stress-strain curve of Saenz relationship [7] and for descending branch, Smith and Young relationship [8] have been used (Figure 3).

\[ f'_c = \frac{E_{dc} \varepsilon'_c}{1 + \left(\frac{E_{dc}}{E_{sc}} - 2\right) \left(\frac{\varepsilon_c}{\varepsilon'_c}\right) + \left(\frac{\varepsilon_c}{\varepsilon'_c}\right)^2} \]  

Equation 1

\[ f_c = f'_c \left(\frac{\varepsilon_c}{\varepsilon'_c}\right) \exp\left(1 - \frac{\varepsilon_c}{\varepsilon'_c}\right) \]  

Equation 2

\[ E_{sc} = f'_c / \varepsilon'_c \quad \text{(MPa)} \]

Eqs (1,2), represent the relations between \( f'_c, \varepsilon'_c, E_0 \) and \( E_{sc} \), which can be found from stress-strain curve and in this paper, the ascending and descending branches of curve have been acquired of equations (1) and (2), respectively.

![Figure 3: Normal concrete stress-strain diagram](image)

2.1.2. Steel Reinforcement

A link8 element was used to model the steel reinforcement. Two nodes are required for this element. Each node has three degrees of freedom, translations in the nodal x, y, and z directions. The element is also capable of carrying plastic deformation. The geometry and node locations for this element type are shown in Figure 4.

![Figure 4. Link8 3-D spar](image)
Here, the stress-strain curve for steel reinforcement used in SCC beams was obtained from steel bars tested in tension. The curve has an initial linear elastic portion, a yield plateau (yield point beyond which the strain increases with little or no increase in stress), a strain-hardening range in which stress again increases with strain and finally a range in which the stress drops off until fracture occurs which has been shown in Figure 5. The following relationships used in this study are those obtained by testing the bars in tension and the stress-strain curve for steel reinforcement has the following characteristics:

1: elastic portion; \[ \varepsilon_s < \varepsilon_y \implies f_s = E_s \varepsilon_s \]  

2: yield plateau; \[ \varepsilon_y \leq \varepsilon_s \leq \varepsilon_{sh} \implies f_s = f_y \]  

3: strain hardening; \[ \varepsilon_{sh} < \varepsilon_s \leq \varepsilon_u \implies f_s = f_y + \left( f_u - f_y \right) \left[ 2 \left( \frac{\varepsilon_s - \varepsilon_{sh}}{\varepsilon_u - \varepsilon_{sh}} \right)^2 \right] \left( \frac{\varepsilon_s - \varepsilon_{sh}}{\varepsilon_u - \varepsilon_{sh}} \right) \]  

Figure 5. Stress-strain diagram of tested tensile steel

Material properties for the steel reinforcement \((\varepsilon_y, \varepsilon_{sh}, \varepsilon_u, f_y, f_u)\) and concrete \((f'_c, \varepsilon'_c)\) are obtained (Figures 2, 3, 5).

3. FINITE ELEMENT MODELING

As an initial step, a FEA requires meshing of the model. In other words, an important step in FE modelling is the selection of the mesh density. A convergence of results for steel reinforcement and concrete is obtained when an adequate number of elements are used in the model; this is practically achieved when an increase in the mesh density has a negligible effect on results. The ANSYS software has been performed and the results are shown in Figures 6, 7.
4. BEAMS PROPERTIES
Four simply supported reinforced SCC beams (SCCB1-SCCB4) with 300*200 dimension and 3000mm length were tested [9] under two point loading (statistical increasing) with a constant moment region (Figure 8). All beams were designed for the shear span to depth ratio of 3.5. The clear cover for the tested beams was maintained at a minimum of 25mm. Different types of electrical and mechanical strain gauges were attached on the steel bars and concrete surface by the first author. Also, the LVDTs were placed at different locations of beams. During the test, the readings of these sensors were recorded by the data logger.
The beams details are presented in Table 1. The typical test set up for the tested SCC beams is shown in Figure 9.

Table 1: Details of testing program of tested beams [9]

<table>
<thead>
<tr>
<th>Beam No.</th>
<th>$f'_{c}$ (MPa)</th>
<th>d (mm)</th>
<th>$d'$ (mm)</th>
<th>$A_{s}$</th>
<th>$\rho/\rho_{b}$</th>
<th>$A'_{s}$</th>
<th>$\gamma_{c}$ (KN/m$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SCCB1</td>
<td>33.0</td>
<td>256</td>
<td>42.9</td>
<td>2Φ18+1Φ16+1Φ14</td>
<td>0.511</td>
<td>2Φ14+1Φ18</td>
<td>22.80</td>
</tr>
<tr>
<td>SCCB2</td>
<td>31.5</td>
<td>255</td>
<td>43.5</td>
<td>4Φ20</td>
<td>0.746</td>
<td>2Φ14+1Φ20</td>
<td>23.03</td>
</tr>
<tr>
<td>SCCB3</td>
<td>35.0</td>
<td>254</td>
<td>45.4</td>
<td>4Φ22</td>
<td>0.91</td>
<td>2Φ14+1Φ25</td>
<td>22.60</td>
</tr>
<tr>
<td>SCCB4</td>
<td>25.0</td>
<td>251</td>
<td>42.0</td>
<td>4Φ28</td>
<td>1.48</td>
<td>2Φ14</td>
<td>22.30</td>
</tr>
</tbody>
</table>

The SCC mix was designed by the first author and the range of fresh properties is summarized in Table 2. It was found that the SCC was consolidated exceptionally well under its own weight for four specimens.
Table 2: Test results of fresh concrete

<table>
<thead>
<tr>
<th>L-box</th>
<th>V-funnel</th>
<th>J-ring</th>
<th>Slump flow</th>
</tr>
</thead>
<tbody>
<tr>
<td>$h_1/h_2$</td>
<td>$t$ (sec)</td>
<td>$t$ (sec)</td>
<td>$h_2-h_1$ (mm)</td>
</tr>
<tr>
<td>0.83</td>
<td>0.40</td>
<td>7.5</td>
<td>13</td>
</tr>
</tbody>
</table>

5. COMPARISON OF RESULTS

The comparison of stress-strain diagrams of SCC and NC are shown in Figure 10.

![Figure 10. SCC and NC concrete stress-strain diagram](image)

The numerical and experimental deflections are compared for two types of concrete and the results are shown in Figures 11, 12.

![Figure 11. Load-deflection curves of experimental and numerical SCC beams](image)
The comparison results of deflections, ductility, cracking, yielding and ultimate loadings and F.E modeling of SCC and NC beams including the percentages of errors are presented in Tables 3-5.

**Table 3: Comparison of deflections and ductility between experimental and numerical results of SCC beams**

<table>
<thead>
<tr>
<th>Beam No</th>
<th>Experimental Results</th>
<th>Numerical Results</th>
<th>Error for $\Delta y$ (%)</th>
<th>Error for $\Delta u$ (%)</th>
<th>Error for $\mu_d$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SCCB1</td>
<td>$\Delta y = 10.73$</td>
<td>$\Delta u = 92.2$</td>
<td>9.5</td>
<td>80</td>
<td>10.85</td>
</tr>
<tr>
<td>SCCB2</td>
<td>$\Delta y = 12.48$</td>
<td>$\Delta u = 47.58$</td>
<td>11.3</td>
<td>42</td>
<td>10.25</td>
</tr>
<tr>
<td>SCCB3</td>
<td>$\Delta y = 12.4$</td>
<td>$\Delta u = 43.43$</td>
<td>11.6</td>
<td>39</td>
<td>9.7</td>
</tr>
<tr>
<td>SCCB4</td>
<td>$\Delta y = 28.79$</td>
<td>$\Delta u = 28.79$</td>
<td>25.8</td>
<td>25.8</td>
<td>-</td>
</tr>
</tbody>
</table>

**Table 4: Comparison of cracking, yielding and ultimate loads between F.E model of SCC and NC beams**

<table>
<thead>
<tr>
<th>Beam No</th>
<th>F.E model of SCC beam</th>
<th>F.E model of NC beam</th>
<th>Error (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1</td>
<td>$P_y = 18.5$</td>
<td>$P_y = 140$</td>
<td>$P_y = 172$</td>
</tr>
<tr>
<td>B2</td>
<td>$P_y = 11$</td>
<td>$P_y = 190$</td>
<td>$P_y = 231$</td>
</tr>
<tr>
<td>B3</td>
<td>$P_y = 17$</td>
<td>$P_y = 219$</td>
<td>$P_y = 268$</td>
</tr>
<tr>
<td>B4</td>
<td>$P_y = 20$</td>
<td>$P_y = 400$</td>
<td>$P_y = 400$</td>
</tr>
</tbody>
</table>

Figure 12. Load-deflection curves of F.E model of SCC and NC beams
Table 5: Comparison of deflections and ductility between F.E model of SCC and NC beams

<table>
<thead>
<tr>
<th>Beam No</th>
<th>F.E model of SCC beam</th>
<th>F.E model of NC beam</th>
<th>$\mu_{num}^{SCC}$</th>
<th>$\mu_{num}^{NC}$</th>
<th>Error for $\mu_d$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1</td>
<td>9.5 80</td>
<td>9.8 78.2</td>
<td>8.42 8</td>
<td>8</td>
<td>5</td>
</tr>
<tr>
<td>B2</td>
<td>11.3 42</td>
<td>11.7 40.8</td>
<td>3.71 3.48</td>
<td>7</td>
<td></td>
</tr>
<tr>
<td>B3</td>
<td>11.6 39</td>
<td>12.3 37.5</td>
<td>3.38 3.05</td>
<td>9.7</td>
<td></td>
</tr>
<tr>
<td>B4</td>
<td>25.8 25.8</td>
<td>24.2 24.2</td>
<td>1 1</td>
<td>-</td>
<td></td>
</tr>
</tbody>
</table>

6. CONCLUSIONS

The following conclusions can be drawn:

Comparison of numerical and experimental results of yield deflection ($\Delta y$), with a percentage error range of 5.1 to 10.8 is an indication of satisfactory results of F.E modelling.

Except beam SCC4 which is reinforced with a high amount of steel reinforcement, all other three tested SCC beams achieved an experimental minimum ductility index, $\mu_{dexp}$ value of 3.5. In other words this an acceptable minimum value for reinforcement of NC beams suggested in seismic regions. It is therefore, concluded that, if reinforced SCC beams are well designed in bending, it can impose sufficient ductility at ultimate loads.

In a comparison of numerical modelling of SCC and NC it was concluded that, the cracking loads in SCC beams are slightly lower than the NC beams. However, the ultimate and yielding loads of SCC beams are slightly higher than the NC beams. The SCC beams are shown to be more ductile than the NC beams both experimentally as well as numerically.

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SEISMIC FINITE ELEMENT ANALYSIS OF IRREGULAR REINFORCED CONCRETE BUILDINGS IN ELEVATION

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ABSTRACT
Near field ground motions are different from ordinary ground motions in that they often contain strong coherent dynamic long period pulses and permanent ground displacements. The dynamic motions are dominated by a large long period pulse of motion that occurs on the horizontal component perpendicular to the strike of the fault, caused by rupture directivity effects. This paper addresses multistorey reinforced concrete buildings, irregular in elevation under near field earthquakes. Two twelve-story buildings with two and four large setbacks in the upper floors respectively, as well as a third one, regular in elevation, have been designed to the provisions of ACI 318-2005 for the high (DCH) and medium (DCM) ductility classes, and the same peak ground acceleration (PGA) and material characteristics. All buildings have been subjected to inelastic dynamic time-history analysis for selected input motions. The assessment of the seismic performance is based on both global and local criteria. It is concluded that irregular buildings demonstrate poor seismic performance in terms of ductility and energy dissipation capacity during severe seismic events. As expected, DCM buildings are found to be stronger and less ductile than the corresponding DCH ones.

Keywords: reinforced concrete buildings; irregularity in elevation; near field; setbacks; seismic performance; time-history analysis

1. INTRODUCTION
Recent major earthquakes (Northridge 1994, Kobe 1995, Chi-chi 1999 and Bam 2003, etc.) have shown that many near-fault ground motions possess prominent acceleration pulses. Some of the prominent ground acceleration pulses are related to the large ground velocity pulses, others are caused by mechanisms that are totally different from those causing the velocity pulses or fling steps. Near fault ground motions, which have caused severe damages in recent disastrous earthquakes, are characterized by a short-duration impulsive motion that will transmit large energy into the structures at the beginning of the earthquake.

The paper addresses multistorey reinforced concrete (RC) frame buildings with setbacks, i.e. a reduction of the length of the building along its height (irregularity in elevation). It focuses on buildings with large setbacks in the upper floors. Irregular configurations either in plan or in elevation were often recognized as
one of the main causes of failure during past earthquakes. Focusing on buildings with setbacks, observed damage after strong earthquakes indicates an inferior performance of this type of structure [1]. Experimental [2, 3] as well as analytical [4] studies involving frames with setbacks designed and detailed to modern codes such as the ACI 318-2005 [5] showed a quite satisfactory seismic performance of this type of structure. Nevertheless, the seismic behavior of reinforced concrete multistorey buildings with setbacks under the near field earthquake has not yet been studied.

The present paper focuses on the seismic performance of multistorey RC frame buildings with setbacks in the upper stories, designed to the provisions of the ACI 318-2005. In order to examine the influence of the design ductility class on the seismic behavior of the buildings, all frames were designed for both the high (DCH) and the medium (DCM) ductility classes. Buildings designed for the low (L) ductility class with low dissipation capacity and low ductility have not been examined here since they are only recommended for low seismicity areas.

2. DESIGN CONSIDERATION

Six twelve story reinforced concrete (R.C.) frame buildings were designed according to the requirements of ACI 318-2005[8], three of them (FRH, FRH-1 and FRH-2) for the high (DCH) ductility class and the rest (FRM, FRM-1 and FRM-2) for the medium (DCM) ductility class, with the same materials and the same peak ground acceleration. The geometry of the typical plane frames of all buildings are shown in Figure 1. Two of them (FRH and FRM) correspond to buildings regular in elevation, without any setbacks, as shown in Figure 1. The other four frames have the same configuration in the lower eight stories and large setbacks (about 40% of the length of the lower storey) in the upper ones, two of them (FRH-1 and FRM-1) in the upper two, and the rest (FRH-2 and FRM-2) in the upper four stories.
Irregular frames FRH-1, FRH-2, FRM-1 and FRM-2 were designed with the aid of modal response spectrum analysis, whereas in the cases of the regular FRH and FRM frames the (static) ‘lateral force method of analysis’ was used. The first 4 modes of vibration were considered in the multimodal analysis of all irregular frames, with total contributing masses of more than 90% in all cases. The natural periods of the frames were found to be 1.21 s, 1.12 s and 1.10 s for FRH, FRH-1 and FRH-2 respectively, and 1.21 s, 1.12 s and 1.07 s for FRM, FRM-1 and FRM-2 respectively. The, strange at first sight, fact that stiffer frames have longer natural periods than the less stiff ones can be attributed to the reduction of mass (because of the setbacks) at a rate greater than the one of stiffness.

The results of a detailed estimation of the required steel quantities and concrete volume show that FRM and FRM-1 have exactly the same cross-section dimensions with FRH and FRH-1 respectively, while the only difference between FRM-2 and FRH-2 concerns the cross-section dimensions of the interior columns of the lower four stories, which results in a 4% increase of required concrete volume in FRM-2 in comparison with FRH-2.

DCH structures generally require less longitudinal and more transverse reinforcement than the corresponding DCM ones. This is a very significant effect of the ductility class, the clear trend being that the percentage of longitudinal steel decreases, while that of the transverse reinforcement increases with increasing ductility class. The difference is more pronounced in irregular structures. On the other hand, ductility class seems not to affect significantly the total amount of the required reinforcement. Based on the foregoing comparisons, it appears that from the economical point of view both ductility classes for medium and high seismicity areas are essentially equivalent.
3. ASSESSMENT OF SEISMIC PERFORMANCE - PROCEDURE

All frames have been subjected to both inelastic dynamic time-history analysis and inelastic static pushover analysis. Inelastic dynamic time-history analysis of the structures was carried out with the aid of the IDARC computer code [6], including several new features and elements, as well as capability for seismic reliability analysis. The dynamic input has been given as a ground acceleration time-history which was applied uniformly at all the points of the base of the structure; only one (horizontal) component of the ground motion has been considered while dynamic soil–structure interaction was neglected. P–Δ effects were considered.

Inelastic static pushover analysis was carried out with the aid of the well-known ETABS000 computer code [7]. Two vertical distributions of the lateral loads were applied: a ‘uniform’ pattern, based on lateral forces proportional to mass regardless of elevation, and a ‘modal’ pattern, proportional to the story lateral forces given by the multimodal analysis. In each case, the ‘target displacement’ was defined as the seismic demand derived from the elastic response spectrum.

The possibility of failure in each member, as well as in each story of the structures, was checked by applying appropriate global, as well as local, failure criteria. Global failure was assumed to coincide with story failure; a dual criterion based on a limiting interstorey drift of 2% and the simultaneous development of a sidesway collapse mechanism involving all vertical members was adopted for assessing storey failure; regardless of mechanism formation, a structure was assumed to have collapsed if the interstorey drift at any location exceeded a limiting value of 3% [8].

The input motions used in this study were two horizontal components of the records from the earthquakes of Naghan (1978), Tabas (1980), Manjil (1986) and Bam (2003), which are among the ones that caused the most serious damage, including collapses and casualties, during the past forty years. All these records are characterized by the fact that they come from surface earthquakes with small epicentral distances (representing the typical destructive earthquakes in Iran). The time–acceleration diagrams are plotted in Figure 2.
Figure 2. Time histories of input accelerograms. (a) Naghan; (b) Tabas; (c) Manjil (d) Bam

4. ASSESSMENT OF SEISMIC PERFORMANCE - RESULTS
Figure 3 summarizes the interstorey drift ratios for the DCH frame structures of Figure 1 for the ‘design’ and the ‘collapse prevention’ earthquake. These results represent the mean values of the drift ratios resulting from any of the eight input motions (every earthquake with two directions) used in the inelastic dynamic time-history analysis. It is permitted to consider mean values if the response is obtained from at least seven records.
As can be seen in Figure 3, interstorey drifts of the irregular frames are quite low, not exceeding 0.40% for the design earthquake and 1.0% for the collapse prevention one, which are well below the adopted failure values (2%-3%). These values can be considered as very satisfactory, particularly for pure frame multistorey structures. Comparing with the regular frames, the interstorey drifts of the irregular frames seem to be similar or even lower, mainly because of the lower natural periods. Exceptions to this trend are the interstorey drifts of FRH-2 and FRM-2 in the upper two stories where the stiffness has been drastically decreased. On the other hand, interstorey drift ratio values of both ductility levels seem to be similar enough, with the values of DCM frames being lower than the corresponding of the DCH ones at the lower stories and higher at the upper ones.

5. CONCLUSIONS
The following remarks that resulted from the analysis of two-dimensional 12-story plane frames strictly apply only in the case of medium-to-high-rise buildings, irregular in elevation but regular in plan, similar to those studied in this paper. For other structures, in particular those with irregularities in plan and for low-rise or very tall buildings, further studies have to be carried out to check the validity of the remarks made herein.

The seismic performance of the studied multistorey reinforced concrete frame buildings with setbacks in the upper stories, designed for the high (DCH) and the medium (DCM) ductility level, can be considered satisfactory, not inferior and in some cases even superior to that of the regular ones, even for motions twice as strong as the design earthquake.

Interstorey drift ratios of irregular frames were found to remain quite low even in
the case of the ‘collapse prevention’ earthquake with an intensity double that of the ‘design’ one. This fact, combined with the limited plastic hinge formation in columns, exclude the possibility of formation of a collapse mechanism.

Most of the input energy of the irregular frames is dissipated in beams where plastic hinges form. Plastic hinges in columns do appear in the case of an earthquake with twice the design intensity, but the ductility requirements seem to be much lower than the available values.

Irregular structures seem to be stronger. On the other hand, all DCM frame structures seem to be, as expected, stronger and less ductile than the corresponding DCH ones.

As far as the effect of ductility class is concerned, buildings of both ductility classes seem to perform equally satisfactorily during the design earthquake. However, a potential weakness in the shear capacity of DCH beams (mainly those of the regular structure) has been detected.

From the economical point of view, differences between DCH and DCM design seem to be very small, even negligible, and no clear trends were detected, other than that the percentage of longitudinal bars in the total reinforcement decreases while that of the transverse reinforcement increases with increasing ductility class.

REFERENCES

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NUMERICAL STUDY OF THE DIAPHRAGM BEHAVIOR OF THE COMPOSITE FLOOR SYSTEMS SUBJECTED TO LATERAL LOAD

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ABSTRACT
The influence of the in-plane flexibility of composite floor systems on the seismic response of the structures may become significant, particularly when considerable floor slab cracking and yielding are expected. As in recent years the use of composite floor systems is increasing, in this study the lateral in-plane behavior of composite floor diaphragms in steel structures is investigated through numerical simulations. The structures considered in the study were two models of the prototype buildings, where the elastic and inelastic response of the diaphragms under lateral load is analyzed using 3-D finite element models and FEM linear and nonlinear structural analysis. It was found that under the seismic load specified in the code, the criterion of diaphragm rigidity is too small, so the composite floor systems can be assumed as rigid body, however under lateral loads with higher amplitudes, by developing the cracks in the concrete slabs, nonlinear behavior and stiffness degradation of the diaphragms might occur. The results showed that for both single story structures the ultimate strength of the diaphragms was very high about 20 to 33 times of the seismic load specified in Iran's seismic code, however the ultimate strength of the second diaphragm was considerable showing an increment about 50–60 percent compared to the ultimate strength of the first diaphragm. The comparisons between the numerical and previously obtained experimental results showed that FEM overestimates the diaphragm response in terms of stiffness and deformability; however conservatively estimates the diaphragms strength.

Keywords: composite floor system, numerical simulation, finite element method, nonlinear analysis, diaphragm, seismic load, crack pattern, ultimate strength, in-plane stiffness

1. INTRODUCTION
The contribution of the floor systems in transferring the lateral loads (seismic actions, wind pressures, etc.) to the vertical structural elements and subsequently to the foundation of the building structures is well known and indisputable. The floor systems in building structures, are usually designed to carry the gravity loads to the vertical structural elements, however they should be also designed to resist the
lateral forces and be able to transfer them to the resisting systems by a diaphragm action. If the floor elements act together in resisting the horizontal action and have the same deflection and show high in-plane lateral stiffness, the floor performance is known as rigid diaphragm behavior. In current design practice of building structures the floors sub assemble, according to the specifications of many building codes are usually considered as a rigid diaphragm. Even this assumption is often used to reduce the degrees of freedom of the structure and simplifies seismic response analysis of many types of buildings, however for some classes of structural systems, the effect of diaphragm deformability cannot be disregarded, especially in the case of rectangular buildings with large aspect ratios where considerable inelastic floor slab behavior is expected [6]. Since the diaphragm behavior is one of the most important factors in the seismic response of the structures, researchers have conducted studies on this subject, but the studies have not a long precedent and they are mostly performed in the last two decades.

An extended numerical parametric study was carried out to study the diaphragm behavior of RC floor systems (slabs and beams). The results show that the influence of aspect ratio on the criterion of the diaphragms rigidity \( \frac{\Delta_s}{\Delta_c} \) is considerable, although there is no clear correlation between these two structural characteristics [10]. The seismic behavior of wood diaphragms in unreinforced masonry buildings has been studied through the tests on three test specimens, using different rehabilitation methods. The results indicate that FEMA 273 tended to overpredict the stiffness and significantly underpredict yield displacement and ultimate deformation levels, while FEMA 356 tended to underpredict stiffness and overpredict yield displacement [7]. The studies on the low rise steel buildings with metal roof deck have shown that the lateral period is influenced by the diaphragm in-plane flexibility and the forces in the resistant elements can be amplified due to dynamics of the flexible diaphragm, also the shaking table results have indicated that the diaphragm in-plane deformations are twice of the values obtained from static analysis [11].

The diaphragm behavior of different types of floor systems usually differs substantially and depends on the details of the floor system, so as the use of composite floors is increasing, due to their low weight and economic benefits, in this paper the behavior of composite floor systems (CFS) (steel beams with upper concrete slab) in typical steel structures under lateral load with the influence of the gravity load, and also the in-plane characteristics of the diaphragm such as the deformability, stiffness, ultimate strength, yield point and crack pattern was investigated.

2. ANALYTICAL MODELS OF THE FLOOR DIAPHRAGMS WITH LINEAR BEHAVIOR
2.1. Design and Description of Prototype Buildings
The structures considered in this study are 3-D single-story typical steel buildings consisting of composite floor and X bracings, common in many countries. The 10.8m×7.2m×3m prototype buildings considered in the study are illustrated in
Figure 1. The girders and floor joists are I shapes supported on box columns braced by X bracings having box sections. The overall geometry of the structures presented in Figure 1 is the same and the main difference is the direction of the floor joists.

Figure 1. The steel buildings prototype: (a) The floor joists parallel to the lateral load. (b) The floor joists perpendicular to the lateral load.

The composite floors were designed with the AISC code specifications and composite structures design handbook [12]. Thickness of the floor slab was obtained as 8cm and the spacing between the floor joists in the structures shown in Figure 1, were set to 108cm and 90cm respectively.

The seismic design of the structure was performed according to the seismic code of Iran [2], where the specified seismic lateral load for the structure, V, is given by:

\[ V = CW, \quad C = \frac{ABI}{R} \]  

Where \( C \) is the seismic shear force coefficient, \( A \) is zonal acceleration, \( B \) is the seismic response factor, \( I \) is the importance factor, \( R \) is the force modification factor and \( W \) is the seismic weight of the structure.

For these administrative building structures in Tehran we have:

\( A=0.35, B=2.5, R=6, I=1 \)

So we have \( C=0.146 \) and the total seismic load calculated for both of the structures, obtained from Eq.(1) is 54.1 kN.

2.2. Linear Analysis

The linear analysis of the structures was performed using SAP2000 computer program. For each structure two finite element models were developed, in the first models the floors were modeled by SHELL element having four nodes in each element to consider in-plane flexibility of the diaphragm. The beams, columns and bracings were modeled by FRAME element and the connection between these elements was modeled by the coincident nodes. The scaled structures with flexible diaphragm were analyzed under lateral load specified in the seismic code [2], with
the influence of gravity load. The FEM model and deformed shape of the structure with flexible diaphragm is presented in Figure 2(a) and (b). Due to flexibility of the diaphragm the displacement of mid point of the diaphragm is more than the side points as shown in Figure 2(b). In the second models rigid diaphragm hypothesis was used and the floors were modeled by rigid diaphragms. Figure 2(c) and (d) shows the FEM model and deformed shape of the structure with rigid diaphragm. In this model the displacements of all points of the diaphragm are the same as shown in Figure 2 (d).

Figure 2. (a) Meshing of the FE Model with Flexible Diaphragm. (b) Deformed Shape. (c) Model of Structure with Rigid Diaphragm. (d) Deformed Shape

2.3. Results of Linear Analysis
The results of analysis of the FEM models for both structures and also the results of the rotating tests are presented in Table 1. In this table RD1 and RD2 are the FE models with rigid diaphragm and FD1 and FD2 are the FE models with flexible diaphragm. Also E1 and E2 are the specimens tested under lateral and gravity loads.
The results show that both of the composite floor diaphragms were rather rigid under the lateral load specified in the seismic code. The difference between the calculated tensile and compressive bracing forces were obtained using Table 1 where for the first specimen were about 17% and 1% respectively, while in the second specimen were 3% and 10.5%.

The net displacement of the diaphragm is the relative displacement of the mid frame to the side frames, which is given by:

\[ \Delta_d = \Delta_m - \Delta_s \]  

(2)

Where \( \Delta_d \) is the diaphragm displacement, \( \Delta_m \) is the displacement of the mid frame and \( \Delta_s \) is the displacement of the side frames or the story drift. The proportion of \( \frac{\Delta_d}{\Delta_s} \) is a criterion to evaluate diaphragms rigidity in some building codes, for example with respect to the specification of Iran's seismic code[2], if \( \frac{\Delta_d}{\Delta_s} \leq 0.5 \), the diaphragm can be assumed rigid. As in these structures proportion of \( \frac{\Delta_d}{\Delta_s} \) was small (0.063 to 0.083), these composite floors under lateral load behave as rigid diaphragms. One of the effective parameters in the diaphragm behavior of floor systems is aspect ratio of the floor plan, so that for high plan aspect ratios, in-plane flexibility of the diaphragms increases significantly, but there is no clear relation between aspect ratio and \( \frac{\Delta_d}{\Delta_s} \) [3]. Therefore in these structures with low aspect ratio¹ (L/D=1.5), the behavior of floor system as a rigid diaphragm, is somehow expectable.

Table 1: Results Obtained from Three Models for the Structures

<table>
<thead>
<tr>
<th></th>
<th>Tensile</th>
<th>Compressive</th>
<th>( \frac{\Delta_d}{\Delta_s} )</th>
<th>Diaphragm net drift (mm)</th>
<th>Story drift (mm)</th>
<th>Diaphragm mid drift (mm)</th>
<th>Lateral load (kg)</th>
<th>Model</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>429.5</td>
<td>433.1</td>
<td>0.05</td>
<td>0</td>
<td>0.30498</td>
<td>0.30498</td>
<td>1380</td>
<td>RD1</td>
</tr>
<tr>
<td></td>
<td>433.3</td>
<td>426.4</td>
<td>0.07</td>
<td>0.0269</td>
<td>0.29835</td>
<td>0.31924</td>
<td>1380</td>
<td>FD1</td>
</tr>
<tr>
<td></td>
<td>519</td>
<td>429</td>
<td>0.082</td>
<td>0.119</td>
<td>0.345</td>
<td>1380</td>
<td>E1</td>
<td></td>
</tr>
<tr>
<td></td>
<td>429.5</td>
<td>433.1</td>
<td>0.068</td>
<td>0.0209</td>
<td>0.29869</td>
<td>0.31915</td>
<td>1380</td>
<td>RD2</td>
</tr>
<tr>
<td></td>
<td>442.8</td>
<td>426.8</td>
<td>0.068</td>
<td>0.0209</td>
<td>0.29869</td>
<td>0.31915</td>
<td>1380</td>
<td>FD2</td>
</tr>
<tr>
<td></td>
<td>440.8</td>
<td>483.1</td>
<td>0.063</td>
<td>0.020</td>
<td>0.32</td>
<td>0.34</td>
<td>1380</td>
<td>E2</td>
</tr>
</tbody>
</table>
3. ANALYTICAL MODELS FOR THE FLOOR DIAPHRAGMS WITH NONLINEAR BEHAVIOR

3.1. Description of Prototype Buildings

In some cases floor diaphragm may undergo lateral loads more than the seismic lateral load of a single story building specified in the building codes. For example the seismic lateral force on a floor diaphragm in lower stories of a multistory building is much more than the seismic lateral load of a single story building with a similar plan. So in the second part of the study, the nonlinear behavior of diaphragms of composite floor systems is studied. In order to ascertain nonlinear behavior of composite diaphragms and study the nonlinear characteristics of diaphragms (such as in-plane deformations, stiffness, ultimate strength, etc.) the stiffness of lateral load resisting system of the structures were increased by doubling the number of X bracings. It was to give priority to the failure of diaphragms compared to the failure of structures. Structures considered in this part of study and meshing of the FEM models are presented in Figure 3. Connections of the columns to the foundation are rigid connections, while the connection of beams and braces to the columns are hinge connections.

![Figure 3. Meshing of the FEM Models](image)

3.2. Theoretical Nonlinear Analysis

The seismic load was simulated by lateral cyclic load applied at the roof level distributed on the floor thickness and the pattern of the amplitudes of lateral cyclic load was the same as the previously conducted experiments. The nonlinear analysis of the structures was performed using ANSYS [8]. The elements used in modeling the structures are described as follows.

3.2.1. Used Elements [1]

-SOLID65:

In this study the concrete slab of composite floor system is modeled by SOLID65 element and the temperature reinforcement is considered by the volume ratio. The connectivity between the concrete and the steel beams is modeled by common joints within a distance same as the spacing of the shear keys.
-**BEAM24**: BEAM24 is a uniaxial element of arbitrary cross-section (open or single-celled closed section) with tension-compression, bending and St.Venant torsional capabilities. The element has plastic, creep, and swelling capabilities in the axial direction as well as a user-defined cross section. In this study BEAM24 element were used to mesh the steel elements of the structural steelwork, such as girders, joists of the composite floors, columns and bracings.

-**BEAM44**: BEAM44 is a uniaxial element with tension, compression, torsion and bending capabilities. This element allows a different unsymmetrical geometry at each end and permits the end nodes to be offset from the centroidal axis of the beam. Since in this element the properties of each end of beam (such as stiffness) may differ, in this study BEAM44 element were used to develop hinge connections of beams and bracings to the columns, so that all steel elements were meshed by BEAM24 element, except the end elements of the beams and braces which were meshed by BEAM44 element, then by releasing the moment of the node located at the connections, hinge connections were created. The application of BEAM24 and BEAM44 elements in developing the FEM model is presented in Figure 4.

![Figure 4. Application of the Beam Elements in Modeling the Structures](image)

### 3.2.2. Loading

#### 3.2.2.1. Gravity Load

The gravity load includes dead and live loads and a load related to scaling and simulation requirements. Because as the scale factor is 0.5 the materials used in the scaled structure must be twice of the prototype ones, so to cover the lack of weight, $Q_\rho$, a load equal to the weight of concrete slab is considered in total gravity load. The total gravity load applied on the diaphragms is given by:

$$Q_{tot} = Q_{DL} + Q_{LL} + Q_\rho = 372 \text{ kg/m}^2 = 3650 \text{ Pa}$$

The total gravity load was applied on SOLID65 element as uniform pressure of 3650 Pa with *Load key*=6.

Also the weight of structural elements was included using base acceleration of $g=9.8 \text{ m/s}^2$ upward which is equivalent to acceleration of structural elements...
3.2.2.2. Lateral Load
The lateral load was applied as uniform compressive pressure on the elements located at the edge of the floor slab. Since lateral cyclic load was applied in reverse directions, in the southern edge elements Load key=2 and in the northern edge elements Load key=4 were used. Amplitudes of lateral cyclic load in each cycle (which are the same as the previously conducted tests) [9] for both structures are shown in Figure 6. The end points of the curves are the failure points of diaphragms of the structures.

![Figure 6. Amplitudes of Lateral Load. (a) First model. (b) Second model](image)

3.3. Results of Nonlinear Analysis
3.3.1. Ultimate Strength
After loading and unloading in each cycle, the lateral loads of the next cycle were applied with larger amplitude as shown in Figure 7. The composite diaphragms concrete failed when the solutions of nonlinear analysis were not converging, because despite the time steps were too small and decreased automatically, and also the number of iterations were too large, after a large number of iterations the nonlinear analysis diverged. The criterion of concrete failure is the criterion of William and Warnke, which represents a surface of failure, using the properties of the concrete, such as uniaxial tensile and compressive stresses and the coefficients of shear transfer in open and close cracks [8]. The displacement solution of side frame (story drift) is presented in Figure 7. The end points of the graphs shown in Figure 7 relates to the divergence of solutions. According to Figure 11 the ultimate strength of the diaphragms are 27 tons and 40.8 tons, respectively. The results
show that the ultimate strength of the second diaphragm is greater than the first one with a factor of 1.511.

3.3.2. Crack Pattern
In both diaphragms some cracks developed under the gravity load which were the same in both models, however cracking under the gravity load were nominal and the main cracks developed under the lateral load. In the first model the first cracks appeared when the lateral load was about 6 tons, then by increasing the lateral load, most of the cracks developed parallel to the joists or the direction of lateral load, but under the loads about the ultimate strength (26 tons), a few cracks developed near the braced frames, which inclined about 45º to the joists. In the second model the first cracks appeared when the lateral load was about 31 tons, then by increasing the lateral load, most of the cracks developed near the braced frames, which inclined about 45º to the joists. Crack patterns of the diaphragms of two models are illustrated in Figure 12 [8].
Performance of diaphragms is generally controlled by a combination of shear and flexural actions. In this study, performance of the composite diaphragms can be perceived from the crack patterns of the diaphragms, so that if the diaphragms are considered as beams on the braced frames as their support, in the first model the crack pattern indicates that the flexural action is dominate, but in the second model the crack pattern shows that the shear action is dominant.

### 3.3.3. Diaphragms' Deformation and Stiffness

Deformed shapes of the diaphragms were extracted using a path through axis 2-2 (shown in Figure 4). For example deformed shape of the diaphragm of the first model under lateral load of 6 tons is illustrated in Figure 9. The horizontal axis is calibrated as diaphragm width, which is 5.4 m, and the vertical axis presents displacement of all points of the diaphragm. Net displacement of the diaphragm can be found from deformed shapes of diaphragm, which is difference of mid and side frames of the diaphragm.

![Figure 9. Deformed shape of the first diaphragm](image)

The analytically obtained load-displacement curves of the diaphragms are shown in Figure 10. The horizontal axis is the net displacement of the diaphragms and the vertical axis is total lateral load applied on the diaphragm [12].

![Figure 10: Load-displacement curves of the diaphragms.(a)First model.(b)Second model](image)

In order to compare in-plane flexibility of the diaphragms, the displacement of two diaphragms versus lateral load is traced in one coordinate system (Figure 11a)). As
shown in Figure 11(a) the displacement of the diaphragms under lateral loads less than 20 tons is almost the same, but under lateral loads more than 20 tons the displacement of the first diaphragm compared to the second one increases significantly. For example under ultimate load of the first diaphragm, the displacement of the first diaphragm is about 2.2 times of the second one. The comparison was made in the joint region, since the ultimate strength of the diaphragms was not the same.

One of the most important characteristics of the diaphragms which affect their behavior is their in-plane lateral stiffness. As the stiffness is the load required for unit displacement in a specific point, slope of the load-displacement curves (shown in Figure 10) represents the in-plane stiffness of the diaphragms. Variation of stiffness of two diaphragms versus lateral load is presented in Figure 11(b) and (c). As shown, in the first model the diaphragm stiffness is rather constant until lateral load is 18 tons (about 60% of the ultimate strength), then decreases about 70% until failure. However in the second model the diaphragm stiffness is constant until 34 tons (about 85% of the ultimate strength), then decreases about 50% until failure [8].

3.3.5. Stress Contours of Diaphragms
Since the structures have low plan aspect ratios, the distribution of shearing stress in the diaphragms is more important. The contours of shearing stress \( S_{xy} \) in the diaphragms are presented in Figure 12. Due to the symmetry of the models, the absolute values of shearing stress in two sides of the axis of symmetry are the
same, but have different signs. As shown in Figure 18 in both diaphragms the maximum shearing stress is observed near the braced frames. Maximum of the shearing stress for the first diaphragm is about 2.04 MPa \((0.452 \sqrt{f'_c})\), and for the second diaphragm is about 2.730 MPa \((0.618 \sqrt{f'_c})\), which shows an increase about 36% comparing to the first one.

![Figure 12. Contours of Shearing Stress in the Diaphragms. (a) First Model. (b) Second Model](image)

### 3.4. Analysis of Results

The results of nonlinear structural analysis by ANSYS are compared with the results previously obtained from quasi-static cyclic lateral loading test as follows. The ultimate strengths of the diaphragms, obtained from nonlinear FEM analysis were 27 tons and 40.8 tons respectively, while the ultimate strengths obtained from the tests were 29 tons and 47 tons, which show errors about 7% and 13% comparing to the values obtained from the tests. The difference between the results can be described as follows; ANSYS computer program can predict failure of the concrete using the criterion of William and Warnke [1], which represents a surface of failure, by means of properties of the concrete. However after failure of the concrete, some other structural elements, such as the columns, braces and the joists, and also the interlocking of the temperature reinforcement with the concrete and the joists, resist the lateral load until overall failure of the structure; so the ultimate strength of both diaphragms obtained from the tests are slightly higher than the analytical ones.

The crack pattern of both diaphragms, obtained from numerical analysis using ANSYS illustrated in Figure 8, are in good agreement with the crack pattern of concrete observed in the previous tests ([4] and [9]). As shown in the diaphragm of the first specimen most cracks are parallel to the direction of lateral load, but in the diaphragm of the second specimen most cracks inclined about 45° to the joists, showing that the test results confirm the results of FEM nonlinear analysis.

According to the results obtained in the previous experiments ([4] and [9]), it is obvious that for both diaphragms the finite element method generally underpredicts
the diaphragms displacements under lateral load and overpredicts the diaphragms stiffness compared to the ones obtained from tests. The difference between the results of two methods can be described as follows; in FEM analysis of diaphragms, size of the elements affect the displacement values of the response, and if the discretisation mesh is not fine enough the stresses of the lateral resisting elements may be determined with a good approximation, but the response displacements may be determined with some errors [3]. In modeling of the structures, with respect to the hardware abilities, the diaphragms were meshed by 11.25 cm x 13.5 cm elements as shown in Figure 3, so by using finer elements in meshing of the diaphragms, the analytical displacement responses of the diaphragms may be closer to the experimental ones. Also concrete is not a homogeneous material and has rather complicated behavior, so modeling the concrete by simplified material models may result in inaccurate results in the nonlinear analysis of concrete elements.

The results analysis show that if two individual diaphragms are designed under gravity load with the same conditions, the diaphragm with joists perpendicular to the lateral load, exhibits a better performance under lateral loads, so it is recommended that in a building with low plan aspect ratio, composite floor systems are so constructed that the direction of the joists in the vicinity of braced frames or shear walls, is perpendicular to the direction that the main lateral load resisting elements act, or the joists are set in a staggered manner all over the plan.

If a building has a high plan aspect ratio, directing the joists in the long direction would lead to better performance of the composite diaphragm, however in some cases directing the joists to be perpendicular to the lateral load would be with some penalties, because if the joists are in the long direction of the diaphragm, they are less efficient under gravity load and it would be more costly.

4. CONCLUSIONS

In this paper the behavior of composite diaphragms was studied in two parts, in the first part the diaphragms were subjected to the lateral seismic load specified in the seismic code and distribution of the lateral load among the resistant elements was studied using FEM analysis with rigid diaphragm and flexible diaphragm hypothesizes and verified with the results of the tests on the half scale specimens. The results show that under the seismic load specified in Iran's seismic code [2], the criterion of diaphragm rigidity ($\Delta_s/\Delta_y$) is too low and the diaphragms can be assumed rigid. Also the forces of bracings calculated from the methods have errors less than 17%, which indicates that using the rigid floor diaphragm model provides adequate results for the stresses of the laterally resisting vertical structural elements and the story drift. The models considered in the first part after increasing the stiffness of the side braced frames, were subjected to the quasi-static reverse cyclic lateral load up to failure. The results show that the second diaphragm (where the joists direction was perpendicular to the lateral load direction) has higher lateral in-plane stiffness and there were no significant stiffness degradation until 85% of the ultimate load, but the first diaphragm (where the joists direction was parallel to the
lateral load direction) had lower stiffness and the stiffness degradation started at a load of 48% of the ultimate load. The ultimate strength of the second diaphragm was considerably showing an increment about 50–60 percent compared to the ultimate strength of the first diaphragm, also for both of the single story structures the ultimate strength of the diaphragms was very high and was about 20 and 33 times of the seismic load specified in Iran's seismic code, respectively. The comparisons between the numerical and experimental results previously obtained by the authors showed that FEM overestimates the diaphragm response in terms of stiffness and deformability; however FEM conservatively estimates the diaphragms strength. Generally it seems that one of the most important parameters in the diaphragm behavior of the composite floor systems is the direction of the joists relative to the lateral load and it is recommended that the composite floor systems are so constructed that the joists direction is perpendicular to the direction toward which the main lateral load resisting elements act.

5. ACKNOWLEDGMENTS
The writers acknowledge the financial support provided by building and housing research center of Iran (BHRC) who founded this research. The supports provided by Center of Excellence of Infrastructural Engineering at the University of Tehran where this research was conducted, is also appreciated. The authors wish to express their appreciation to eng.K.Khalili Jahromi and also the technical staff of the structural engineering laboratory of BHRC, for their most useful collaboration in this project.

REFERENCES


INFLUENCE OF CENTRAL VERTICAL BAR ON THE BEHAVIOUR OF HIGH STRENGTH CONCRETE TRANSFER BEAM-COLUMN JOINTS

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ABSTRACT
Beam column joint (BCJ) specimens tested under monotonic loading were compared with joint shear failure predicted according to EC8-NA [1] and ACI352 [2].
Inadequacy of these design codes for accurate estimations of the shear stresses at BCJ are identified. A design rule for the prediction of BCJ failure for high strength concrete (HSC) is given. The proposed method offer better accuracy when the results are compared with design rules from the above codes and research results.
Finite element numerical models for BCJ specimens were compared with the experimental ones. Furthermore parametric investigations of the influence of Central Vertical Bar, CVB, on the shear capacity of HSC- BCJ were conducted.
Strut and tie model for BCJ with CVB was developed to guide the designers towards using the proposed design rule to calculate the amount of shear CVB and stirrups required in order to resist the excessive joint shear.

Keywords: high strength concrete, transfer beam column joint, central vertical bar

1. INTRODUCTION
The shear design of beam-column joint (BCJ) is normally assessed in seismic countries where ACI [2], AIJ [3&4], IKU [5], AETL [6] and EEFIT [7] reports following earthquakes have identified BCJ as critical part of the reinforced concrete frame structure. The joint shear design for BCJ has been the subject of numerous research projects in the past three decades. This paper investigates the shear behaviour of external beam column joints of HSC column and transfer beam (see Figure 1) exposed to monotonic loading.
Many tall reinforced concrete frames are built with transfer beams to provide clear spaces in their entrance halls. With the advantages of HSC, such buildings usually have HSC columns. The external BCJ (see Figure 1) made of transfer beam and HSC column has unique shear behaviour, which has not been investigated fully by other researchers.
The authors' investigations on 12 beams [8,9], Figure 2, indicated that:
- When the shear span to depth ratio a/d = 3 then HSC beams shear resistance may be less than that of NSC beams (Figure 3a).
- When adding CHB in the beams then shear resistance of HSC beams
significantly improves and become greater than that of NSC.

Figure 2. A multi storey RC frame with transfer beams of $3 \geq \frac{h_b}{h_c} \geq 2.5$

The reasons for such behaviour are due to (i) the stabilising arching affect in the beam as the result on the presence of CHB (Figure 3b) (ii) the double-strut action produced by the presence of central bar in addition to the main reinforcement (Figure 3c).

The ratio of beam depth to column depth is defined as the aspect ratio ($h_b/h_c$) has significant influence of BCJ behaviour (Figure 3a). Taylor’s [10] demonstrated that shear behaviour of short beam is analogous to the behaviour of BCJ when aspect ratio $\leq 2$. Similarly Motamed [8&9] has shown that the shear resistance in HSC beams with CHB produces stabilising arching affect, due to the dowel action as well as double strut action, is comparable to short beams shear behaviour, hence HSC beam with CHB will behave similar to BCJ with central vertical bar (CVB). Therefore, since the short beam behaviour is analogues to the behaviour of BCJ thus, it can be assumed that $h_b/h_c \approx a/b$ (Figure 3a). Similarly, BCJ shear resistance with vertical central bars in the column with aspect ratio $3 \geq \frac{h_b}{h_c} \geq 2$ is analogous to HSC beam, $3 \geq a/d \geq 2$ with CHB (Figure 3b).
Following review of the EC8-NA [1] and ACI352 [2] design methods and the past experimental research on BCJ, an empirical design equation for the joint shear is introduced which is proportional to the joint concrete strength, the shear resisting contribution of the dowel action from the vertical central bars and the confinement stirrups.

The proposed design rule for joint shear allows for prediction of quantity of the vertical central bar in the column as shear reinforcement in high strength concrete BCJ with large aspect ratio.

2. CALCULATION OF JOINT SHEAR FORCE

A brief review is that $V_{u,joint} = T_n - V_{col}$ (Figure 4), where $V_{u,joint}$ is the joint shear, $V_{col}$ is the horizontal shear force across the column and $T_n$ is tension force in the tension reinforcement of the beam which is given by $T_n = M_n/z$, where $M_n$ is the beam moment at the column face and $z$ is the flexural lever arm. The theoretical joint shear force is dependent on the assumptions used to calculate $M_n$ and $z$. $M_n$ is taken as $M_n = P (L + d')$ as shown in Figure 4, where $L$ is the distance from the load $P$ to the face of the column and $d'$ is the distance from the face of the column to the centroid of the of column reinforcement as shown in the Figure.
The tensile force in the beam reinforcement is calculated by section analysis assuming that plane section remains plane. The rectangular-parabolic stress block defined in EC2 [11] is used for the concrete. The stress is assumed to reach a maximum value of $0.8f_{cu}$ at a compressive strain of 0.002. The width of the compressive stress block is taken as the beam width in the analysis of the beam-column joints. An elasto-plastic stress-strain response is assumed for the reinforcement with an elastic modulus of 200 GPa. No material factors of safety are applied.

2. ANALYSIS OF THE AVAILABLE TEST DATA

There is a general lack of agreement among researchers over the influence of
variables such as concrete strength, column loading, joint aspect ratio, joint stirrups, beam thrust, beam reinforcement and column vertical bars on the joint shear behaviour of the external BCJ. Furthermore numerous tests on BCJ under cyclic loading simulating earthquakes behaviour have been performed; however, these researches have a number of shortcomings such as lack of detail investigation of the influence of shear stress at BCJ due to incremental strain development in the reinforcement.

In order to investigate these factors, available data from tested BCJ specimens were statically used to develop the parametrical values for the proposed equation of concrete contribution to joint shear at BCJ.

A finite element model, Figure 8, has been developed for typical specimens and loaded with incremental monotonic loading condition. FE model was used to study the above influences as well as to compared its results with those of the test results and with the predications of the proposed design equation.

Research on monotonically-loaded, external beam-column joints were carried out in the UK, by Ortiz [12], Taylor [14], Scott [15], Scott Hamill [16], Parker & Bullman [17], Wilson [20] and Vollum [19]; similarly in Germany by Kordina [13]. Test data from these experiments are shown in Table 1.

The relationship between shear index and stirrups index, Figure 6, show that there is a linear increase of shear in the joint as the amount of stirrups increases. However this occurs after all the concrete contribution to resist the joint shear has been taken into account. Neither of the above two equations make provision for this behaviour even though both design methods specify minimum stirrup requirements. Furthermore neither of the two equations predicts the degree of dependency of joint shear strength on joint aspect ratio, Figure 7.

A shear analysis is carried out to develop a relationship between concrete strength and the joint shear strength for the specimens shown in table 2. These analyses show that the joint shear strength has a closer relationship to \( (f'_c)^{2/3} \) of EC8-NA rather than \( (f'_c)^{1/2} \) of ACI/ ASCE Committee 352. This is because the variance of shear index of specimens without stirrups is 0.29 (0.54-0.25=0.29) for EC8 and 0.43 (0.94-0.51=0.43) for ACI, Figure 6.

![Figure 6. Relationships between Joint Shear index and Stirrup index according to EC8-NA and ACI 352](image-url)
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Table 2: Table for Shear indices: Vj/bchcfc2/3 (average value=0.525) and Vj/bchc√fc and
Stirrup indices: Asjefy/bchcfc2/3 and Asjefy/bchc√fc. Shaded and bold specimens are in HSC
Researcher

Ortiz

Kordia

Taylor

Scott

Scott&
Hamill

Parker &
Bullman

Sarsam
Vollum
Wilson

Identity

Bar
Detail

Fc
MPa

(MPa)

P
(kN)

BCJ1
BCJ2
BCJ3
BCJ4
BCJ5
BCJ6
BCJ7
RE2
RE3
RE4
RE6
RE7
RE8
RE9
RE10
P1/41/24
P2/41/24
P2/41/24
A3/41/2
D3/41/2
B3/41/2
C3/41/2
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C4
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C4AL
C7
C3L
C6
C6L
C9
C4ALN
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C4ALH
C6LN0
C6LN1
C6LN3
C6LN5
C6LH0
C6LH1
C6LH3
4b
4c
4d
4e
4f
5b
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EX2
EBCJ6
EBCJ8
J1

L Bar
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120
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fyb

Asjefy/
bchc√fc

Vj/bchc√fc
√MPa

Asjefy/
bchcfc2/3

Vj/bchcfc2/3

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0.54
0.54

√MPa

MPa2/3

MPa2/3


The dotted lines in the graphs represent the empirical values 0.525 and 1.058 of equations (1) and (2) respectively. The results of the specimens below the dotted lines indicate over estimation of the joint shear. Ignoring the minimum reinforcement requirements; the numbers of joint failures which are within the safe prediction of EC8-NA [1] are 28 out of 56 tests i.e. 50% of total specimens. Whereas for ACI 352, the numbers of safe prediction of joint failures are 23 out of 56 test i.e. 41%. (These are shown above the horizontal dotted line, Figure 6).

A linear relationship between shear index and stirrup index can be plotted when the shear indices are above 0.35 and 0.7 for EC8-NA and ACI 352 respectively. From the graph, Figure 6, it can be noted that the upper limit of stirrup index for EC8 ≤ 0.4 and for ACI ≤ 0.75.

From table 2 it can be concluded that for EC8-NA [1], the mean values for shear index in BCJ for L-reinforcement, Figure 5, is 0.54 and for U-reinforcement, Figure 4, is 0.49.

3. PROPOSED DESIGN EQUATION FOR EXTERNAL BEAM COLUMN

Both design codes ACI 352[2] and EC8-NA [1] specify minimum shear stirrup requirements, however, they do not give provision for the joint strength to be increased by the stirrups.

The design recommendations of these codes fail to predict the observed dependence of joint shear strength on the joint aspect ratio, as well as the influence of HSC and detailing of the anchorage on the behaviour of BCJ. Also they do not provide any recommendation if the amount of stirrups is not adequate in order to provide sufficient shear strength at BCJ when the shear forces are high.

As noted above the HSC beams may be weaker in shear than NSC beams when span depth ratio is 3, it can also be deduced that HSC-BCJ will be weaker than NSC-BCJ when the joint aspect ratio exceed 2.5.

Past research work by Motamed [7] on 12 beams demonstrated that for the design of HSC beams with a/d=3, CHB produced superior shear capacity due to the development of dowel action which in turn enhanced the stabilising arching affect in the beams.

Using Baumann's [21] dowel cracking expression, the dowel force causing cracking is:

\[ V_{du} = D_{cr} = 1.64 \, h_c \, d_b \, f_{cu}^{1/3} (n)^{1/4} \] (for n number of bar in the beam) \hspace{1cm} (3)

\[ V_{du} = D_{cr} = 1.95 \, h_c \, d_b \, f_{cu}^{1/3} \] (for n = 2 i.e. bar at mid-depth, BCJ with CVB) \hspace{1cm} (4)

Where \( d_b \) = diameter of the dowel bars and \( n \) is number of bars,

\( V_{du} = \) dowel force, \( f_{cu} = \) cube crushing strength of concrete of 150 mm cubes in N/mm\(^2\).

The stabilising arching effect in the beam with a/d = 3 makes the beam perform like a short beam 2≤a/d≤3 and is analogous to BCJ shear (Figure 3a ).

BCJ with central vertical bar in column, the dowel shear resistance is
\[ V_{jd} = V_c + 1.95 h_c d_b f_{cu}^{1/3} \]  

(5)

\[ V_c = \gamma (f_c)^{2/3} b_c h_c \]  

(6)

\[ V_{jd} = \gamma (f_c)^{2/3} b_c h_c + 1.95 h_c d_b f_{cu}^{1/3} \]  

(7)

Where \( \gamma \) is 0.54 or 0.49 for L-type, or U-type detail connections shown in Figures 4 and 5.

Proposed design rule is based on refining EC8-NA [1] design rule by using \( \gamma \) factor for beam detailing and including the dowel action from the central bar within the depth of the column.

The proposed method for designing shear stirrups in BCJ adopted from Fip Recommendation [22] for short beams is:

\[
F_{nw} = \left( \frac{2 \times \frac{5}{8} \times h_b / h_c - 1}{3 - N_a / F_n} \right) K \cdot F_n
\]  

(8)

where \( K = 2/3 \) for all perimeter BCJ or \( K = 1 \) for corner BCJ, described in para 2 of page 5. \( F_{nw} \) is the yield force in the beam reinforcement or \( F_{nw} = \sum A_{st} f_y \), \( A_{st} \) is total area and \( f_y \) is the yield stress of stirrups, \( F_n \) is the shear force \( V_{u, joint} \) at BCJ (Figure 4), \( N_a \) is the axial force acting on the column, if any. The value 5/8 is portion of depth of beam where the stirrups are effective.

As the angles \( \Theta_1 \) and \( \Theta_2 \) between the struts and ties decrease, Figure 7, the aspect ratio increases, it is therefore desirable to introduce vertical central bar when \( f_{cu} \geq 60 \text{MPa} \) and \( \Theta_1 \leq \tan^{-1}0.5 \).

Looking at equation (8), when \( h_b / h_c \leq 1.25 \) no joint stirrups would be required, this is checked with Wilson's experimental results which has \( h_b / h_c = 1 \), table 2. Without stirrups the shear index is 0.54, which is the same as the predicted Figure to design proposal rule of equation (7) when no central vertical reinforcement, dowel bars, are used because \( \Theta_1 \leq \tan^{-1}0.5 \).

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**Figure 7. Strut and tie model for BCJ with central vertical reinforcement**
The reason \((f_c)^{2/3} b_e h_c\) was taken for concrete contribution in the proposed rule is the result of the comparison with \((f_c)^{1/2} b_e h_c\). The accuracy of EC8-NA for predictions compared to experiments was 50% as compared to ACI352 [2] which was 41%, Figure 6.

VC joint shear from the concrete compression strut action is \(V_c = 0.54 f_c^{2/3} b_e h_c\) for L detailing shown in Figure 5, and \(V_c = 0.49 f c^{2/3} b_e h_c\) for U detailing shown in Figure 4.

4. CONCLUSION

1. A design method has been developed, based on statistical data of published 56 test results of BCJ, to calculate the shear resistance in HSC and NSC beam-column joint.
2. The proposed equation is a function of aspect ratio and the magnitude of shear force in BCJ and lower-bound theorem of plasticity maintained.
3. The results given by the proposed design equation are 79% of the total actual experimental data while the results produced from EC8 provided only 21% of the actual experimental results (assuming the experimental results is equal 1).
REFERENCES
THE EFFECT OF STEEL MESH RATIO AND AXIAL LOAD ON THE BEHAVIOR OF STRENGTHENED BRICK WALLS WITH RC OVERLAY

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3Phd student, Dept of civil Engg. Tarbiat Modares University, Tehran, Iran

ABSTRACT
Concrete is one of the most important materials used in many types of construction. This material is widely used in seismic rehabilitation of buildings, particularly in strengthening of masonry buildings. The latter covers a wide range of historical to conventional brick buildings which are the most vulnerable in the earthquake prone areas. One of the most available, economical and simple conventional techniques used in strengthening of brick buildings is 3 to 5 mm thick concrete overlay (shotcrete) with steel mesh on the brick wall surfaces. This method of strengthening improves the seismic behavior and lateral and in-plane strength of brick walls which depends on the thickness, strength of concrete and the amount of reinforcement. In this paper, the effect of shotcrete on strength and stiffness of brick walls has been investigated utilizing micro-modeling through ABAQUS software based on discrete elements method. Results of numerical and an experimental analysis of the in-plane shear behavior of strengthened brick walls are compared and discussed. The main variables considered in this investigation are the compressive axial load applied on the wall as well as the reinforcement ratio.

Keywords: RC overlay, strengthening, brick walls, discrete element

1. INTRODUCTION
Shotcrete has grown into an important and widely used construction technique, especially in strengthening of masonry structures. The masonry brick wall is a composite material which has no similar directional properties due to its mortar joints as a plane of weakness. In many researches carried out, two different approaches based on macro and micro models have been used. Its numerical representation shall be based on micro-modeling for its individual components (brick and mortar) or macro-modeling for a masonry element (composite unit). In such a case if the material is regarded as an anisotropic homogeneous continuum, then the interaction between the components can be ignored in modeling and analysis. Depending on the desired level of accuracy and the simplicity, the detailed micro-modeling or simplified micro-modeling may be used. In detailed micro-modeling the units and mortar in the joints are represented by continuum elements whereas the unit-mortar interface is represented by
discontinuous elements. In this approach the material properties like modulus of elasticity, Poisson’s ratio, inelastic properties of the bricks and mortar are taken into account. In micro-model approach it is possible to characterize separately mortar, blocks and their interfaces, adopting suitable constitutive laws for each component, which take into account their different mechanical behavior. The micro model is probably the best tool available to analyze and understand the real behavior of masonry, particularly concerning its local response but requires an intensive computational effort. The macro-models constitute an effective method to analyse the global response of masonry structures. In such an approach, masonry is regarded as an equivalent material, where mortar and blocks are jointed together, and appropriate relations are established between averaged masonry strains and averaged masonry stresses. Lourenço (1996) has proposed a non-linear constitutive model for in-plane loaded walls based on the plasticity theory. Therefore macro-modeling can be used to reduce time consuming and establishing a relation between average masonry strains and average masonry stresses. Finally either use of micro-modeling or macro-modeling of masonry brick buildings or components requires a description of the material obtained experimentally. In the present study, the micro-model is used to obtain the actual behavior of walls by assuming that brick, mortars and their interface as three separate elements.

2. EXPERIMENTAL WORKS

A series of experiments have been studied by second and third authors on six walls in two groups in order to study the effects of reinforced concrete overlay on the behavior with the following specifications:

Group-1: Samples tested in this group, used for studying the rocking mode of failure, had 1800 mm length, 1200 mm height and 200mm thick, under 39.22 KN gravity force. Lateral load was applied in cyclic manner on the wall. The walls are named as bellow:
- No Shotcrete Brick Wall 1- "NSW1"
- Single side Shotcrete Brick Wall 1- "SSW1"
- Double side Shotcrete Brick Wall 1- "DSW1"

Group-2: Samples tested in this group, used for studying the shear sliding mode of failure, had 1800 mm length, 800 mm height and 200mm thick, under 39.22 KN gravity force. Lateral load was also applied in cyclic manner on the wall. The walls are named as bellow:
- No Shotcrete Brick Wall 2- "NSW2"
- Single side Shotcrete Brick Wall 2- "SSW2"
- Double side Shotcrete Brick Wall 2- "DSW2"

With these descriptions and considering the experimental results, numerical modeling was carried out utilizing the software ABAQUS.
3. NUMERICAL MODELING OF MASONRY SAMPLES
Defining a reliable interface element is a primary step for the simplified micro-modelling approach. For the interface element two kinds of stiffness as normal stiffness and tangent stiffness are considered, which are given respectively by equation 1.

\[ k_n = \frac{E_b E_{m}}{E_b + E_{m}} \]
\[ k_t = \frac{G_b G_{m}}{G_b + G_{m}} \]  

(1)

Where \( t_{jn} \) is the actual thickness of the joint, \( E_b \) and \( E_{m} \) are the modulus of elasticity, \( G_b \) and \( G_{m} \) are the shear modulus, for brick and mortar respectively. It is assumed that the mortar shear behavior in two horizontal directions is approximately the same. These directions are defined as \( n \) and \( s \) directions. Shear strength is depended on cohesive parameters, internal frictional angle and applied normal stress. For high vertical stress, shear failure occurs through brick crushing. The elastic domain is bounded by a composite yield surface that includes tension, shear and compression failure. Nonlinear behavior of the masonry units in compression involves parabolic hardening and then parabolic-exponential softening in both directions with different fracture energies. Compressive strength is obtained from brick and mortar prism experiment. It has been recommended to use concrete propositional values for fracture energy in compression [1]. The micro-modelling utilized in this research considers the brick, mortar and interface elements and analysis was carried out through ABAQUS computer program. For modeling of masonry units, nonlinear tension behavior of them, involves an exponential softening curve distinctively for both directions, with mode of fracture energies \( G_{fnw} \) and \( G_{fxw} \). Based on descriptions in reference [3], an average value of the bond mode I fracture energy equal to 0.012 Nmm/mm² was adopted. In simplified micro-modelling, it is important how to define the interface element. According to the reference [2] the approximate amount of mode II fracture energy

(a) None strengthened  (b) Single side strengthened  (c) Double side strengthened

Figure 1. Samples of Tested brick walls
is suggested equal to 0.1c. Cohesive type elements, used for interface elements, and they are composed of two faces separated by a thickness. The relative motion of the bottom and top faces measured along the thickness direction represents opening or closing of the interface. Stretching and shearing of the mid-surface of the element are associated with membrane strains in the cohesive element [4]. The available traction-separation model in ABAQUS assumes initially linear elastic behavior followed by the initiation and evolution of damage. When surfaces are in contact they usually transmit shear as well as normal forces across their interface. There is generally a relationship between these two force components. The relationship, known as the friction between the contacting bodies, is usually expressed in terms of the stresses at the interface of the bodies. The friction model applied here is the classical isotropic Coulomb friction model. The model defines the critical shear stress, \( \tau_{\text{crit}} \), at which sliding of the surfaces starts as a fraction of the contact pressure, \( p \), between the surfaces (\( \tau_{\text{crit}} = \mu p \)). \( \mu \) is known as the coefficient of friction. In the default model the coefficient of friction is defined as a function of the equivalent slip rate and contact pressure [4].

4. NUMERICAL MODELING OF TESTED BRICK WALLS
A non-strengthened (NSW1), a single side strengthened (SSW1), and double side strengthened (DSW1) brick wall samples were selected from a group of experimentally tested specimens [5]. All samples have 1800 mm length, 1200 mm height and a thickness of 200mm. Mortar average thickness was 1 cm. These walls were subjected to a 4 ton distributed gravity load and a cyclic lateral load at their upper level.

a) Wall NSW1
This wall is a non-strengthened brick wall. For the numerical analysis of this wall the fracture energies for mode I, II and III were selected from reference [4]. The upper and lower beams are assumed to be rigid in order to minimize the time of analysis. The gravity load is applied in a linear increscent procedure within first 5 seconds and remains constant to 20\textsuperscript{th} second. Lateral displacement of the wall increases linearly from 5\textsuperscript{th} second to 20\textsuperscript{th} second. The wall analysis results for lateral displacement and Von-Mises average stresses are given in Figure1. Good agreement is obtained in comparison with numerical results. The predominant failure mode for this wall was rocking.

b) Wall SSW1
Standard tests on concrete overlay and rebar steel mesh and material properties are necessary for numerical modeling of either single or double side strengthened brick wall. Compression and tension behavior and damage functions curves (both in compression and tension) of concrete layer are defined using proper behavior models. Brick and interface properties are described as above and rebar mesh specifications are given in Table 2. The reinforced concrete elements in ABAQUS program was carried out using 3D concrete element and embedding rebar mesh surface element. The location of rebar element is assumed to be placed at mid
surface of concrete element and tied to concrete element via a contact element. Also complete compatibility is assumed for reinforced concrete overlay and brick wall connection. The analysis of the model is carried out for 4 ton gravity load and the relevant results are compared with that of experimental work. Figure 3 show deformations for two loading states A and B for wall SSW1 corresponding to its concrete and brick faces. For both analytical and experimental results the failure rocking mode is observed. It is evident that the specimen SSW1 was twisted due to unbalanced distribution of stiffness in its section. In order to study the accuracy of the numerical modeling, experimental and numerical shear-displacement curves for this wall is compared and illustrated in Figure 3-e, which indicate good correlation.

<table>
<thead>
<tr>
<th>Brick</th>
<th>Cohesive Element</th>
<th>Contact Element</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density (kg/m³)</td>
<td>2.10x10⁻⁹</td>
<td>2.36x10⁻⁹</td>
</tr>
<tr>
<td>Elastic</td>
<td>$E=1020 \text{ N/mm}^2$</td>
<td>$\nu=0.15$</td>
</tr>
<tr>
<td>Damage</td>
<td>$K_{22}=0.1$, $K_{33}=36$, $K_{33}=36$, $\lambda=0.027$, $\lambda=0.140$, $\lambda=0.140$</td>
<td></td>
</tr>
<tr>
<td>Tangential Contact</td>
<td>Normal Contact</td>
<td>Hard Contact</td>
</tr>
</tbody>
</table>

Table 1: Engineering Properties of Non-strengthened Brick Wall NSW1

Figure 2. Comparison of Experimental and Analytical Results for Wall NSW1
Table 2: Engineering properties of reinforced concrete overlay

<table>
<thead>
<tr>
<th></th>
<th>Density (ton/mm³)</th>
<th>Elastic Properties</th>
<th>Damage Plasticity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete overlay</td>
<td>ρ=2.4x10⁻⁹</td>
<td>E=2600 N/mm²</td>
<td>Dilation Angle=25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>v=0.20</td>
<td>Eccentricity=0.1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>f_b0/f_b=1.16</td>
<td>K=0.67</td>
</tr>
<tr>
<td>Rebar properties</td>
<td>ρ=7.85x10⁻⁹</td>
<td>E=210000 N/mm²</td>
<td>Viscosity Parameter=0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>v=0.20</td>
<td>Yield Stress=640</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Plastic Strain=0</td>
</tr>
</tbody>
</table>

Figure 3. Comparison of Experimental and Analytical Results for Wall SSW1

c) Wall DSW1
The modeling of double side strengthened wall (DSW1) in ABAQUS is almost similar to that of SSW1. The material properties are also the same. Numerical modeling is represented in Figure 4-a. In order to study the accuracy of the numerical modeling, experimental and numerical shear-displacement curves are prepared and compared in Figure 4-b. From this figure it is evident that stiffness of both result are the same in elastic state and good agreement is obtained for plastic state. Due to numerical instability the maximum deformation of 6mm was imposed during the numerical analysis.
5. PARAMETRIC STUDYING
The verification and calibration of the modeling of strengthened wall specimens, yields the non-linear numerical analysis as a tool to obtain the effect of some design parameters, such as amount of shear reinforcement and magnitude of axial compression load on the capacity of such specimens. So in this section these effects are discussed and concluded.

6. EFFECT OF REINFORCEMENT
In this study the amount of reinforcement throughout the wall surface is considered as a variable. For this purpose two cases were considered. In the first case, for single fixed spacing the wall is modeled with different rebar size. In the second case, for a specific rebar size, the effect of various spacing is studied. It is found that an increase in the quantity of reinforcement (through amount of rebar or change of spacing) has a direct influence on load carrying capacity. This is clearly illustrated in the numerical modeling results shown in Figures 5. From this figure we can conclude that the maximum shear strength increases with the increase of reinforcement rebar or reducing the spacing. The interesting point is that the initial stiffness does not much vary for both cases.

7. EFFECT OF AXIAL COMPRESSION STRESSES
The influence of axial compression stress on masonry shear strength is illustrated in Figure 6. These figures show the Load-Displacement envelops of the two masonry walls (SSW1 and DSW1) that had the same dimensions and reinforcement details, but were subjected to varying levels of axial compression stress. This figure shows that for higher rebar diameter the bearing capacity of the wall increases and for lower diameter, as the higher load applied the lower capacity results.
Figure 5. Comparison of Experimental and Analytical Results for Wall DSW1

a) Effect of shear reinforcement on the strength of masonry wall (SSW1)

b) Effect of shear reinforcement on the strength of masonry wall (DSW1)

c) Spacing effect of shear reinforcement on the strength of masonry wall (SSW1)

d) Effect of shear reinforcement with various spacing on masonry shear strength of DSW1
8. CONCLUSIONS
According to the obtained results from the calibrated non-linear numerical analysis it is concluded that shear capacity of the strengthened brick walls is due to the contribution of brick wall, reinforced concrete overlay and steel rebar. Also initial diagonal cracks did not widen significantly under increasing lateral displacements, but instead new sets of diagonal cracks formed and gradually spread over the wall diagonals, accompanied by higher energy dissipation and more ductile behavior. The applied axial compression load has a significant influence on the in-plane shear performance of masonry shear walls, mainly because it suppressed the tensile field in a material inherently weak in tension.

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COMPARISON OF DIFFERENT CURING EFFECTS ON
CONCRETE STRENGTH

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ABSTRACT
The purpose of this investigation was to conduct a laboratory test program on how
much different curing conditions affect the attainable strength of concrete. To
achieve this purpose, a laboratory test program was conducted. The laboratory
program consisted of casting 150 mm by 150 mm concrete cubes using eight
different mix designs and subjecting them to six different curing conditions. In
order to investigate the influence of curing conditions, on the compressive strength
of concrete cubes, for each mix design three cubes were chosen for every curing
regime. The curing regimes employed were: immersion in drinking water; covering
with wet hessian and polythene sheet; keeping under dry laboratory conditions;
keeping in open air; curing compound and steam curing. Except for steam curing
system, the specimens of which were tested at the age of three days, for all other
curing conditions, the compression tests were performed at the age of 28 days. It
has been found that the curing system greatly influences the concrete strength.
While the highest gain in compressive strength was recorded for cubes covered
with wet Hessian and polythene sheet, the lowest gain in compressive strength was
recorded for the specimens cure using steam curing.

Keywords: concrete, compressive strength, curing systems

1. INTRODUCTION
Concrete is a mix of cementitious (binding) solids [e.g., cement (calcium silicates,
calcium aluminates, and calcium alumino-ferrites) and sometimes fly ash (aluminates
and silica) and micro-silica], aggregate (sand and stones), and water. The
cementitious solids of concrete, upon mixing with water, react in highly exothermic,
temperature-dependent hydration reactions (the higher the temperature, the faster the
hydration reactions) producing a firm, hard mass. There are four major stages in the
hydration reactions: 1) surface reactions produce a “gel” on cementitious particles
and release heat, lasting about 30 min, 2) hydration is slowed for several hours
because diffusion of water into the cement particle is inhibited by the gel, 3) vigorous
hydration and heat development occur for up to 20 h as water reaches un-hydrated
cement inside the gel coating (stiffening of the concrete occurs during this stage), and
4) hydration continues to decline for years [1-3].
To ensure that hydration continues, especially at the surface, the concrete must be cured. Curing means water at the surface of the concrete is retained to allow the concrete to hydrate to a point where it has a strong, durable structure. If curing is inadequate, the water evaporates and hydration stops, resulting in a low-strength concrete. If adequate moisture isn’t maintained in the curing environment, the concrete won’t develop maximum compressive strength, and cracking may occur. Durability of the concrete may also be reduced due to inadequate hydration of the cementitious material.

Ambient atmospheric conditions can adversely influence the thermal and moisture structure of freshly poured concrete. If concrete becomes too warm or temperature gradients too large during the first several days after the concrete is poured or if there is insufficient water in the concrete, the concrete may crack or may not develop its maximum potential strength, reducing its long-term durability [4-7]. Surface drying may even affect the underlying concrete, as water will be drawn from the lower levels into the dry surface concrete. Any significant internal drying also will slow or stop hydration and the structure may not gain adequate strength.

For hydration to continue, the relative humidity inside the concrete has to be maintained at a minimum of 80%. If the relative humidity of the ambient air is that high, there will be little movement of water between the concrete and the ambient air and no active curing is needed to ensure continuation of hydration. Prevention of the loss of water from the concrete is of importance not only because the loss adversely affects the development of strength, but also because it leads to plastic shrinkage, increased permeability and reduced resistance to abrasion.

Continuous curing for a specified time, starting as soon as the surface of the concrete is no longer liable to damage is desirable. Such conditions can be achieved by continuous spraying or pounding or by covering the concrete with wet burlap. Probably the best method for curing concrete, although sometimes the least practical, is to flood the surface continuously with water for the first week after placement. But, if concrete dries between soakings, this alternate wetting and drying may actually damage the concrete. When water curing, the sprinkler should be going continuously for at least one week. On inclined or vertical surfaces, soaking hoses can be used. If w/c is low, continuous wet curing is highly desirable.

Another method of curing is called water barrier method. The techniques used include covering the surface of the concrete with overlapping polyethylene sheeting. White sheeting is preferable because it has the advantage of reflecting solar radiation in hot weather [8].

Method of spraying curing compounds, which form a membrane may be used as well. It is obvious that the membrane must be continuous and undamaged. The timing of curing is also critical. The curing spray should be applied after bleeding has stopped. The most common way to cure new concrete is through a liquid membrane-forming curing compound also known as "cure and seal". These materials are usually sprayed or rolled on the surface. When dry, they form a thin film, which restricts moisture evaporation from the surface.

Timing is most important when using a curing compound. These products must be applied as soon as final finishing is complete. Otherwise, they could mar the
concrete's surface. Also, the ready mix concrete supplier should be checked for recommendations on what to do when cold/freezing temperatures are anticipated. The next most important thing is the application rate. In this regard the manufacturer's recommendations should be followed completely. The optimum time is the instant when the free water on the surface of the concrete has disappeared so that water shine is no longer visible [9].

Most penetrating sealers are made from derivatives of silicone called silanes or siloxanes designed to penetrate concrete pores. Once there, they react with the alkaline materials and moisture present to form silicone, making concrete water-repellent. While penetrating sealers usually cost more, they should last longer. Another reason for penetrating sealers popularity is that, when properly applied, they don't change the concrete's appearance. The major concern is that there can be no other membrane cure or sealer on the concrete when applying and the concrete must be at least 28 days old.

Internal concrete temperature is the most important factor affecting early compressive strength of concrete. Because of this, external heat is usually applied to produce high early compressive strengths concrete products after 12 to 18 hours of curing. Temperature is critical to meeting the dual concerns of higher early strength or reduced curing time. These methods are called accelerated curing methods. High early concrete strengths are most efficiently produced by increasing the internal temperature of the concrete while maintaining high moisture content in the curing environment. Heating reduces the relative humidity of the air surrounding the concrete. Thus, moisture must be added to the heated air to maintain the same relative humidity of the air.

Three heating methods are commonly used to accelerate curing: 1) Discharging steam or hot air directly into the curing environment puts the heating medium directly in contact with the concrete. 2) Enclosing steam or hot water in pipes heats the concrete by convection and radiation. 3) Attaching electrical resistance wires to the forms and covering them with insulation heats the product by heating the forms. Circulating steam around the products is one of the most widely used accelerated curing methods, primarily due to the ease of producing and transporting steam to the concrete member. It’s an efficient method that increases the temperature and maintains a 100% relative humidity around the concrete products. Steam can be produced in high or low-pressure boilers, then piped to the casting bed, or generated by smaller steam packs located close to the products. An advantage of steam is that it contains relatively large quantities of heat per pound of steam at a relatively low temperature. This provides both an effective and economical method of transferring heat from the boilers to the concrete products. Heating air and discharging it directly into the curing environment can also increase internal temperature. There are two problems with this type of system. First, exhaust gases of unvented fossil fuel heaters contain carbon dioxide that combines with calcium hydroxide, a byproduct of cement hydration, forming weak calcium carbonates instead of strong calcium silicate hydrates. This produces a white powder on the concrete’s surface. Second, reduced moisture in the air allows surface drying of the concrete. If heated air is used to accelerate curing, the products should be covered.
to prevent moisture loss or misted with water to increase the relative humidity of the surrounding air and prevent premature drying.

The accelerated curing cycle can be divided into three periods of preset, rising temperature, and maximum temperature. Little or no cement hydration occurs during preset. Initial set ends the preset period. Heat shouldn’t normally be applied until after initial set has occurred. Duration of the preset period is affected by admixture type and dosage, cement type, presence of pozzolans or ground granulated blast furnace slag, initial concrete temperature, and air temperature in the curing environment.

Advantages of proper curing includes: a less permeable, more water-tight concrete; reduced permeability means the concrete will be more resistant to freezing, salt scaling and attack by chemicals; prevents formation of plastic shrinkage cracks caused by rapid surface drying; increases abrasion resistance as the surface concrete will have a higher strength and significant reduction in scaling problems.

Curing should begin immediately after the finishing operation. Minimal delay is especially important in hot and/or dry weather to avoid rapid evaporation from the concrete surface. The benefits of curing concrete are significant, as can be the problems if curing is not performed as detailed above.

In order to investigate the influence of curing conditions, on the compressive strength of concrete cubes, for each mix design, three cubes were chosen for every curing regime. The curing regimes employed were: immersion in drinking water; covering with wet hessian and polythene sheet; keeping under dry laboratory conditions; keeping in open air; curing compound and steam curing. Except for steam curing system, the specimens of which were tested at the age of three days, for all other curing conditions, the compression tests were performed at the age of 28 days.

2. EXPERIMENTATION
2.1. Materials and Specimens
Concrete used in during these experiments, was made from ordinary Portland cement (binder), natural zone 2 sand (fine aggregate), basalt aggregate with maximum size of 20mm (coarse aggregate) mixed with sufficient drinking water and required additives where necessary. The mix proportions of eight different mixes used in this investigation are listed in Table 1.

<table>
<thead>
<tr>
<th>No.</th>
<th>Cement (Kg/m³)</th>
<th>Water (Kg/m³)</th>
<th>Fine aggregate (Kg/m³)</th>
<th>Coarse aggregate (Kg/m³)</th>
<th>Super Plasticizer (Kg/m³)</th>
<th>Silica Fume (Kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>250</td>
<td>205</td>
<td>1227</td>
<td>661</td>
<td>0</td>
<td>0</td>
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<tr>
<td>2</td>
<td>360</td>
<td>205</td>
<td>1050</td>
<td>730</td>
<td>0</td>
<td>0</td>
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<tr>
<td>3</td>
<td>435</td>
<td>205</td>
<td>962</td>
<td>740</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>4</td>
<td>512</td>
<td>205</td>
<td>895</td>
<td>730</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>5</td>
<td>603</td>
<td>205</td>
<td>820</td>
<td>715</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>6</td>
<td>574</td>
<td>195</td>
<td>651</td>
<td>937</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>7</td>
<td>556</td>
<td>130</td>
<td>685</td>
<td>937</td>
<td>7.85</td>
<td>0</td>
</tr>
<tr>
<td>8</td>
<td>513</td>
<td>130</td>
<td>685</td>
<td>1080</td>
<td>7.85</td>
<td>43</td>
</tr>
</tbody>
</table>
In order to see the effect of different curing regimes on the compressive strengths of concrete, 150 mm cubes were prepared using mix designs shown in Tables 1. All of the cubes were covered with damp hessian and left to cure in the moulds for 24 hours. They were then removed from the moulds and labeled. Depending on the curing method chosen, the cubes were kept in water tank, covered with wet hessian and polythene sheet, covered with chemical curing agent, left under dry laboratory conditions, left in open air, or moved to accelerated steam curing chamber. Except for the cubes cured under steam which were tested at the age of three days, the compressive strength of the specimens cured under other curing regimes were measured at the age of 28 days in accordance with BS1881: Part 116 [10]. The steam curing systems was performed as specified in ASTM C 267 and 579. The temperature cycle used in this system of curing is shown in Figure 1.

3. RESULTS AND DISCUSSION
The results of measured compressive strengths of cubes made using eight different concrete mixes and cured by immersion in water tank are shown in Figure 2.
As it can be seen from this figure, the compressive strengths recorded for eight different concrete mixes range from 10 to about 75 MPa.

As is shown in Figure 3, the compressive strengths of the cubes made out of eight different concrete mixes, cured by wet hessian and polythene sheet coverage, are shown in Figure 3. This figure shows that, under this system of curing the cubes compressive strengths are seen to be between about 12 to 80 MPa. If these values are compared with their relative values recorded for the water immersion curing systems, it can be seen that covering the concrete cubes with wet hessian and polythene sheet tend to increase the cubes compressive strengths. It should be noted that although this increase is not very significant but it reflects the importance of the covering systems and their influence on the strength attained.

![Figure 3. Compressive strengths of eight different mixtures, cured under wet hessian and polythene sheet](image)

The results of the cubes compressive strengths cured using curing compound are shown in Figure 4. It can be seen from this figure that the measured compressive strengths of the cubes cured under this system of curing, has gained strengths from about 10 to about 64 MPa. Comparison of these results with those obtained for the other two curing regimes tends to show that the use of curing compound as curing agent produces lowest compressive strengths among the three curing systems.

Examination of the results shown in figure 5, which belongs to the concrete cubes cured by steam curing, tends to suggest that this curing method has produced the lowest compressive strengths among the four curing regimes discussed so far. Because while the lowest compressive strength recorded for this system appears to be about 6 MPa, the highest value recorded is about 57 MPa.
Figure 4. Compressive strengths of eight different concrete mixtures, cured using curing compound.

It should be noted compared with 28 days used for other curing systems, the age of testing used for this system of curing was three days. The results of this series of experiments tends to show that this curing system can be used where early concrete strength is of paramount importance.

Figure 5. Compressive strengths of eight different concrete mixtures, cured by live steam.

The compressive strengths of the cubes left under dry laboratory conditions, are shown in Figure 6. Examination of Figure 6 shows that if the concrete cubes are left under dry laboratory condition, compared with the other curing systems
discussed so far, their compressive strengths would tend to decrease. This decrease appears to be more for weaker concrete mixes.

Figure 6. Compressive strengths of different concrete mixtures, left under dry laboratory conditions

Figure 7. Compressive strengths of eight different concrete mixtures, left in open air

Figure 7 shows the compressive strengths of the cubes kept in open air after their removal from the mold. Examination of these results tends to indicate that compared with normal wet curing, leaving the concrete cubes in open air affects their compressive strengths. The compressive strengths of the cubes kept in open air for 27 days, appears to rage from about 8 MPa, to about 66 MPa. Comparison of these values with the respective values of the cubes kept under dry laboratory
conditions, tends to show an increase for the attainable compressive strengths of the cubes kept in open air. It should be noted that during the conduction of these experiments the temperature of the open air never reached below about 10 degree C. Comparison of the respective results shown in figure 8 tends to suggest that, among the curing systems employed covering of the concrete cubes with wet hessian and polythene sheet appears to have the highest positive effect on the attainable concrete compressive strength. It can also be seen from this figure that, compared with the results obtained from other curing regimes, steam curing tends to produce the lowest cube compressive strengths for the employed mix designs. Figure 8 also shows that compared with wet coverage, the immersion method of curing, tends to produce lower compressive strength.

The collective results of all the cubes cured, using six different curing regimes are depicted in Figure 8.

![Figure 8. Compressive strengths of eight different concrete mixtures cured under different curing systems](image)

4. CONCLUSIONS
From the results presented and discussed in this paper following conclusions can be made:

1) Different curing systems have different effects on the compressive strength of concrete.
2) Among the curing systems employed in this research, covering with wet hessian and polythene sheet produced the highest concrete compressive strength.
3) In comparison with covering with wet hessian and polythene sheet, the immersion curing system, produced lower compressive strength.
4) Steam curing produced the lowest compressive strength among the curing systems examined.
5) Compared with wet curing systems, leaving the cubes in open air and dry laboratory conditions after 24 hours of casting, tends to produce lower compressive strength.
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AN INVESTIGATION ON THE BEHAVIOUR OF ONE-LAYER AND TWO-LAYER 3D PANELS IN SHEAR AND FLEXURAL TEST

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ABSTRACT
The 3D panels are a new system of construction. This system is used as wall and ceiling in buildings. The structural behavior of 3D panels is dependent on the strength and rigidity of connector elements. In this article flexural and shear tests have been conducted on six 3D panels. The results have been compared with results of finite element software, ANSYS. The details and results of the test program are described, and the observed behaviour patterns are discussed.

Keywords: 3D panels, shear test, flexural test, sandwich panel

1. INTRODUCTION
Precast concrete sandwich panels (PCSP) have two concrete faces and one polystyrene layer between concrete faces. The concrete faces are connected to each other with shear connectors. The arrangement and spacing of shear connectors in PCSP vary depending on several factors, such as desired composite action, applied load, span of the panel and type of shear connectors used. There are no specific rules for arranging the connectors. The complex behaviour of PCSP due to its material nonlinearity, the uncertain role of the shear connectors and the interaction between various components has led researchers to rely on experimental investigations backed by simple analytical studies. The lack of information on the behaviour of this important type of construction is due to the high cost of full scale testing and the extreme difficulty of fabrication of small-scale models.

2. EXPERIMENTAL STUDIES
2.1. Characteristics of Panels
The diameter of longitudinal bars and shear connectors is 3 mm. The dimensions are shown in Figure 2. Characteristics of panels are given in Table 1.
2.2. Flextural Test
The panels were tested using four-point test according to ASTM D3043 [1] and the data was transferred to the computer. Etch test was carried on until the complete failure of the panel. Cracking pattern was similar in all panels at this stage. The test set up for flexural test is shown in Figure 3.

Table 1: Characteristics of panels

<table>
<thead>
<tr>
<th>No.</th>
<th>Dimension (m)</th>
<th>Compressive strength $10\times10\times10$ (kg cm$^{-2}$)</th>
<th>$f_c'$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.15×1×3</td>
<td>284 Top.</td>
<td>227.2 Top.</td>
</tr>
<tr>
<td>2</td>
<td>0.15×1×3</td>
<td>318 Top.</td>
<td>254.4 Top.</td>
</tr>
<tr>
<td>3</td>
<td>0.15×1×3</td>
<td>315 Bot.</td>
<td>252</td>
</tr>
<tr>
<td>4</td>
<td>0.15×1×3</td>
<td>303 Bot.</td>
<td>242.4</td>
</tr>
<tr>
<td>5</td>
<td>0.15×1×3</td>
<td>313 Top.</td>
<td>250.4 Top.</td>
</tr>
<tr>
<td>6</td>
<td>0.15×1×3</td>
<td>315 Bot.</td>
<td>244.8</td>
</tr>
</tbody>
</table>

Figure 3. Flexural test set up
Figure 4. Cracking pattern in one layer panel in flexure
2.3. Shear Test
The shear test set up is shown in Figure 6. The load is near one of the supports in order to simulate pure shear conditions.

Cracking pattern for two-layer panels is shown in Figure 7.

3. RESULTS AND DISCUSSIONS
3.1. Flexural Test Results
Flexural test results are shown in the following Figure. The load bearing capacity
of one-layer panel is more than two-layer panel but the ductility of two-layer panel is more.

3.2. Shear Test Results
In shear test, one-layer 3D panels fail at 14 ton and have brittle behavior but two-layer panels have a ductile behavior although their load bearing capacity is about 5 tons. Figure 9 shows the behavior of one-layer and two-layer 3D panels in Shear test.

4. THEORETICAL STUDIES
4.1. Finite Element Modelling
Solid 65 and link8 elements are used to model concrete and bars in ANSYS respectively. Translations at Z direction are restrained at both supports but at X direction only one support is restrained. The following Figures show the finite element model.
Figure 10. Model for one-layer panel in flexure

Figure 11. Model for two-layer panel in flexure

5. COMPARISON OF THEORITICAL AND EXPERIMENTAL STUDIES
Experimental and theoretical results have been compared in Figure 12 and 13 for one-layer and two-layer panels.

Figure 12. One-layer panels in shear

Figure 13. Two-layer panels in shear
6. CONCLUSION
a) The rigidity of one-layer panels is more than that of two-layer panels.
b) In flexure there is no difference in the load capacity of one-layer and two-layer panels although two-layer 3D panels have no concrete in the middle layer.
c) By eliminating concrete from the middle layer and substituting it with bars the ductility of the panel increases.

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EXPERIMENTAL INVESTIGATION OF THE JACK ARCH SLAB RETROFITTED BY CONCRETE LAYER

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ABSTRACT
Considering the widespread use of jack arch roofs and the need for seismic retrofitting of this system of flooring in Iran the behaviour of the retrofitted form of these slabs is unknown. The Iranian building codes also do not deal sufficiently with this type of roofing and as a result little control is applied on their method of construction. The retrofitting method of adding a concrete layer (CL) is a method which was first introduced in Romania after the earthquake of 1990. This paper reports on an experimental investigation of this method and comparison with other retrofitting methods. For this purpose, a number of slabs with different methods of retrofitting such as the Romanian method, the method recommended by the Iranian Standard 2800, the two way method and a slab without retrofitting were constructed. Then, the slabs were loaded step by step in out of plane direction and the load-displacement pushover curves for the slabs were obtained. Using these curves, the seismic strength parameters of different slabs are determined and compared.

Keywords: jack arch slabs, retrofitting, concrete layer, masonry, pushover test

1. INTRODUCTION
Jack arch slabs have been extensively used to floor and roof urban and rural buildings in Iran. The jack-arch flooring system is stable under normal static conditions. The brick arches transfer the gravity loads, mainly in compression, to the supporting steel beams. The load is then transferred via the steel beams in flexure and shear to the load-bearing walls. However, reports of slab damage and collapse in recent earthquakes in Eastern Europe and Iran by Razani and Lee (1973) [1]; Maheri (1990) [2]; Maheri (1998) [3] and Maheri (2003) [4], reflect the weakness of the unanchored slab under dynamic loading. To overcome this problem, Moinfar (1968) [5] suggested that the slab beams be joined together at their ends by either transverse beams or by steel tie bars. This form of anchored jack-arch slab has a better seismic response because the relative movements of the slab beams are somewhat prevented. The Iranian seismic code; Standard 2800 [6], has adopted these suggestions and many slabs have recently been constructed using these anchoring methods.
It should be noted that the contemporary jack-arch slab construction in Iran is still considered a non-engineered slab in the Iranian seismic code, and there are no
particular design procedures for their engineered design. Simple methods of increasing the seismic performance of the slab in the form of inter-span transverse beams have been proposed and their effectiveness investigated both experimentally and numerically (Maheri and Imanipour [7]). Finally, procedures for engineered design and construction were introduced (Maheri and Rahmani [8] and Maheri [9]).

Following the Romanian earthquake of 1990, a number of damaged jack arch slabs were retrofitted by adding a reinforced concrete layer on top of the floor, effectively making the slab to act as a composite slab.

In this paper, a jack arch slab retrofitted with a concrete layer (Romanian method), is experimentally investigated and compared with slabs retrofitted with other methods. For this purpose, four full-scale jack arch slabs were constructed using the conventional material and workmanship. One slab was tested without any retrofitting and the other three slabs were first retrofitted by one of the; concrete layer, two-way slab and the Standard 2800 anchoring methods and then each slab was subjected to out-of-plane pushover loading and the load-displacement curves have been obtained for each slab.

**Method for Retrofitting of Jack Arch Slabs**

The above different methods of retrofitting jack arch slabs are here further discussed.

### 2. CONCRETE LAYER (ROMANIAN) METHOD:

In this method, the flooring on the slab is first removed. Then a mesh of reinforcement with maximum bar spacing equal to 100cm, is welded on the slabs beams. Finally, the slab is covered with a layer of concrete having an average thickness of around 5 cm (Figure 1).

![Figure 1. Concrete layer method for retrofitting jack arch slabs](image)

### 3. TWO WAY METHOD (MAHERI)

In 1995, Maheri presented a new method for retrofitting of jack arch slabs [10]. In this method a series of secondary beams are placed between the primary beams of the slab (Figure 2). By this method the one way jack arch slab is changed to a two way slab. Also, because the masonry arches of the slab are divided in the locations of the secondary beams, these discontinuities reduce the contribution of the masonry arch in resisting the load of the slab and effectively reduce their role to
infills. Maheri has introduced this method of construction as an engineered version of the jack arch system and has proposed procedures for its engineered design and construction.

4. IRANIAN BUILDING CODE METHOD
This method is recommended by standard 2800. In this method a cross belt is welded on the slab beams (Figure 3).

Test Specimens and Load Setup
All the specimens are made with the same size and materials. Only their retrofitting method is different. The slabs are 3.2 m×4 m in size each having 5 primary beams. The beams are IPE 12 with spacing equal to 80 cm. Traditional clay bricks are used for making the masonry arches. The rise of arch is 5 cm. In turn, out of plane pushover load was applied to each slab. The out of plane load was applied on a line in the middle of the slab to distribute the load in one direction using two hydraulic jacks (Figure 4). In each step a total 5.72kN load is applied to the specimens. The out of plane displacements of the slabs were recorded by 6 mechanical gages in each load step (Figure 5).
Figure 4. Loading setup

Figure 5. Gage positions

Figure 6. Crack pattern in the traditional jack arch slab
5. TRADITIONAL JACK ARCH SLAB
This specimen is made with the mentioned properties without any retrofitting. The results of the test on this slab can be used as a benchmark. The test was carried out on the slab 28 days after casting of the concrete layer. During the loading of the slab, the first crack occurred diagonally at one corner of the slab at the load of 22.88 kN. Further diagonal cracks developed at other corners and parallel to the corner cracks as shown in Figure 6. The ultimate load which could be sustained by the slab was 62.92 kN. However, the slab retained its integrity and no brittle failure was observed in the brick arches. The load-displacement curve for this slab, using the central gage (gage 2) is presented in Figure 7.

6. SLAB RETROFITTED BY CONCRETE LAYER:
After making the jack arch slab, the slab was retrofitted as follows:

a) The flooring was first removed and the beams top flanges and masonry arches were cleaned.

b) Slab reinforcement: If the beams of the slab are IPE 16 or more, considering the brick width (10 cm), the top flange of the beams and a minimum 6 cm of the beams web would be in contact with concrete. Therefore, using bars $\Phi 12@50$ cm in a direction perpendicular to the beams appeared to be sufficient. However, in many slabs, beams of IPE 14 or less are used and the contact between concrete and beams would not be sufficient. For this reason a connecting method, similar to that used in composite slabs was adopted. In this method, shear keys are used for increasing the concrete-beams contact. 5 cm long, $5\times5\times5$ cm angles were welded at 30 cm spacing for shear keys. A mesh with bars $\Phi 8@30$ cm, corresponding to the minimum steel ratio of $\rho_{\text{min}}=0.002$ was then welded on to the shear keys. The concrete having $f'c=30$ MPa was used to an average depth of 5 cm to cover the reinforcement mesh. After 28 days, the test was carried out in a similar manner to the first slab.
The first cracks occurred at the corners of the slab in a diagonal pattern similar to the non-retrofitted slab, at the load of 143kN (Figure 8). A small number of further diagonal cracks occurred at higher loads (Figure 8). The loading continued until the ultimate strength of the slab (194.5kN). Similar to the first slab, the behavior of this slab was also ductile and no brittle failure or collapse happened in the brick arches. The load-displacement curve is presented in

![Crack pattern in the Concrete Layer method](image)

**Figure 8. Crack pattern in the Concrete Layer method**

![Load-displacement curve for Concrete Layer method](image)

**Figure 9. Load-displacement curve for Concrete Layer method**

7. **SLAB RETROFITTED BY THE TWO WAY METHOD**

After construction of the jack arch slab, this method of retrofitting was applied to it. In this method IPE 12 were used for secondary beams and were placed in two...
rows at 1.33 m distance apart. The test was carried out in a similar way to the other slabs.
The first crack occurred at an outer brick arch at 51.48kN. After this initial crack, the diagonal cracks occurred at the corners at 85.8kN (Figure 10). The test continued until the ultimate strength of the slab (102.96kN). Similar ductile response was noted for this slab, where no spalling or collapse of brick arches happened. The load-displacement curve for this retrofitted slab is presented in Figure 11.

8. SLAB RETROFITTED BY THE STANDARD 2800 RECOMMENDATIONS
Reinforcing bars Φ14 were used for retrofitting this slab. The cross diagonal bars
were welded to top of the flanges of the main beams. After 28 days, the test was carried out in a similar manner to the other slabs. The first crack occurred at 51.48 kN at one corner. At higher loads, his crack followed by some cracks parallel to the primary beams (Figure 12). The test continued until the ultimate strength of the slab at 74.36 kN. Similar to the other slabs, no brittle failure was noted in the brick arches. The load-displacement curve for this retrofitted slab is presented in Figure 13.

Figure 12. Crack pattern in the standard 2800 method

Figure 13. Load-displacement curve for standard 2800 method
9. COMPARISON OF RESULTS

For comparison of the test results, the strength and performance parameters, derived from the load-displacement curves of gage 2 for the four slabs, are presented in Table 1.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Traditional (non-retrofitted)</th>
<th>Concrete layer</th>
<th>Two way</th>
<th>Standard 2800</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ultimate Strength (kN)</td>
<td>63</td>
<td>194</td>
<td>102</td>
<td>36</td>
</tr>
<tr>
<td>Ultimate displacement (mm)</td>
<td>22</td>
<td>45</td>
<td>29</td>
<td>25</td>
</tr>
<tr>
<td>Toughness (kN.mm)</td>
<td>6690</td>
<td>69000</td>
<td>14000</td>
<td>7900</td>
</tr>
<tr>
<td>Ductility</td>
<td>1.7</td>
<td>2.84</td>
<td>2.21</td>
<td>1.46</td>
</tr>
</tbody>
</table>

Ultimate Strength

The concrete layer method increased the ultimate strength of the slab by up to 3 times. This method has the highest effect on increasing the ultimate out of plane strength of the jack arch compared to the other retrofiting methods (Figure 14).

Ultimate displacement

The Ultimate displacement in the slab retrofitted by the concrete layer method was 198% more than the traditional slab (Figure 15).

Ductility ratio

Ductility ratio in the slab retrofitted by concrete layer was 167% more than the traditional slab (Figure 16).

Toughness

The concrete layer increased the toughness of the slab 10 folds compared to the traditional jack arch. This method has the highest effect on the toughness with respect to the other retrofitting methods (Figure 17).

Weight of slabs

The increase in the weight of the slab due to retrofitting is not suitable, because it increases the gravity and seismic loads and puts further demands on other elements of the buildings. Adding a concrete layer increased the weight of the slab by 25%. This is a considerable amount of weight increase compared to other retrofitting methods (Figure 18).

Construction cost

In the concrete layer method, the retrofitting costs are divided in two parts. First, cost for retrofitting the slab, which is around 2.6 times the cost of construction of a
traditional slab. Second, the costs for retrofitting other element, due to weight increase. This part is a function of the type of structural system and is unique for each building (Figure 19). It seems that the concrete layer method, though effective in considerably increasing the strength and performance parameters of the slab, compared to other methods, it suffers from being more time consuming, technically more difficult to perform and particularly far more costly.

In following Figures, 1, 2, 3 and 4 respectively stand for traditional, standard 2800, two way and Concrete Layer.
10. CONCLUSIONS

Based on the results of the tests presented in this paper the following conclusions can be drawn regarding the effectiveness of the concrete layer (Romanian) retrofitting method:

1. The method is very effective in increasing the strength and ductility of the jack arch slab and enhancing its seismic performance. However, it is time consuming and particularly suffers from the excess cost, both in terms of retrofitting the slab itself and the secondary costs of strengthening other elements of the building due to the increased weight.

2. To reduce the secondary costs of the method, it is recommended that after the positioning of the concrete layer, the brick arches be removed, in effect, converting the jack arch slab into a composite steel-concrete slab. However, by doing so, the primary costs of the retrofitting will be increased due to the removal of the brick arches and construction of the necessary false ceiling.

3. Considering the cost-ineffectiveness of the method, it is recommended that it be used to retrofit special buildings or buildings in which due to the overstrength of its members, the secondary cost of retrofitting are minimal.

REFERENCES


تغییرشکل‌ها و آسیب‌های وقوع آمده در برج‌های خنک کننده بینی در اثر نشست‌های غیرکنواخت

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چکیده

نشست غیرکنواخت در سازه‌های بزرگ، از اهمیت زیادی برخوردار است. این موضوع در برج‌های خنک کننده، به‌دلیل برقراری ابعاد سازه در حدود قطر و ارتفاع، اهمیت بیشتری می‌یابد. نشست غیرکنواخت تحت‌جایی در این برج‌ها به‌دلیل طول زیاد، بایستی برای افزایش آن باشد. همچنین در این برج‌ها، برای جلوگیری از تغییرات غیرخاطف مصالح و هندسه صورت افزایش است. نتایج نشان می‌دهد بی‌بی‌گیرشکل افقی بین دو کن و اسیب‌کشی زیادی می‌بیند. همچنین سنون و پوسته‌های تغییرمکانی بسته به وزن پایان کننده و تغییرات، تغییری‌ها در فاکتوری که سبب تغییر مکانی‌ها در سازه‌ها می‌شود در درصد‌های بیشتری از این اتفاقات تشکیل می‌دهند.

کلیدواژه‌ها: برج‌های خنک کننده بینی، نشست غیرکنواخت، مدل پلاستیستیک آسیب دیده بینی

۱- مقدمه

برج‌های خنک کننده سازه‌های غیر אוپن‌سیسی هستند که از پتوه ساخته شده است. با توجه به طول زیاد

پایه این ابرکنگ، باید ارتفاع زیاد و نشست‌های غیرکنواخت را توسط دو ایجادان نهایی متقابل در سنون و در نتیجه نشست

غرب‌کنواخت می‌شود.

نشست غیرکنواخت در سازه‌ها به عنوان یکی از ایجاد‌های سازه‌های غیر اوپن‌سیسی است، باعث ایجاد انحرافات

اولین تحقیق در مورد نشست غیرکنواخت برج‌های خنک کننده، به نام "گلدست" در سال ١٩٧٢ خورشیدی گرفته

[1] وی برای سازه‌ها به شکل‌ها، آنها به اندکی در بخش‌های مجاور چاپ برنامه نسبت به سه‌نواه‌ها مجاری خود چاپ شده باشد و
میزان این گالیکسی با اندازه‌بندی پایه که نیروی محوری ستون ناتی زیر مرده در آن این گالیکسی برای صفر گردید.

وی همچنین فرض می‌کند این نشست، صرفاً موج تغییر در نیروی پیش ستون مجازی گردید.

بعد از «گوگان»، وی در زمینه و بررسی نشست غیریکتوخت پرداخت. وی در انتقال خود از مدل و فرضیه 

«گوگان» استفاده نمود. وی بررسی نشست کاشت از دیدگاه که نیروی تفاوتی ناشی از بار مرده آن به صفر تقلیل یابد. همچنین این نشست، فقط موجب تغییر در نیروی پیش ستون مجازی گردید. وی در نتیجه گوگان، وی به استفاده از روش محدود و توسط کامپیوتر آنالیز نمود و مشاهده کرد با دور شدن از محل ستون نشست کرده این نشست در ستون‌ها کمتر است.

سپس «کریشنای» در سال 1992 به بررسی نشست غیریکتوخت در پرده‌های خانه کننده پرداخت. او در آنالیز خود از منطق «گوگان» پره جست و اثر نشست یک ستون را بررسی کرد، وی به مقدار نشست را به یکی از شرایط اعمال کرده که در آن، نیروی محوری ستون در اثر بر مرده صفر گردید. وی با استفاده از ترم افزایش، برج می‌شود. 

برخی «نوشته» در مقاله نیروی ستون‌ها صرفاً یک ستون را وارد محاسابات نموده بود ولی «کریشنای» ستون‌ها را توسط برنامه آنالیز نمود و به دلیل شکل‌پذیری ستون، قسمتی از نشست به پوسته منتقل می‌شود. نیروگاه اسپهان را تحت مدل ریاضی کسب کرده در محدوده استاتیک آنالیز نمود و به این ترتیب ریزشی که به میان‌ترین تغییرات نیروها در قسمت پایین وادارشده با گزارش ارتفاع، مقدار نیروهای کاهشی می‌پذیرد.

در سال 2004 «شری» به مطالعه نشست غیریکتوخت پرداخت. وی نیز نشست غیریکتوخت را به صورت مدل ریاضی کسب کرده، در نظر گرفته و با استفاده از نرم افزار ANSYS5.4، برخی از آنالیز غیر خطی نمود و به این ترتیب ریزشی که نشست‌ها به دلیل نیرویهای مختلفی است که مقدار زیادی نسبت به حال خطا کاهش می‌یابد و بر مقدار تغییر شکل برج افزایش پیدا کند [9].

مدل ریاضی نشست غیریکتوخت

در سال 1977، «چیلسکی» برای رسیدن به یک الگوی مناسب نشست، نشست غیریکتوخت 4 برج را در مدت 6 سال اندازه‌گیری کرد [4]. و برای هر برج در سال‌های مختلف نشست‌های غیریکتوخت بدست آمد را با سری‌های ریاضی تقریب زد و به مدل ریاضی زیر می‌پرداخت:

$$\omega = A_0 + A_1 \cos \beta + B_1 \sin \beta + A_2 \cos 2\beta + B_2 \sin 2\beta + A_3 \cos 3\beta$$

$$+ B_3 \sin 3\beta + A_4 \cos 4\beta + B_4 \sin 4\beta$$

که در رابطه فوق، $\omega$ نشست بر حسب متراً، $\beta$ زاویه مرکزی و $A_0$ تا $A_4$ و $B_1$ و همچنین $A_1$ و $A_2$ و $A_3$ و $A_4$ و $B_1$ و $B_2$ و $B_3$ و $B_4$ عددهای ثابت

1-Krishna
2-Ciesielski
در اینجا بر اساس مدل هندسه شده توسط رابطه (1) آنالیز کرده و چنین تجربه گرفته که در صورتی که نشان گذاره، طبق جدول این برچه، تا حدا وارد است که مکان است باید خرای آنها توجه کند.

اندازه گیری‌های واقعی انجام شده توسط «چیلسکی» نشان می‌دهد که فرض نشان صرفاً یکی از ستون‌ها در برچه‌ای یکنکت که به عنوان نشان غیر یکنواخت، نمی‌تواند گویه همه واقيع داشته باشد، دلی به مقدار برای تعیین یک مدل مناسب به طوری که توانای پاسخگویی هر گونه نشست غیر یکنواخت احتمالی باشد، نیاز به رابطه کلزا» و «ممتازاً»، مدل ریاضی بهتری در برای نشست غیر یکنواخت برچه‌ای خنک کننده به صورت زیر ارائه نمودند [5]؛

\[ \omega = 0.03 \cos 2\beta \]

(3)

\[ \omega = \Delta U \cos(n\theta) \]

(4)

که در آن \( \Delta U \) پرتاب نصف حاکم اختلاف نشست اعمال شده ی \( \theta \) را به زاویه مرکزی \( n \) با توجه به تعداد \( \Delta U \) ستون‌های بریخ خنک کننده از \( \theta \) نصف تعداد ستون‌های بریخ می‌تواند در نظر گرفته شود. آنها با اعمال رابطه (3) برای دو مقدار \( n = 2 \) و \( n \) نشان‌دهنده شده بودند، نیروهای داخلی و همچنین تغییر‌گرایی سازه بریخ مذکور بر این اثرات که در مطالعه اثر نتیجه‌گیری کرده و مشاهده نمودند افزایش نیروهای شغبی مکنده در نواحی بریخ‌ها سخت کننده تحریک و فوکالی قابل توجه است.

از طرف دیگر «کاتا» و همکارانش نشست بریخ خنک کننده از دیدگاه آمار برای نمودند [6] این که در برای نشست غیر یکنواخت قالم برچه‌ای خنک کننده در نظر گرفته که \( U = U_0 + \sum_{i=1}^{n} U_i \cos(i\theta - \theta_i) \)

(5)

مقدار نشست یکنواخت می‌باشد و \( U_0 \) به ترتیب عبارتند از ضربی و خاک‌ها و اختلاف فاز برای هر مونیک آم. این اختلاف‌های فاز \( \theta_0 \) به صورت تصادفی تغییر می‌کنند، بنابراین نیازی به محاسبه ندارند. مقدار متوسط نشست \( U_0 \) توسط آنالیز کامپیوتری و با مدل کردن خاک زیر بریخ توسط فنرهاي بدست می‌آید. بنابراین تعیین شد که با روش‌های مختلفی قابل حصول می‌باشد. یک روش ای این است که با داشتن شناخت کافی از خاک منطقه و اندامه گیری‌هایی که روی سازه می‌شود، ضرایب بدست آیند. نظر کاری که "چیلسکی" انجام داد در روش دیگر با توجه به اطلاعات موجود در زمینه سازه و خاک، زیر آن به تبعیین انحراف معیار و میانگین نشست اقدام نموده و با توجه به آن دو متغیر، نشست ماکزیم و نشست مینیم را بدست آورد و در تابع اختلاف نشست حاصل می‌شود.

1- Kaluza
2- Mateja
با توجه به اینکه نوع خاک می‌تواند نقش تعیین‌کننده‌ای در رابطه با حداکثر اختلاف نشست داشته باشد، لذا

\[ \frac{\partial}{\partial \theta} \]...

\[ \omega = \frac{k}{2 - k} U_0 \cos(n\theta) \]

به تعداد سنونه‌های برخ دو کننده از دو نصف تعداد سنونه‌های برخ می‌تواند در نظر گرفته شود. \( U \) گمانگین نشست برخ تا آمر ورود به فاصله دید عالیکی می‌باشد و \( K \) نصف اختلاف نشست حداکثر درون سنونه‌ها یا می‌باشد و ضریب \( k \) حداکثر بسیاری است که با توجه به جنس خاک می‌توان آن را با پنج‌نیا استخراج نمود. برای مقادیر \( k \) و \( U \) برای خاک‌های رسی و حداکثر سطح خاکی که برابر با \( 0.27 \) برای خاک‌های ماسه‌ای مشخص است. \( k \) برای خاک‌های آسفالتی مشخص نشست در مسیرهای مختلف پیشنهاد داده شده، برای مشاهده جدول (1) مشخص می‌شود که مقدار نشست کل در خاک‌های ماسه‌ای کمتر از خاک‌های رسی می‌باشد و اما اختلاف نشست در خاک‌های ماسه‌ای نسبت به حداکثر است در این خاک‌های رسی، این اختلاف نشست بسیار کوچکتر از نشست حداکثر می‌باشد. \[ \text{جدول 1: مقایسه نشست در خاک‌های ماسه‌ای و ماسه‌ای} \]

<table>
<thead>
<tr>
<th>ریس (به طور معمول تحکیم)</th>
<th>ماشه</th>
<th>یافته و با کمیت‌یافته از حد (تحکیم یافته)</th>
</tr>
</thead>
<tbody>
<tr>
<td>پیش‌گیر</td>
<td>( r_{max} )</td>
<td>به خصوص تحت اثر بارزهای \</td>
</tr>
<tr>
<td>کوچک</td>
<td>بزرگ</td>
<td>سریع</td>
</tr>
<tr>
<td>ازه‌سی</td>
<td>نشست</td>
<td>به شکل بی‌شکلی</td>
</tr>
<tr>
<td>اکتیو</td>
<td>بزرگ در ویژه‌ی ( \beta ) قاده‌دای</td>
<td>معمولاً خیالی کمتر از</td>
</tr>
<tr>
<td>( r_{max} )</td>
<td>\</td>
<td>( \Delta r_{max} )</td>
</tr>
<tr>
<td>رابطه‌ی بین ( \Delta r_{max} )</td>
<td>( \rho_{max} )</td>
<td>( \Delta r_{max} )</td>
</tr>
<tr>
<td>ازه‌سی</td>
<td>به‌سه‌شیبی</td>
<td>بزرگتر ( \beta ) قاده‌دای</td>
</tr>
<tr>
<td>اکتیو</td>
<td>سریع</td>
<td>سریع اتفاق می‌افتد</td>
</tr>
</tbody>
</table>

مدلسازی بدن برخ

در این تحقیق، برج شاذند از دو ارتفاع 130 متر و شاخه منظم بین 35/183 متر مدل شده است. بپ، سنونه و پوسته به ترتیب فلزهای حجمی \( \text{S}4, \text{S}3, \text{R} \) (shell) و \( \text{B}3, \text{I} \) (beam) و \( \text{C}3 \) دارند. مدل مانبند شده برج در شکل (1) نشان داده شده است. المانهای حجمی در هر 3 درجه آزادی انتحال دارند. المانهای طی و پوسته‌ای در هر 3 درجه آزادی انتحال دارند. با توجه به
اینکه ضخامت المان‌های پی نسبت به دو دیگر کش بالاتر است. اگر استفاده از المان‌های پیوسته در پی باشد ایجاد خطاهای در بزرگ بوده است، همچنین با در بررسی سطح مقیاس بزرگ المان‌های پی نسبت به طولشان و تابع بی‌دیده در مورد کم کشی بی‌پیشی خاک استفاده از المان‌های تیر در پی نماید. منطقی نیز به دولت عده ساختمانی ساخته شده است. استفاده از المان‌های حجمی در استخراج، باعث می‌شود که نتایج قطع براساس تشکیل شود، که این کار از نظر مهندسی قابل قبول نمی‌باشد و پیش از استخراج‌های نیروی و لنگر را نیز داشته باشیم لذا المان‌های تیر انتخاب گردیدند. جهت بدست آوردن نتایج دقیق در تکنیکی محل آنالیز پوشه به ستونها، تا انتخاب 22 متر، از مش بندی ریزتری در پوشه استفاده شده است.

![شکل 1 - مدل المان بندی شده بر جر شانزده اراک](image)

برخی از آرامترهای پی به صورت آرامترهای منفرد و برخی دیگر، به صورت لایه‌ای و یا از مدل شده است. آرامترهای استوایی به صورت منفرد و آرامترهای پیوسته به صورت لایه‌ای و یا از مدل شده است. ویژگی‌های فولاد برای مدلسازی آرامترهای به صورت دو خطی، با ساخت شوینده السامانی‌یکی در نظر گرفته شده است.

در جدول (2) مشخصات فولاد مورد استفاده در آرامترهای آمده است.

<table>
<thead>
<tr>
<th>$f_y$ (MPa)</th>
<th>$E$ (MPa)</th>
<th>$E_s$ (MPa)</th>
<th>$\rho$ (kg/m$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7850</td>
<td>$21E5$</td>
<td>$21E3$</td>
<td>7850</td>
</tr>
</tbody>
</table>

برای مدل کردن بین، در پی و بویه، از مدل پلاستیسیته آسیب دیده بین؛ در دستون‌ها از مدل ترک پوششی استفاده شده است. مدل پلاستیسیته آسیب دیده بین، با تابع آرامی‌سازی مطابقت پهپدی دارد. برای باینابای سختی به هنگام وارد شدن از کشی به فشار را در نظر می‌گیرد، همچنین در این مدل، کاهش سختی به هنگام پاربرداری، در نظر گرفته می‌شود که این کاهش با افزایش کرنش پلاستیکی، افزایش می‌یابد. مشخصات بین به کار رفته در جدول (3) آمده است. با توجه به مشخصات زیر، مختصه‌های نشش-کرنش فشاری و کششی پراورد شده‌اند.

شده‌اند .
جدول 3: مشخصات بتن به کار رفته در مدل

<table>
<thead>
<tr>
<th>$f'_i$ (MPa)</th>
<th>$f''_i$ (MPa)</th>
<th>$E_s$ (MPa)</th>
<th>$E_o$ (MPa)</th>
<th>$V$</th>
<th>$\rho$ (kg/m$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>28</td>
<td>2/3</td>
<td>31500</td>
<td>31500</td>
<td>1/2</td>
<td>2400</td>
</tr>
</tbody>
</table>

مدلسازی خاک زیر سازه

برای مدل کردن خاک، از فرآیندی در 3 جهت استفاده شده است. ساخته فرآیندی که تأیید از شعاع‌های پی و مدول برشی خاک می‌باشد، از روی نمودارهای ارائه شده، محاسبه شده است[10]. مدل ریاضی نشست غیر بکوانتیت

به‌طور کل، نشست چاپی از طریق نشست چاپی با ماسه‌ای بودن $k$, نصف حداکثر اکثرا نشست پی باشد که با ماسه‌ای بودن خاک، حداکثر اختلاف نشست اعمال شده همان نشست میانگین پی، $U$, $U$، می‌باشد؛ یعنی

$\omega = 0.5 U \cos(n \theta)$

در اینهای و ایجادهای موجود آمده

اینگونه باشد، سپس موهای مختلف نشست به آن اعمال گردد و آنالیز‌ها صورت گرفت.

منظور از مود نشست صفر، آنالیز تحت وزن می‌باشد. نتایج تحلیل‌ها در نمودارهای مربوطه آمده است.

1-

![شکل 1](image1)

شکل 1- نشانه‌های مورد نشست

2-

![شکل 2](image2)

شکل 2- نقشه آبیاری کششی پی

3-

![شکل 3](image3)

شکل 3- درصد اکثرا دیدگی کششی پی
تغییر شکل‌ها و اسباب‌های بوجود آمده در برج‌های خانگی‌هند... / 17

در اثر نشست غیر یکنواخت، حداکثر اسباب کششی پی از مقدار صفر در حالی تحت وزن به مقدار 65٪ در مود نشست، رسیده است. لازم به ذکر است که در اثر نشست غیر یکنواخت، پی هیچ گونه اسباب فشاری ندیده است و مقدار آن در تمام مواد صفر می‌باشد.

در مود نشست n=2 و n=3 و n=4، مانند حال تحت وزن، اصلاً اسباب کششی ندیده است. و بعد از آن با افزایش شماره مود نشست، درصد اسباب دیدگی کششی افزایش یافته است. به طوری که مقدار آن در مود نشست n=18، رخ داده است. یعنی در اثر نشست غیر یکنواخت، 87٪ پی اسباب کششی دیده است.

نشست غیر یکنواخت باعث شده است که در پی، تغییر شکل افقی بوجود آید. به طوری که در جراین ترین حالات، که مود نشست n=6، مقدار مکرزم آن از 27/7 mm در حالی تحت وزن به مقدار 65٪ رسیده است. یعنی مقدار آن، 11/7 برابر شده است. همچنین مکرزم تغییر شکل نسبی پی 3/03000/0/300000 درصد می‌باشد. (منظر از تغییر شکل نسبی پی، نسبت تغییر شکل افقی به قطر آن می‌باشد).

شکل (5)، شکل تغییرات پی را در مود نشست n=18 نشان می‌دهد. گفتند است که قرم تغییرات پی، برای وضوح بیشتر، مقیاس شده است.

شکل 4- مکرزم تغییر شکل افقی بوجود آمده در پی

شکل تغییرات پی نسبت به حال وزن
شکل ۵- قریب‌تری به‌افته یی در مود نشست ۱۸

چانچه کانتور آسیب کششی یی را در مود نشست ۱۸ را در نظر بگیریم که میزان آسیب کششی و نیز درصد آسیب دیدگی کششی یی، از بقیه مودها بیشتر است: مشاهده می‌گردد که ناحیه آسیب کششی یی از لبه خارجی آن که در پشت سطون می‌باشد، آغاز شده و به ترتیب در ناحیه اطراف ستون‌ها گسترش یافته است. لازم به ذکر است که نرخ آسیب کششی در اطراف ستون‌هایی که نشستشان یا در نظر گرفتن نشست تحت وزن

\[ n = u_0 + 0.5u_0 \]

می‌باشد و میانگین بیشتر است از ستون‌هایی که نشست همزمان وزن‌بان، می‌باشد. این پی اطراحی ستون‌هایی که نشست غیرکنارآست بان شده به سمت بالا جابجا یا بازگشت باشد، بیشتر آسیب کششی دیده‌اند. کانتور آسیب کششی یی در مود نشست ۱۸ در شکل (۶) نشان داده شده است.

شکل ۶- کانتور آسیب کششی یی در مود نشست ۱۸

شکل ۷- ماکزیمم تغییر‌مکان جانبي بوجود آمده در ستون‌ها
در اثر نشست غیرپکوخت، ماکزیمم تغییر مکان جانبی ستون‌ها از ۱/۸۵ mm در حالت وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن ب‌ه‌م ۳۵/۱۸ mm در حالت وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به وزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن به الوزن بأسماء يوجد أسماء في العدد 19/
در پوسته خود این مطالعه، افزایش وزن و مقدار نشست در آن درصدی است که وابسته به تعداد سیانه‌ها و تعداد ترک خرده است. این نشست در مواردی که تعداد ترک‌هایی که در کل سیانه‌ها وجود می‌آید را نسبت به مقدار مکرر مقدار تعداد ترک‌ها، نمای شود که در نمودار (10) نشان داده می‌شود. تعداد ترک‌های سیانه‌ها در مقدار نشست در مواردی که احتمال نشست در سیانه‌ها در 10% (می‌باشد در نمودار) تغییر نمی‌کند. می‌تواند در آنها ماهیت نشست در شکل 11- ماکزیمم تغییر مکانی در مورد آزمایش در پوسته


tغییر نشست غیرنکته‌ای، مقدار آسیب دیدگی کششی در کل پوسته در تمام ماهیت‌های نشست به جز ماهیت

\( n=2 \) نشست در آن (می‌باشد)، این می‌تواند سیانه در ماهیت‌های نشست در سیانه‌ها در 10% (می‌باشد در نمودار) تغییر نمی‌کند. می‌تواند در آنها ماهیت نشست در شکل 11- ماکزیمم تغییر مکانی در مورد آزمایش در پوسته

\[ n=2 \]
همچنین در مود نشست ۵% یا ۴/۸ نتایج حذفی رنگ اکتشافی دچار اسپی دیده شده است و مقیاس این اسپیدیکس نیز در آب‌میوهی که مدیر مراکزیم آن، ۱۰٪ می‌باشد.

لازم به ذکر است که در اثر نشست غیرکنارختی، پوسه در هر یک از مودهای نشست، اسپیلاشی ندیده اسری در گونه‌ای که مقیاس آن در تمامی مودها برای صفر بسته بود.

ماکزیم تغییر مکان چانی پوسته از ۲/۱۰ مم در مود نشست ۳/۱۰ مم ریزیده است. بنی همکیا مکرمت آن نسیب به حالت ورژن ۳/۰۷ در برای شده است. همچنین ماکزیم تغییر مکان نسبی آن ۹۴۵/۰، ۹۵/۰ درصد می‌باشد. ماکزیم تغییر مکان چانی پوسته تحت نشست غیرکنارختی که در ۰/۱۰ مم بوده است در ارتقای ۶/۱۳ متری پوسته (بنی ۹۴/۰، ارتقای پوسته) رخ داده است. همچنین قابل مشاهده است که در اکثر مودهای نشست، ماکزیم تغییر مکان چانی پوسته در ابتدا ناحیه حذفی رنگ تحت‌نیاپوسه (نشست ۲/۱۰) در آن رخ داده است. گفتگوی این که شکل مزارک برای وضوح بهتر بزرگ‌نمایی شده است.

نتایج گستر

۸۷% می‌اسب کشیشی دیده است به گونه‌ای که مقیاس ماکزیم آسیب کششی آن، ۵۵٪ می‌باشد. همچنین اطراف ستونها، بیشتر اسیب کششی دیده است. اما یپ هیچگونه اسیب فشاری ندیده است. ماکزیم تغییر شکل نسبی یپ ۵۳/۰ درصد می‌باشد که در مود نشست ۴/۶ رخ داده است لذا این مود برای کنترل تغییر شکل یپ در بارگذاری نشست غیرکنارختی، مناسب‌تر می‌باشد. ماکزیم تغییر مکان نسبی بیشتر در مود نشست در ستون‌ها، ۱۳/۸ درصد می‌باشد. همچنین ۵۰٪ سنون‌ها دارای خودروهای اسیب اطراف رنگ فوقانی و ناحیه جدولی رنگ تحتالمی اسپی کششی ناجیزی دیده است به گونه‌ای که مقیاس ماکزیم آن ۴۲٪ می‌باشد. اما
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کلیدواژه‌ها: مقاوم‌سازی خمشی، تیرهای بتن مسلح، مدل آپسید پلاستیکی بتن، CFRP

مقدمه
در حالی که سرویس اعضا خمشی سازه‌های بتن مسلح ممکن است نیاز به تعویض به علت خشکسالی، آرام‌نوری، و خرابی بتن، انتقال نیروی بتن از افزایش بارهای پرده‌دار و یا خرابی‌های پیش‌تر بتن، نشان‌دهنده کاهش استحکام و ظرفیت خهاب یکی از تأثیرات اصلی در این راه‌اندازی طراحی بیشتر برای مقاوم‌سازی دیسکی به درصد آرام‌نوری بتن، کاسته می‌شود.

چکیده
بزرگی عدیدی در تیرهای خمشی بتن مسلح تقویت شده با CFRP و نزدیک سطحی (NSM) به سطوح خارجی عضاء و استفاده از CFRP به روش NFPM به عنوان یکی از تکنولوژی‌های جدید می‌تواند یکی از راه‌هایی با قدرت تقویت خم شی بی‌چسبانی FRP بروهای تقریبی که در سطوح خارجی کشانده شده‌اند، شود. نظر به اینکه در تقویت با روش NFPM نزدیک سطحی (NSM) فرهنگ به دنبال این موضوع در مرور و ارزیابی دیدگاه‌ها و مدل‌سازی اثرات نزدیک سطحی روش‌های تقویت شده که در آن به مقاومت‌سازی مداخله می‌شود، به عنوان آزمایشگاهی از آن‌ها استفاده شده‌اند. نوارهای FRP از این نظر اهمیت داشته‌اند.

کلمه‌های کلیدی: Tریهای خم‌شی کسیFRP، NFPM، سطح‌های نزدیک، مقاومت‌سازی، نوری آرام، آماری، پاسخ‌های طراحی، جواب‌ها، نمونه‌های آزمایشگاهی.
شناخته شده جهت تقویت خمشی برخی، دیوارها و دالها، چسباندن و ورقه‌ای به سطح خارجی آنها می‌باشد. از خروردها و مصالح مطابق با شرایط اعمال شده برای آنها و مواد در اکثر کاربردهای مربوط به فضه‌سازی و مقاومت به سطح خارجی برخی از این بهبودها در مقایسه با جداسازی، افزایش یافته در نتایج ایجاد می‌شود.

در این بخش، شکست در ورقه‌ای که به‌عنوان یکی از موارد اصلی توانایی مقاومت به سطح خارجی در بخش اول نشان داده شد، با توجه به موارد مالیاتی، آشفتگی و همچنین به‌عنوان یکی از موارد اصلی توانایی مقاومت به سطح خارجی در بخش اول نشان داده شد، برای آنها نتایج ایجاد می‌شود.

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1- بررسی مطالعات آزمایشگاهی نمونه‌ها


2- مدل سازی عدید تیرهای تقویت شده در نرم افزار اجزاء محدود

1-۱ بخش بتنی

تیرهای بتنی بصورت سه بعدی مثل شده اند. در حالی که امکان مدلسازی آنها به صورت دو بعدی نیز وجود دارد. در آنالیز تیرهای بصورت کامل مثل شده اند. لیکن جهت کاهش زمان محاسبات می‌توان از تقاضای تیرها نیز استفاده نمود. جهت مش بردن تیر از طرف C3D8R استفاده شده است.
دو روش جهت اختصاص المان آرمانی وجود دارد. در روش اول، از المان تیر و با خریدا جهت معرفی آرمانی استفاده و در روش دوم آرمانی‌ها در یک و یا چند ناحیه بهصورت بی‌پشتیهای با فاصله پیکاواخت معرفی می‌شوند. لاهی جهت آرمانی بهصورت لاهی‌های خش، هم در حجم المان با ضخامت تابی برای‌سازمان، سه مساحت در می‌گردند تا فاصله میلگردشان در نظر گرفته می‌شوند. در این مقاله از المان خربا با مش بندی سه بعدی خصوص T3D2 جهت معرفی آرمانی‌ها استفاده است.

CFRP

وارقه‌های و نوارهای Hibbitt

ورقه‌های با المان‌های استاندارد و خویش با نام S4R مدل شده است. این نوع المان توسط Karlsson & Sorensen Inc (1997) نیز بصورت المان خریا مدل شده است.

2-2 شیب سازی رفتار مصالح

1-2-1 بنی

از مدل استاتیک پلاستیک برای شیب سازی رفتار این استفاده می‌شود. این مدل برای اولین بار در معرفی شده است. در روش دوم از اولین ابتدا بیان شده است. در این مقاله از هر دوی این ناحیه توسط اساسی از استاتیک، روش گستری در خاک با دیسک و گیره به‌صورت ایجاد شده است. در این روش، اطلاعات مربوط به روش گستری در کشف با استفاده از این روش کشف گردیده است.

2-2-2 فولاد

از مدل استاتیک پلاستیک برای هر مدل سازی رفتار فولاد استفاده شده است. در این روش فولاد تا رسیدن به ناحیه ترسیم، استاتیک می‌باشد. در این ناحیه تحت ناحیه ترسیم فولاد نیز مدل ضریب کششی فولادی \(E\) به مقدار 201 N/m به دست می‌آید.
بررسی عددی رفتار تیرهای خمشی بین مسئله تقییت...\(29\)

شکل ۵- منحنی نرم شووندگی بین: (a) تقریب خطی (b) تقریب دو خطی (c) حالت کلی

شکل ۶- انحراف شکست در نمودار نشی تغییر مکان


CFRP

از مدل شکست ترد برای شیبی سایر رفتار ورقه‌ای و نوارهای CFRP استفاده می‌شود. دراین روش فرض می‌شود که کرنش نهایی، خطی است. دراین نقطه ثابت کرنش می‌باشد و ماده تمامی تغییرات برای خود را یک‌پاره از دست می‌دهد. پارامترهای مورد نظر این مدل ضریب کشسانی پلیمرهای ایلایی (\(E_{\text{FRP}}\))، ضریب پوشاک (\(v\)) و کرنش نهایی شکست (\(f_{\text{c}}\)) می‌باشد.

۳- تحلیل عددی

تحلیل عددی دراین مطالعه با استفاده از نرم‌افزار ABAQUS انجام شده است. دراین شیب سازی از تماس Embeded region برای ادغام نوارهای CFRP برای چسباندن ورقه‌ای (Tied Contact) مقید به‌بنادر استفاده شده است. از مجموع ۶۷ نمونه آزمایشگاهی مراجع [۱] و [۲]، خاک و یک پس‌نمونه به‌نام B2-12D-1L15، B7-16D-1L15، B6-16D-1L10، B3-12D-2L15 در مرحله [۲] و خاک‌پوش، ۹-1Fa&amp;b ۱-6 از مرجع [۱]. جهت حل مدل حل عددی استفاده شده است.

۴- مقایسه نتایج عددی و نتایج آزمایشگاهی

نمونه برای تغییر مکان وسط دهانه نمونه‌ها برای چهار تیر سری اول و چهار تیر سری دوم به همراه نتایج آزمایشگاهی آن در شکل‌های ۸ و ۹ آورده شده است.
5- مقایسه بین نتایج آنالیز اجزای محدود با روابط ACI440.2R02 و ACI318-05
نسبت بارهای نهایی که براساس روابط ACI440.2R02 محسوب شده در جدول شماره 1 برای نمونه‌های نشان شده، با ورقه‌ای ناهنجاری شده است. این نسبت‌ها برای نمونه‌های B3 و B0 از یک کمتر است. در حالیکه برای CFRP نمونه‌های B7 و B0 بنابراین تحلیل نتایج نمونه‌های CFRP به روش NSA در جدول شماره 3، مداربری ارائه شده در مرجع [1] محاسبه و با نتایج عدید مقایسه‌ای نتایج شده است (جدول 2). مقاومت نهایی تیروهای ناهنجاری نشده نیز براساس روابط 05-318 ACI [13] محاسبه و درجدول زیر با عنوان P_{\text{NSM}} ارده شده است.

شکل 8-الف- نمودار بار گچبری و سطح دهنده برای تیروهای مقاوم شده با ورقه‌ای CFRP

شکل 8-ب- نمودار بار گچبری و سطح دهنده برای تیروهای مقاوم شده با ورقه‌ای CFRP
بررسی عددی رفتار تیرهای خمشی بتن مسلح تقویت شده…/ 31

شکل 9- نمودار بار جاجیبی و سطح دهانه برابر تیرهای مقاوم شده با نوارهای CFRP به روش NSM

جدول ۱: مقایسه بین نتایج اجزاء محدود و روابط محاسبه شده توسط ACI440.2R.02

<table>
<thead>
<tr>
<th>نام نمونه</th>
<th>ρ</th>
<th>P_con (kN)</th>
<th>P_ACI (kN)</th>
<th>P_FEA (kN)</th>
<th>P_P (kN)</th>
<th>P_Yield (kN)</th>
<th>P_Yield_con (kN)</th>
<th>P_Yield_FEA (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B2-12D-1L15</td>
<td>0.3ρ_b</td>
<td>41.25</td>
<td>68.98</td>
<td>61.30</td>
<td>1.67</td>
<td>1.48</td>
<td>0.89</td>
<td></td>
</tr>
<tr>
<td>B3-12D-2L15</td>
<td>0.6ρ_b</td>
<td>60.86</td>
<td>83.59</td>
<td>86.37</td>
<td>1.37</td>
<td>1.41</td>
<td>1.03</td>
<td></td>
</tr>
<tr>
<td>B6-16D-1L10</td>
<td>0.6ρ_b</td>
<td>88.51</td>
<td>98.42</td>
<td>1.45</td>
<td>1.61</td>
<td>1.11</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B7-16D-1L15</td>
<td>0.6ρ_b</td>
<td>88.51</td>
<td>98.42</td>
<td>1.45</td>
<td>1.61</td>
<td>1.11</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

جدول 2: مقایسه بین نتایج اجزاء محدود و روابط محاسبه شده در مرجع [1]

<table>
<thead>
<tr>
<th>نام نمونه</th>
<th>ρ</th>
<th>P_con (kN)</th>
<th>P_Yield (kN)</th>
<th>P_FEA (kN)</th>
<th>P_Yield_con (kN)</th>
<th>P_Yield_FEA (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>61-Fa</td>
<td>0.684ρ_b</td>
<td>18.9</td>
<td>20.9</td>
<td>27.1</td>
<td>1.1</td>
<td>1.43</td>
</tr>
<tr>
<td>61-Fb</td>
<td>0.684ρ_b</td>
<td>18.9</td>
<td>20.9</td>
<td>27.1</td>
<td>1.1</td>
<td>1.43</td>
</tr>
<tr>
<td>91-Fa</td>
<td>0.47ρ_b</td>
<td>20.6</td>
<td>24.5</td>
<td>27.3</td>
<td>1.18</td>
<td>1.32</td>
</tr>
<tr>
<td>91-Fb</td>
<td>0.47ρ_b</td>
<td>20.6</td>
<td>24.5</td>
<td>27.3</td>
<td>1.18</td>
<td>1.32</td>
</tr>
</tbody>
</table>

۶- نتیجه‌گیری

در این تحقیق اثر ورقا و نوارهای FRP در تقویت خمشی تیرهای بتن مسلح با استفاده از تحلیل‌های عددی و مدل آماده‌کردن فلزات پلاستیک بتن بررسی و براساس مقایسه بین مقادیر بدست آمده از تحلیل‌های عددی، آزمایشگاهی و آن به ناحیه ای نتایج زیر بدست آمد:

- تحلیل عددی انجام شده با استفاده از مدل آماده‌کردن فلزات پلاستیک با استفاده از ترم افزایش اجزاء محصور قادر به پیش بینی منحنی بار – تغییر مکان، بار شکست و حالت شکست تیرهای تقویت ABAQUS در می‌باشد.
- شکست بین موثرترین پارامتر در بین شکست‌های ناشی از جداسازی ورقه‌ای از CFRP سطح بدل بار ترک خودگی کشنده می‌باشد.
32

between conferences, workshops, and seminars.

• Conclusion


13. ACI 318-05. Building code requirements for structural concrete (318M-05) and commentary (318RM-05). Farmington Hills (Michigan, USA): American Concrete Institute (ACI); 2005.
بررسی تأثیر دمای اولیه بتن بر میزان جذب آب و جذب آب مویشیه آن

چکیده
شرایط ساخت بتن تأثیر بیزیری آن را از شرایط اقیمت‌خود به خصوص دما اجتناب تاپذیر نموده است و از آنجا که دمای محیط همواره تابع زمان و مکان می‌باشد، لذا بررسی آن به عنوان یک پارامتر متنوع در بتن و آثار تغییرات ان‌حائز اهمیت است.

در این مقاله این اثر بر خصوصیات جنگلی بتن که مالک مهمی از دوام بتن یا می‌باشد مورد بررسی قرار گرفته است. در این راستا نمونه‌هایی با دماهای اولیه 10، 20 و 40 درجه سانتی‌گراد ساخته شد و در شرایط آزمایش‌گاهی و کارگاهی عمل کرد. مشاهده شد که در سیسا 28 روز بتن با دمای اولیه 30 درجه سانتی‌گراد جذب اولیه نهایی را در بین سایر نمونه‌ها دارا بوده و با افزایش دما از 30 درجه سانتی‌گراد ها بتن افزایش می‌یابد. در جذب مویشیه نیز نتایج مشابهی احراز شد و با افزایش دمای اولیه بتن از 30 به 40 درجه سانتی‌گراد یافته.

کلیدواژه‌ها: دمای اولیه، بتن تازه، جذب آب اولیه، جذب آب به نهایی، جذب آب مویشیه

1 - مقدمه

یکی از اهمیت‌های ساختمانی برصرف در ساخت بتن ساختمان‌ها سکوک، جوانو و همینطور در سازه‌های زیرزمینی مثل تونل‌ها و خطوط لوله‌های بتنی می‌باشد. در حالی که، بتن یک از مصالح بسیار بادوام به شمار می‌آید که می‌تواند در شرایط محیطی بسیار سخت مثل شرایط محیطی دریایی، صنعتی و غیره، مقاومت خوبی را از خود به نمایش بگذارد. این نکته، از سیمان، آب و مواد افزودنی‌های مختلف همگی از مواد تشکیل دهنده بتن هستند که میزان نوع و شرایط آنها همگی در کیفیت بتن تأثیر می‌گذارند و این اثر این مصالح وابسته را در کنار یکدیگر می‌چسبانند. این مقاله، به تبع حضور سیمان در بتن صورت گرفته و مقاومت‌های بتن‌های تعریف چه از نظر مکانیکی و چه دوامی کمتر از مقاومت‌های سخت‌گیرانه انتظار می‌دارد. بنابراین، این محصولات واکنش‌های سیمان، به عنوان ضعیف ترین بخش در بتن، شناخته می‌شوند و لذا این واکنش‌ها و پارامترهای تأثیر گذار در آنها از اهمیت بسزایی برخوردار است که تاکنون مطالعات فراوانی در این زمینه انجام گرفته است.
یکی از این پارامترها تاثیر گذار دلیل بیش از اندازه افزایش نسبت کننده وابستگی آب و درون بیشتر این تاثیر گذار بوده و تاثیرات خاصی بر ویژگی‌های بین گذراند این تاثیرات چه در دوره ساخت بین تازه، چه در دوره عمل اویو و چه در دوره بیرون بردازی از تغییرات نتیجه‌گیری شد.

به نظر می‌رسد بلکه تاثیرات زیادی که دلیل بین تازه با توجه به روش ساخت آن از دامی محيط می‌گردد و همچنین تغییراتی که در دمای مناسب در طی فصول، ماه‌ها، روزها و حتی سایر روز مشاهده می‌گردد، دمای (خلاص و یا خشک کردن و یا یکپاره دردسری از آن) روی نسبت به دمای محيط اطراف آن می‌یابد. ضمن آنکه این دمای تازه به نسبت واکنش‌های هیدراسین تیر افزایش می‌یابد که این بیده به خصوص در مورد بینهای حجم

مقداد قابل ملاحظه یا پایین می‌کند.

لذا بررسی اثرات دما در بین تازه بر روی خصوصیات مقاومت و دوام آن موضوع با اهمیتی است که در این تحقیق که شده است این تاثیرات در مورد هیدراسین جدی بین که پارامتر مهم در دام آن محسوب می‌شود مورد بررسی قرار گرفت.

2-کلیات

در حقیقت به انتخاب خصائص مکانیکی، کلیات اثرات نامطلوب بر دیوار در برگیرنده بیشتری می‌آیستد از میان بین تازه است. سه نوع سیال وجود دارد که عوامل بر دیوار بین انرژی گذارند و می‌توانند به داخل بین وارد شودند (خلاص و یا خشک کردن و یا یکپاره دردسری از آن). حالا این موارد می‌توانند به روش‌های مختلف در بین جابجا شوند. اما کلیه این جابجایی‌ها محدود به ساختار خیمه هیدراسین شده سیمان و ساختار شیمیایی هیدراسین است که بین سیمان و اب انجام می‌شود. ساختار هیدراسین یا به عبارت دیگر ریز ساختار شیمیایی موجود در بین است. بین این تأثیرات متفاوت است.

به طور کلی، خیمه سیمان هیدراسین شده شما و سیستم مهم است:

1- سیستم جامد
2- سیستم منافذ
3- سیستم محلول در منافذ

سیستم‌های جامد محلول در منافذ مربوط به ساختار شیمیایی و سیستم منافذ محوج به ساختار فیزیکی بین می‌باشد. سیستم جامد در واقع محصولات هیدراسین است که شامل زل. H-C-S- و فازه اولیات و فرتیت است. سیستم منافذ شعل منافذ زل است که اندوز آن‌ها به‌صورت کوچک در حداکثر 40 nm متغیر می‌شود. سیستم منافذ شعل منافذ زل است که اندوز آن‌ها به‌صورت کوچک در حداکثر 40 nm متغیر می‌شود. به علاوه منافذ دیدگی نیز وجود دارد که اندوز آن‌ها به‌طور کم‌تر از منافذ موجود است که در نتیجه تراکم ناپایداری بین ایده‌های می‌شود.

به‌عنوان نمونه، سیستم محلول بر می‌گردد. که این محلول عمداً شامل هیدروکسید سدیم NaOH
بررسی تاثیر دمای اولیه بین بر میزان جذب آب و جذب آب.../ ۲۵

هیدرولیس Cu(OH)۲، کلسیم KOH است.

کل حجم منافذ موئین موسوم به تخلخل است و معمولاً برای مقایسه کیفیت بین از مقدار تخلخل استفاده می‌شود. ارتباط بین منافذ موئین تقسیم کننده‌ای در نفوذ‌پذیری و دوام دارد. در واقع منافذی که مستقل و بدون ارتباط به یکدیگر هستند، اثری در نفوذ‌پذیری ندارند و در شکل (۱) رابطه بین ارتباط منافذ در نفوذ‌پذیری نشان داده است.

حجم منافذ موئین تابع نسبت آب به سیمان می‌باشد. بعضاً دیرگ، مقادیر اولیه خیر سیمان ورجه هیدرولیس بر حسب ویژوی می‌باشد.

ویژگی‌های منافذ موئین اتمسفری. هرچه نسبت آب به سیمان بیشترشاد حجم منافذ موئین افزایش می‌یابد.

وقت را به سطحی بین مهم‌ترین عامل تعیین کننده شدت آسیب پذیری سازه‌هاست. حجم موئین نفوذ‌پذیری بین به عنوان میزان سنجش دوام بین است. نفوذ‌پذیری کم، نشان دهنده مقاومت زیاد بین در مقابل حکمت آب است. در واقع بسیاری از پویشگری‌ها نفوذ‌پذیری را مهم‌ترین کننده مقاومت به دوام می‌دانند. این فرضیه در مورد بی‌هایی که به طور دائم غوطه ورند صحت دارد. زیرا در جنین شرایطی بین اشاعه و در معرض سیان‌ها ارتقاء آب است. اما در موارد دیگر که سازه در معرض هوا قرار دارد (مانند یلها)، فرآیند نفوذ‌پذیری صادق تیست.

در جنین شرایطی سطح دوام بین اشاعه نیوته و غیر از بخشی از سازه (که در معرض جنین و سیان‌ها) در معرض سروش آب می‌باشد، به عبارت دیگر جذب موئین به چای نفوذ‌پذیری، کتیل کننده عبور آب است.

گرمالتونه عبارت است از جذب شده بین که از طریق پذیری میشود یا افزایش مقادیر نفوذ‌پذیری موئین بین، به مقدار جذب موئینه افزوده میشود. در جذب موئینه، نیرویی که می‌تواند آب می‌شود حاصل اختلاف فشار بین دو فلز که (ناتی از آب در منافذ بین) است. بنابراین نیروهای موئین، در بین کاملاً اشاعه وجود ندارد و حکمت آب متوقف می‌گردد. اگر ۱۵ یکشش نفوذ‌پذیری باشد معمولاً ارتقاء آب (۱) معادل به دست می‌یابد.
در این پژوهش از سیمان پرتلند نوع 2 استفاده شد. همانندی این ماده با حد اکثر اندازه 7/4 و همچنین شن نیمه شکسته با حد اکثر اندازه 19 میلیمتر استفاده گردید. آب مصرفی نیز آب شهر تهران بود.

طرح اخلاط مطلوب با طرح اختلاط ملی ایران انجام گرفت. نسبت آب به سیمان برای 0/47 و عیار سیمان برای 400 کیلو گرم فرض شد.

در ادامه سیمیشن تا دماهای 100 و 300 درجه سانتیگراد در مخلوط سیمان به تازه ایجاد گردید. دمای هوا در حین ساخت بین در حدود 32 درجه سانتیگراد بود و یا ایجاد دماهای مورد نظر از فرمول پیشنهادی در ACI استفاده گردید. رابطه (1)

$$ T = \frac{1}{\pi^2 (c + G_a + S_a) + W_v} \cdot \frac{1}{\pi^2 (c T_c + G_a T_a + S_a T_s) + T_w W_m + T_a W_a + T_s W_s + W_v} \cdot \frac{1}{r (c T_c + G_a T_a + S_a T_s) + T_w W_m + T_a W_a + T_s W_s + W_v} $$

در اینجا $T$ به میزان نیروهای کاراکتریستیک سیمان می‌گوید.
درصد وزنی با استفاده از میانگین گری نتایج ۳ آزمون مکمی ۱۰۰ میلی‌متری در مورد نمونه‌های آزمایشگاهی و دو عدد میانه ۷/۵ میلی‌متری از نمونه‌های کارگاهی به دست آمد. نتایج آزمون‌های برای انجام این آزمایش به این صورت بود که پس از خارج شدن از قالب، ابتدا به صورت ۷۷ روز به داخل محیط آب منتقل شدند، سپس از آن خارج و توزین شدند. برای خشک شدن به دو روش مختلف به دو دامای ۱۱۰ درجه سانتی‌گراد انتقال یافتند. پس از ذخیره، روز و رسیدن آزمون‌ها به وزن تابیت (m) آزمایش‌مکروک با غوطه‌ور کردن آنها در آپ آغاز گردید. در ادامه با خارج ساختن آزمون‌های غوطه‌ور شده از آب در زمان‌های ۴۴، ۲۷ ساعت، ابتدا آب اضافی آنها توسط یک چرخه دستگاه شیمیایی به دست آمد که بعد از دو روز به دست آمد که بعد از ۱ روز هنگام جذب آپ نمونه‌ها بسیار اندک است و پس از ۳ روز تغییر خاصی نمی‌کند. همچنین نمونه‌ها کمتر از آپ که هنگام خارج شدن از
حوضچه دارا هستند، جذب آب می‌کنندن این آزمایش، جذب آب یک ساعت به عنوان جذب آب اولیه و جذب آب سوزه به عنوان جذب آب نهایی در نظر گرفته شده است.

\[
\frac{m_t - m_0}{m_0} \times 10 \times t
\]

جدول ۲: نتایج جذب آب اولیه (۱ ساعت به حسب \%) تا نام طرح و میزان جذب آب اولیه

<table>
<thead>
<tr>
<th>شرایط عمل اوری</th>
<th>سن ازامونه</th>
</tr>
</thead>
<tbody>
<tr>
<td>T=۴۰</td>
<td>۴/۳۷ آزمایشگاهی ۲۸ روزه</td>
</tr>
<tr>
<td>T=۳۰</td>
<td>۴/۲۸ آزمایشگاهی ۲۸ روزه</td>
</tr>
<tr>
<td>T=۲۰</td>
<td>۴/۱۵ آزمایشگاهی ۲۸ روزه</td>
</tr>
<tr>
<td>T=۱۰</td>
<td>۴/۲۳ آزمایشگاهی ۲۸ روزه</td>
</tr>
</tbody>
</table>

جدول ۳: نتایج جذب آب نهایی (۷۲ ساعت به حسب \%) تا نام طرح و میزان جذب آب نهایی

<table>
<thead>
<tr>
<th>شرایط عمل اوری</th>
<th>سن ازامونه</th>
</tr>
</thead>
<tbody>
<tr>
<td>T=۴۰</td>
<td>۹/۶۱ آزمایشگاهی ۲۸ روزه</td>
</tr>
<tr>
<td>T=۳۰</td>
<td>۹/۴۱ آزمایشگاهی ۲۸ روزه</td>
</tr>
<tr>
<td>T=۲۰</td>
<td>۹/۰۸ آزمایشگاهی ۲۸ روزه</td>
</tr>
<tr>
<td>T=۱۰</td>
<td>۹/۲۷ آزمایشگاهی ۲۸ روزه</td>
</tr>
</tbody>
</table>

جدول ۴: میزان آب جذب شده نمونه‌ها در پایان سن ۲۸ روز

<table>
<thead>
<tr>
<th>شرایط عمل اوری</th>
<th>سن ازامونه</th>
</tr>
</thead>
<tbody>
<tr>
<td>T=۴۰</td>
<td>۱۰/۳۱ آزمایشگاهی ۲۸ روزه</td>
</tr>
<tr>
<td>T=۳۰</td>
<td>۱۰/۱۵ آزمایشگاهی ۲۸ روزه</td>
</tr>
<tr>
<td>T=۲۰</td>
<td>۹/۸۶ آزمایشگاهی ۲۸ روزه</td>
</tr>
<tr>
<td>T=۱۰</td>
<td>۹/۹ آزمایشگاهی ۲۸ روزه</td>
</tr>
</tbody>
</table>

علت اختلاف میزان جذب آب با تغییرات دمای اولیه بین را می‌توان اینگونه تفسیر نمود، در بن منافذ متعادل وجود
همانطور که در مورد مقاومت تنش بان انشا شده، دمای اولیه بالا (در این آزمایش دماهای بالاتر از نمونه $T=20$) باعث افزایش سرعت هیدراسیون و تولید سریع محبوبات میگرد، ولی این سرعت بالایی هیدراسیون باعث تجمع محبوبات حول ذرات هیدرات نشد سیمان گسته و شکل آن سیمان به طور کامل هیدرات نمی‌گردد. این محبوبات فرصت کافی جهت توزیع یکنواخت بین ذرات بین را داشته و هدایت باعث ایجاد ضعف تر و نفوذ نیز در بنگرگردد( ضمن آنکه در بنگ ممکن است نقاط ضعیف موشی بنگ وجود داشته باشد). همچنین در دماهای اولیه بالا (مثل نمونه $T=10$) درجه باینی هیدراسیون سیمان باعث افزایش جذب آب نسبت به دماهای بالا (مثل نمونه $T=20$) است.

اختلاف نفوذ‌پذیری در نمونه‌های آزمایشگاهی و کارگاهی را می‌توان در شرایط مناسب تر عمل آوری نمونه‌های آزمایشگاهی دانست زیرا در معرض آب قرار داشتن این نمونه‌ها باعث ایجاد هیدراسیون کامل تن و بافت بهترینست به نمونه‌های کارگاهی گمشته است.

در پنج این اختلاف کمتر از آنچه بیش از پنین می‌شود واقع گشت که شاید نتانی این پیده را ناشی از دمای بالا در اوری نیست به نمونه‌های آزمایشگاهی بالا وارد است.

همچنین در این آزمایش بدیع خروج نمونه‌ها در پایان 28 روز عمل آوری آزمایشگاهی و زن، انشاب آنها انداره گردید و سپس جهت انجام آزمایش جذب آب در از ابتدای ابتدایی. ولی بعد از سه روز موارد و شرایط مجدید نمونه‌ها در آب و تابیت شدن جذب آب آنها مشاهده شد که نمونه‌ها به میزان آب از دست داده، قادیر به جذب آب نمی‌گردند و حدود 90% آب از دست داده را مجددا جذب می‌کنند. نمونه (2) این 10% آب غیر قابل جذب مجدد، آب موجود در منطقه زلی و یا منطقه غیر مرتفع موجود در محبوبات هیدراسیون می‌شود که پس از نفوذ‌پذیری شدن آنها، آب قادر به نفوذ مجدد در آن نیست.

همچنین مشاهده شد که روندی بین درصد آب جذب شده نمونه‌ها در پایان سی 28 روز و درصد جذب آب آنها پس از سه روز بسیار می‌باشد (نمودار(1))

نمودار 1- تغییرات میزان جذب آب اولیه (1 ساعت) با تغییرات دما
جذب آب مویینه

ازمایش جذب آب مویینه طبق دستور RILEM CPC 11.2، TC 14-CPC انجام گردید. این ازمایش فقط در مورد نمونه‌های آزمایشگاهی ۱۰×۱۰ و ۲ نمونه از هر مخلوط انجام گرفت. نحوه عمل آوری و تشکیل نمونه‌ها در گرمخانه مطابق با ازمایش تعبیه جذب آب بود. در این ازمایش، آزمایش‌های خشک شده در داخل نظر گرفته وی از طرف گرفته گرفته که کف آن اندکی بالاتر از کف طرف باند برای این منظور نمونه‌ها بر روی تخته‌های باریک چوبی قرار داده شد و سپس طرف تا ارتفاع ۵ میلی‌متر بالاتر از کف آزمایش‌ها بر از گرفت. در تمام مدت آزمایش سطح آب ثابت نگاه داشته شد.

اثدارهای جذب آب مویینه در فواصل زمانی ۲۴ و ۷۲ ساعت از زمان قرار دادن آزمایش‌ها در آب صورت گرفت. هنگام توزین، آزمایش‌ها به‌ترتیب از درون آب خارج شد به همکاری پانه سطحی آن خشک شد، نمونه‌ها وزن شد و مجدد داخل نظر گرفته گرفته، به این ترتیب با داشتن وزن آزمایش خشک شده در گرمخانه و (m۰) وزن آزمون‌های زمان باز در واحد سطح یا عمق مطالعه تهیه آب از حسابه شد. (۳)
بررسی تأثیر دما

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\[ i_t = \frac{m_t - m_0}{A} \]

\[ (3) \]

که در آن:

[mm] \( i_t \)

[gr/mm²] \( t \)

[gr] \( m_0 \)

[gr] \( m_t \)

[mm²] \( A \)

با ترسیم نقاط به دست آمده در دستگاه \( S \) و \( C \) در نقطه یک خط مستقیم از این نقاط، ضریب جذب آب مویینه \( S \) (شیب خط برآورش شده) و ثابت جذب آب مویینه \( C \) (عرض از مبدأ خط برآورش شده) به دست آمد. رابطه عمومی خیاب برآورش شده به شکل زیر است:

\[ i = C + S \sqrt{t} \]

\[ (4) \]

که در آن:

[cm] \( i \)

[cm²] \( S \)

[cm/hr⁰.⁵] \( C \)

[hr] \( t \)

در روش دیگر میزان خیاب (ت) را نیز رسم نمود و \( S,C \) را با برآورش خیاب در این دستگاه مختصات بدست آورد.

\[ i = C + S \log(t) \]

\[ (5) \]

از دستگاه مختصات انتخاب می‌گردد که خیاب برآورش داشته باشد در آن دستگاه بهترین ضریب همبستگی \( C \) و \( S \) را برای نقاط در آن دستگاه ایجاد کند.
نتایج جذب موئینه آزمون‌ها در جدول (5) مشاهده می‌شود. ضرایب همبستگی (R) بین 0.961 و 0.987 که با طور میانگین 976 می‌باشد. ضرایب همبستگی بین آمده مناسب به‌ودی و باکتری‌ها گزینی قابل قبول نقطه نمودار بر حسب جنس زمان می‌باشد.

اگر نمودار بر حسب لگاریتم زمان ترکیب شود، ضرایب همبستگی بین مقدار 961/0 و 972/0 به طور میانگین 976/0/خواهند بود که در مقایسه با روش چند زمان همبستگی ضعیف تری بین نقطه برقرار خواهد بود.

(جدول (۶))

بنابر این مقادیر تابث جذب و ضرایب جذب موئینه نمودار جنس زمان مدت نظر خواهد بود.

ضرایب جذب موئینه (S) در حالت‌های مختلف ارائه شده و مقادیر ضرایب از T=10 0/43 0/57 0/99 1/40 0/218 0/134 T=20 0/47 0/62 1/05 1/38 0/291 0/134 T=30 0/52 0/71 1/22 1/57 0/326 0/154 T=40 0/48 0/63 1/38 1/54 0/249 0/156

(جدول ۵) نتایج آزمایش جذب آب موئینه

<table>
<thead>
<tr>
<th>ضرایب</th>
<th>جذب موئینه [cm/hr0.5]</th>
<th>جذب آب در واحد مساحت برای با عمق</th>
<th>نام طرح</th>
<th>ساعت</th>
<th>ساعت</th>
<th>ساعت</th>
<th>ساعت</th>
<th>ساعت</th>
<th>ساعت</th>
<th>T=10</th>
<th>T=20</th>
<th>T=30</th>
<th>T=40</th>
</tr>
</thead>
<tbody>
<tr>
<td>0/987</td>
<td>0/143</td>
<td>0/218</td>
<td>1/40</td>
<td>0/99</td>
<td>0/57</td>
<td>0/43</td>
<td>T=10</td>
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<td></td>
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<tr>
<td>0/971</td>
<td>0/134</td>
<td>0/291</td>
<td>1/38</td>
<td>1/05</td>
<td>0/62</td>
<td>0/47</td>
<td>T=20</td>
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</tr>
<tr>
<td>0/961</td>
<td>0/154</td>
<td>0/326</td>
<td>1/57</td>
<td>1/22</td>
<td>0/71</td>
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<tr>
<td>0/966</td>
<td>0/156</td>
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<td>0/63</td>
<td>0/48</td>
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</table>

(جدول ۶) مشخصات خط برآذری شده

<table>
<thead>
<tr>
<th>ضرایب</th>
<th>جذب موئینه [cm/hr0.5]</th>
<th>جذب آب در واحد مساحت برای با عمق</th>
<th>نام طرح</th>
<th>ساعت</th>
<th>ساعت</th>
<th>ساعت</th>
<th>ساعت</th>
<th>ساعت</th>
<th>ساعت</th>
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هر چند مقادیر تابث جذب موئینه به نوعی نشان دهنده تابث جذب آب موئینه بنا گشته است. اما با توجه نمودار نشان می‌دهد که در بحث جذب آب موئینه، اهمیت جذب آب موئینه نسبت به تابث جذب موئینه مهم‌تر به‌شمار می‌رود و بررسی تابث جذب آب موئینه مهم‌تر به‌شمار می‌رود. اما با توجه به تکرار تابث جذب آب موئینه، روند جذب آب موئینه بنا یاد بررسی شود. لذا "ضرایب جذب آب
موزه‌های سیانی

نتایج گیری

1- درجه و سرعت هیدراسیون سیمان عمومی تأثیر گذار بر میزان جذب آب یافت. این تاثیر در پایان بهبود و تقویت دیگر کمترین جذب آب را دارا یادم است. در حالی که تا حدود ۹۰٪ با کاهش دما در جذب آب از نوع آب روند در هر دو سری نمونه‌ها آزمایش‌گاهی و چالش پیکان بهبود، در پایان آزمایش جذب آب مجددی یافت می‌گردد.

2- در این آزمایش مشاهده شد که نمونه‌ها حدود ۹۰٪ را گزارش کرده‌اند. نتایج آزمایش و بیش‌تری بین نمونه‌ها در برابر اندازه‌گیری بریش بهبود نشان داد.

3- روند بین تأثیر میزان آب جذب شده نمونه‌ها در هنگام خروج از حوضچه‌های پس از ۸۸ روز دارای آنها در مداه‌های مختلف، پیکان است.

4- روند تغییرات ضریب جذب آب مویینه در این آزمایش نشان می‌دهد که بین T=۲۰ درجه و S=۴۳ آزمایش جذب آب و جذب آب مویینه در این آزمایش کم و پیکان می‌باشد.

5- پیشنهاد می‌شود این آزمایشات در سنین بالاتر نیز انجام گیرد تا ادامه این روند در سنین به اندازه‌گیری بالا به‌کار گیرند.
دفتر تحقیقات و میزان فنی، سازمان برنامه و پژوهش، شماره نشریه: 438.

۲- (۱۳۸۵) "خواص بنیان". ترجمه: فصلی، ه، ابوبکر برومنی، رمضان‌پور، ع، پاسداشتی، رساله "گرداشته‌های بنایی در سازه‌های بتن مسلح". مرکز تحقیقات ساختمان و مسکن، کد ۲۳۹-۱۳۷۰، شماره نشریه: ۲۳۷.

۳- (۱۳۸۵) "خواص بنیان". ترجمه: فصلی، ه، ابوبکر برومنی، رمضان‌پور، ع، پاسداشتی، رساله "گرداشته‌های بنایی در سازه‌های بتن مسلح". مرکز تحقیقات ساختمان و مسکن، کد ۲۳۹-۱۳۷۰، شماره نشریه: ۲۳۷.

۴- (۱۳۸۵) "خواص بنیان". ترجمه: فصلی، ه، ابوبکر برومنی، رمضان‌پور، ع، پاسداشتی، رساله "گرداشته‌های بنایی در سازه‌های بتن مسلح". مرکز تحقیقات ساختمان و مسکن، کد ۲۳۹-۱۳۷۰، شماره نشریه: ۲۳۷.

۵- (۱۳۸۵) "خواص بنیان". ترجمه: فصلی، ه، ابوبکر برومنی، رمضان‌پور، ع، پاسداشتی، رساله "گرداشته‌های بنایی در سازه‌های بتن مسلح". مرکز تحقیقات ساختمان و مسکن، کد ۲۳۹-۱۳۷۰، شماره نشریه: ۲۳۷.

۶- (۱۳۸۵) "خواص بنیان". ترجمه: فصلی، ه، ابوبکر برومنی، رمضان‌پور، ع، پاسداشتی، رساله "گرداشته‌های بنایی در سازه‌های بتن مسلح". مرکز تحقیقات ساختمان و مسکن، کد ۲۳۷.


تاثیر دماهای بین تازه بر مقاومت‌های مکاتبی و الکتریکی آن در سنین مختلف

چکیده

از آنجا که عمدتاً ساخت تن در شرایطی در نمایاندن به محیط اندازه‌گیری دیگر اغلب محدود می‌گردد، ایجاد مسند مطالعه در این مقاله تاثیرات دماهای بین در مقاومت‌های فشاری، کششی و همچنین مقاومت ویژه الکتریکی آن مورد بررسی قرار گرفت. در این راستا نمونه‌هایی با دماهای اولیه ۱۰ و ۲۰ درجه سانتی‌گراد ساخته و مشاهده گشت که با افزایش دماهای بین مقاومت گذشته کوتاه مدت آن قسمت‌های افزایش می‌یابد ولی مقاومت‌ها دراز مدت آن به خصوص در سن ۳۰ روز کاهش یافته است.

همچنین از میزان کشش برزیلی بر روی این نمونه‌ها انجام شد و مشاهده شد که تا سال ۲۸ روز نمونه‌های با دماهای اولیه ۱۲ درجه سانتی‌گراد دیگران مقاومت کششی را دارا می‌باشند و با افزایش دماهای اولیه مقاومت کاهش می‌یابد. در مورد مقاومت ویژه الکتریکی نیز نتایج مشابه مشاهده گشت که این نتایج از سن ۹۹ روز مقدار اختلاف مقاومت‌های قابل توجهی بین کرد و بین با دماهای اولیه ۱۰ درجه سانتی‌گراد بین‌ترین مقدار مقاومت ویژه الکتریکی را دارا بود.

کلیدواژه‌ها: دماهای اولیه، بین تازه، مقاومت فشاری، مقاومت کششی، مقاومت ویژه الکتریکی

۱. مقدمه

بتن یکی از مصالح ساختمانی پرطرف‌سر در ساخت بناها است. ساختمان‌ها، سکوها، جدول و همین‌طور در سازه‌های زیرزمینی مثل تونل‌ها و خطوط لوله‌های بینی می‌باشند. در حالت کلی، بتن یکی از مصالح سیاسی بادی به شمار می‌آید که مناسب در شرایط محیطی بسیار دارای ویژگی‌های جدی در صنعت، غیره مقاومت خوبی را از خود به نمایش بگذارد. این اتفاق به سیمان، آب و سایر مواد ویژه می‌تواند شکل‌دهنده بتن حسند که میزان، نوع و شرایط آنها همکاری در کیفیت بتن تاثیر گذار است. ولی آنچه این مصالح اولیه را در کنار یکدیگر همچون جسم همکار میده و اکتشافی است که به تبع حضور سیمان در بین صورت گرفته و محصولات آن ضمن ایجاد چسبندگی لازم بین مصالح از مقاومت‌های بر خوردار است که این مقاومت در
یتیهای منعطفه چه از نظر مکانیکی و چه دوامی کمتر از مقاومت‌های سنگداته است. بنابراین محبوبیت واکنش‌های مسیر سمیا، به‌عنوان ضعیف‌ترین بخش در بین شناخته گم‌شوران و لذا این واکنش‌ها و باران‌های تأثیرگذار در آنها از اهمیت زیادی برخوردار است که تاکنون مطالعات قرارا در این زمینه انجام گرفته است. 

یکی از این باران‌های تأثیر گذار دما بین می‌باشد. از آنکه که واکنش هیدرولیس سیمان یک واکنش شیمیایی نسبت کمی باشد درجه حرارت بین در هر مرحله از امر در روند این واکنش تأثیر گذار بوده و تاثیرات خاصی بر ویژگی‌های سیمان تأثیرات چه در دوره ساخت‌بندی نازه، چه در دوره عمل آری و چه در دوره بهره برداری آن در خواص بین تأثیر گذر است.

به نظر می‌رسد به دلیل تاثیرات زیادی که دما بین تازه با توجه به روش ساخت آن از دمای محیط می‌گردد و همچنین تغییراتی که در دمای محیط در طی فصول، ماه‌ها، روزها و حتی ساعت‌های روز مشاهده می‌گردد، دمای اولیه بین یک بارت‌سرتر تا تابیر ذیپر نسبت به دمای محیط اطراف آن می‌باشد که پیوسته با تغییراتی آن در حال تغییر است. ضمن آنکه اگر در بین تازه به سبب واکنش‌های هیدرولیس یک افزایش می‌یابد که این پدیده به خصوص در مورد بینهای حجم مقدار قابل ملاحظه‌ای پدیدا می‌کند.

لذا بررسی اثرات دما در بین تازه بر روی خصوصیات مقاومت و دوام‌های مفید پرداخته به دست آمده است که که در این تحقیق نسبت بین تازه است این تاثیرات در مورد مقاومت فشاری، کششی و مقاومت ویژه الکتریکی بین مورد بررسی قرار گرفد.

2-کلیات

1- اثرات دما بر روی خواص بین تازه

عواملی افزایش دما در بین سبب افت رویان و کاهش پشت اسلام‌در در بین تازه می‌گردد و نتیجه‌کارکرده یک متغیر در بین ایده‌ها می‌گردد و تخیلی بیشتر و جداسازی بیشتر بیشتر بیشتر از رخخت بین به قابل ممکن است اتفاق بیافتد. لذا ممکن است در کارکرده‌ها باره یک در را افزایش می‌دهد که در تحقیق افزایش نسبت بین به سیمان، حجم بیشتر حجار و تنها کاهش در اثبات هر دو افزایش بین نیز باید یک افزایش حجم سیمان نسبت به سیمان بین از نظر مقاومت دوم ضایع حجم می‌گردد. که در دیگر مشاهده خود، افزایش می‌گردد (تغییر می‌کنند سیمان بیشتر خود دمای هیدرولیس را از این طریق نمی‌باید).

در 305 مورد که مربوط به بین رزی در هواه گرم است؛ عواقب افزایش دما در بین تازه را این‌گونه بررسیده

است:

- 1
  - نیاز به آب بیشتر در بین
  - افزایش کاهش اسلام
- 2
  - گیرش سرعتی در تئیه جابجا، تراکم، پرداخت خشک‌تر و رسک بالاتر ایجاد در درجه‌های سرد
- 3
  - استفاده بیشتر در تشکیل تکه‌های بلاستیک حاشیه از جمع ششی
- 4
  - سختی بیشتر در کنترل هواه محسوس شده در بین (هندگی که از موانع حجاب را استفاده می‌گردد)
مقاومت بتن در بسیاری از موارد به عنوان یک ازاری نسبی در نظر گرفته می‌شود. گو اینکه در بسیاری از موارد عملی ممکن است سایر مشخصاتی از آن مانند دوام و نفوذپذیری اهمیت بیشتری داشته باشد.

مقاومت یک تصویر کلی از کیفیت بتن به دست می‌دهد و با استفاده از خمیر هیدرات شده سیمان رابطه مستقیم دارد. با توجه به ماهیت واکنش‌های هیدرولیک سیمان، دما عاملی تاثیرگذار بر مقاومت بتن در کنار عوامل دیگری مانند ذرات قرار، نسبت آب به سیمان، نوع سیمان، جهت سیمان، نگهداری آنها، آب اختلاف، موارد افزودنی، شرایط عمل آوری، شکل و هندسه نمونه، رطوبت نمونه و... می‌باشد.

تأثیر دما در دوران عمل آوری بر مقاومت بتن توسط پژوهشگران زیادی مورد بررسی قرار گرفته و آنچه از نتایج آنها می‌توان استنباط کرد آنست که افزایش دما در دوران عمل آوری بتن چنین مهم با حفظ رطوبت باشد. باعث افزایش انگکس می‌گردد. مقاوت آن تا سن 28 روز می‌گردد و در سنین بالاتر مقاومت‌های نهایی بتن را کاهش می‌دهد. همچنین از آنجا که درجه هیدرولیک سیمان تابعی از زمان و دمای مقاومت بتن ممکن است بر منابع بلوغ، که به صورت تابعی از زمان و دمای عمل آوری می‌باشد، بررسی گردد.

**شکل 1** - تأثیر دماهای عمل آوری بر مقاومت فشاری در عمرها 1 و 28 روز

**۲.۳- فشار بتن تحت فشار تک محوری**

منحنی تنش کرنش حداکثر ۲۰٪ مقاومت نهایی (f'_c) و تنش خیزی و ارتجاعی نشان می‌دهد و این بدين دیل است که تحت بارگذاری کرنش حداکثر ریز ترکا در ناحیه انفعال و از دو هم نمی‌خورند. در تنش‌های بالاتر از این نقطه منحنی تا دو بینهایت حدود ۲۵۰/۰/۹/۵/۰ تا c'/f'_c این احتیاج بینهایت بیضا می‌کند و نسبت خمیدگی تندی پیدا کرده و سرانجام نزول پیدا کرده تا آنکه نمونه به گسیختگی برسد.
مقادیم ۱۴-۲ مقادیم کششی بین روش فشاری و کشش بین با یکدیگر در ازبین مستقیم هستند و معمولاً نسبت مقادیم کششی به فشاری در محدوده ۰/۰۳ تا ۱/۱ است. روابط زیادی برای ارتباط این دو مقادیم مطرح شده است و یکی از آنها آزمایش شده و در آزمایشات مشخصات سختگانه، که فنی و نحوه عمل آوری آن و مواد موادی هر یک به ناحیه در این مقادیم و تغییرات آنها باعث گزار است که هیچ رابطه‌ای را نمی‌توان در این مورد تعیین کلی داشته باشیم. وجود جواب‌هایی نیز افزایش مقادیم فشاری نسبت به مقادیم کششی در این مورد می‌کند. تلاطم ناقض نیز افزایشی در مقادیم کششی ایجاد می‌کند.

روش مستقیم آزمایش مقادیم کششی بسیار انگیزه‌بخش و این امر در مدل‌های زیادی در این مورد است. متفاوت‌ترین روش‌های این‌ها به‌منظور مقادیم کششی بین روش و نیمه (ASTM-C78) و کششی ناشی از خم شدن سطحی (ASTM-C469) می‌باشد.

- آزمایش دوم نیم‌شن: در این آزمایش استاندارد ۱۵۰۰ سانتی‌متری در امتداد قطرش به صورت خواهید. تحت فشار قرار گرفته و به طور پیوسته و با سرعت ثابت در محدوده کششی بین و بین ۱۴ تا ۷ کیلوگرم در سانتی‌متر مربع ایجاد شود. نشان نشان و سبب ایجاد کششی یک‌واختی در راستای عمود بر امتداد قطر قائم می‌شود مقادیم کششی دو نیمه شدن از رابطه

\[ T = \frac{2P}{\pi LD} \]  

(۱)

بندیست می‌آید که در آن \( T \) تنش کششی بار شکست L طول نمونه و D قطر نمونه می‌باشد. در مقادیر با تابع مقادیم کششی حاصل از روش مستقیم، مقادیم کششی حاصل از روش دو نیمه شدن بین ۱۰ تا ۱۵ درصد بیشتر است. (شکل ۱۳)
مقامت الکتریکی بتن یا میزان مقاومت الکتریکی بتن پارامتر سیاسی مهمی است که به شکل غیر مستقیم، شدت فراورده نفود بذیری و خوردن بتن را مورد ارزیابی قرار می‌دهد. در مقایسه با بتن با مقاومت الکتریکی کم، که در آن جریان به آسانی به‌نواحی آندی و کانالی عبور می‌کند، در بتن با مقاومت الکتریکی بالا، فراورده نفود بذیری و خوردن کند خواهد بود.

مقاومت الکتریکی بتن به طور کلی برای ارزیابی غیر مستقیم ویژگی‌های بتن همانند نفود بذیری آن و میزان خلل و فرج مرتبتاً با یکدیگر مورد استفاده قرار می‌گیرد.

این آزمایش به‌این ترتیب از آب یا اشباع از محلول آب نمک انجام گیرد و تندیج بتن غیر اشباع یا خشک به منابع بالاتر از آن اشباع خواهد بود.

مقاومت الکتریکی بتن تابع پارامترهایی مانند شرایط است.

1- ساختار فیزیکی بتن: با افزایش تخلخل (به ویژه متفاوت برگ) از مقاومت الکتریکی کاسته می‌شود.

2- ساختار شیمیایی بتن: وجود عناصر شیمیایی در منافذ بتن با مقاومت الکتریکی اثر دارد. مثلاً وجود کلر در محلول منافذ باعث کاهش مقاومت الکتریکی می‌گردد. همچنین در محلول منافذ بتن پون‌هایی از قیبل که مانند به نوع و مقدار آنها اثر مهمی بر مقادیر مقاومت الکتریکی K+، Si4+ و Ca2+ دارد.

3- رطوبت: افزایش رطوبت در بتن از مقاومت الکتریکی بتن کاهش می‌یابد.

4- پلاریزاسیون ذرات: همجاری بادن دو نقطه‌ای موجود در بتن و جریان، باعث کاهش مقاومت الکتریکی بتن می‌گردد.

5- درجه هیدراسیون سیمان: در خلال چند ساعت اولیه پس از محلول ترمو مقداری مقاله روزه بتن خمی کردن می‌تواند سیمان با سرعت زیاد می‌شود و از آن پس با روند کمتری افزایش می‌یابد. این مقاله به ترتیب می‌گردد. اگر آنها بتن یا بتن شرکت شود. خشک بتن مقدار مقاومت ویژه را افزایش می‌دهد.

ظرفیت خاکی بتن با عمر آن و افزایش فرکانس کاهش می‌یابد، خمر سیمان خالص با نسبت آب به سیمان 0/0 و با عمر بسیار است.
6- عباره سیمان: هر تغییر در حجم نسبی خمیر سیمان مقاومت الکتریکی بین را تحت تأثیر قرار می‌دهد.

مشاهده شده است که که در یک نسبت آب به سیمان ثابت، افزایش عیار سیمان در مخلوط باعث کاهش مقاومت ویژه الکتریکی آن می‌گردد. علت عده‌ای پیده‌ای است که به شدت که در بین برابر

عبور جریان نسبت داده شده است.

7- نسبت آب به سیمان: جریان الکتریکی در داخل بتن مرطوب اساساً به وسیله عمل الکتروپلیژنی هدایت می‌شود یعنی با چنین جریان الکتریکی از میان زل صورت می‌گیرد. هر افزایش در حجم آب و در تمرکز پونه‌های موجود در آب منفی، مقاومت ویژه خمیر سیمان می‌شهد و در مواقع مقاومت ویژه به شدت با افزایش در نسبت آب به سیمان کاهش می‌یابد.

8- نوع سیمان و مصالح: نوع سیمان در رسانای الکتریکی بین تأثیر گذار است سیمان‌های پر آلومینیوم مقاومت الکتریکی بالاتری در بین نسبت به سیمان‌های پرغلت معمولی ایجاد می‌کند.

هداهنگ الکتریکی سطح‌های نیز در سیمان‌های بین تأثیر گذار است هر چه سیمان‌های سطح‌های بیشتر باشد، بین مقاومت الکتریکی کمتری خواهد داشت.

9- افزودنی‌های عموماً افزودنی‌های شیمیایی مقاومت ویژه بین را کاهش نمی‌دهند ولیکن می‌توان موارد متفاوتی ویژه را به منظور تغيیر مقاومت ویژه به کار پرداخت. افزودنی‌های پوزولانی مانند خاکستر بادی و دوده سیلیسی در افزایش مقاومت الکتریکی چشمگیری می‌آید. مقاومت الکتریکی بین‌ها معمولی حاوی 10 درصد میکروسیلیس چاپیریز شده بیش از 3 برای بین معمولی بدون میکروسیلیس دست آمده است.

10- مدار نمونه آزمایش: افزایش دمای بین در هنگام آزمایش، مقاومت الکتریکی با افزایش می‌دهد.

11- تأثیر ولتاژ و فرکانس جریان: مقاومت ویژه بین با افزایش ولتاژ و فرکانس زیاد می‌شود در کنار پارامترهای فوق به هنگام آزمایش اولیه بین نیز از طریق تاثیر آن به ساختمان خمیر هیدراته شده سیمان می‌گذارد بر مقاومت الکتریکی بین تأثیر گذار باشد.

مقایسه ویژه الکتریکی

انچه که در بحث مقاومت الکتریکی بین حائز اهمیت است مقایسه ویژه الکتریکی بین می‌باشد زیرا مقاومت ویژه الکتریکی وابسته به ماده می‌باشد و به شکل و ابعاد آن بستگی ندارد. مقایسه ویژه الکتریکی از رابطه (1)

$R = \frac{P L}{A}$

مقایسه الکتریکی اهمیت ($\Omega$)

$\frac{R}{\Omega \cdot m}$

مقاومت ویژه الکتریکی ($\Omega$)

$P$ طول نمونه (m)

$\frac{L}{\Omega \cdot m^2}$

سطح مقطع تهیه‌شب
۳- فرضیات اولیه آزمایش
در این پژوهش از سیمان پرتلند نوع ۲ استفاده شد. همچنین از ماسه شکسته با حد اکثر اندازه ۱۵/۷۵ و همچنین
شین نیمه شکسته با حد اکثر اندازه ۱۹ میلیمتر استفاده گردید. ایب مصرفی نیز اب شهر تهران بود.
طرح اختلاف مطلق با طرح اختلاف ملی ایران انجام گرفت. نسبت آب به سیمان برابر ۴۰/۰ و عیار سیمان برابر
۴۰۰ کیلوگرم فرض شد.
در ادامه سعی شد تا دماهای ۱۰، ۳۰ و ۴۰ درجه سانتی‌گراد در مخلوط تا نهایت ایجاد گردید. دماهای هوا در حین
ساخت بین ۲۰ تا حدود ۲۵ درجه سانتی‌گراد بود لذا ایجاد دماهای مورد نظر از فرمول پیشنهادی در

\[
T = \frac{0/22(c T_c + G_d T_G + S_d T_s) + T_w W_m + T_G W_G + T_s W_s)}{0/22(c + G_d + S_d) + W_i}
\]

که در آن \(c, G_d, S_d, T_s, T_G, T_c, T_i, W_s, W_G, W_m\) به ترتیب وزن سیمان، شین در حالت خشک و ماسه در حالت خشک و
به ترتیب میزان آب کل، آب مصرفی، رطوبت شین، رطوبت ماسه و آب به ترتیب
دماهای سیمان، شین و ماسه است.
و چنینچه در شرایط خاص برای خشک کردن یک بیابی به خشکی از آب از مدت استفاده شود اگه عبارت

\[
W_i = \frac{(W_m - W_i)T_w + W_i}{T_w W_m}
\]

به ترتیب آب مصرفی و جرم بیاب و \(T_i, T_w, W_m\) به ترتیب دماهای آب مصرفی و دماهای بیابی نیز.
از انجا که دقتی در روش انجام داده نمی‌باشد در نتیجه این مطالعه می‌باشد. برای این منظور با یک گیاهانی در
روش ایجاد دماهای مختلف در مختل‌های دماهای مختلف در حالت تا نهایت دماهای آب‌پاشا. برای این منظور با یک گیاهانی در
دو روش طبیعی حجم آب و بیاب به‌بین دست آمده به‌عنوان این روشات یا برای ایجاد انگکی کمتر از دماهای مطلوب ما
لحاظ کردد تا نهایت دماهای مطلوب در بین احراز گردید، نتایج در جدول ۱ دیده می‌شود.

جدول ۱: مقادیر آب و بیاب استفاده شده و دماهای آنها در هر مخلوط

| دمای بین | دمای سیمان | دمای بیاب | دمای ماسه | دمای شین | آب مصرفی | وزن | مقدار خشک | دماهای ارسال | دمای طرح | با اندازه | خشکی | خشکی |
|----------|-------------|-----------|----------|---------|------------|-----|------------|-----------|-----------|---------|-------|-------|-------|
| T=۱۰     | ۳۰          | ۳۰        | -۱۰      | ۵       | ۱۳۰        | ۲۰  | ۱۰۹       | ۱۰۹       | T=۶      | T=۶     | ۱۹۵   | ۱۹۵   |
| T=۲۰     | ۳۰          | ۳۰        | -۱۰      | ۵       | ۲۴۳        | ۲۰  | ۲۴۳       | ۲۴۳       | T=۲۰     | T=۲۰    | ۲۰۰   | ۲۰۰   |
| T=۳۰     | ۳۰          | ۳۰        | -۱۰      | ۵       | ۲۳۹        | ۲۰  | ۲۳۹       | ۲۳۹       | T=۲۰     | T=۲۰    | ۲۰۰   | ۲۰۰   |
| T=۴۰     | ۳۰          | ۳۰        | -۱۰      | ۵       | ۲۳۹        | ۲۰  | ۲۳۹       | ۲۳۹       | T=۴۰     | T=۴۰    | ۲۰۰   | ۲۰۰   |

۴- آزمایشات انجام شده
در این آزمایش میزان روانی بر بین تازه مطلوب استاندارد ASTM143 انجام گردید.
در ادامه برای تعیین مقاومت فشاری مخلوط‌های ساختمانی از آزمون‌های مکعبی 100 و 150 میلی‌متری نمونه‌های عمل آمده آزمایشگاهی و همچنین مزه‌های به قطر 5/7 میلی‌متری تپه شده از نمونه‌های عمل آمده در شرایط کارگاهی براساس استاندارد ASTM C617-94 و BS1881 استفاده گردید.

آزمایش مقاومت کششی مطابق با استاندارد ASTM-C469 به صورت دو نمونه گردن و ترکاندن نمونه‌های استوانه‌ای 30×15 سانتی‌متر و مزه‌های 7/5 میلی‌متری کارگاهی در اثر آب خلال خاک در امتداد طول نمونه انجام می‌شود(آزمایش کشش بزرگی). سن آزمایش در مورد هر دو سری از نمونه‌ها 28 روز است.

4-1- آزمایش تعیین روتوئی بتن تازه پس از ساخت هر مخلوط آزمون اسلامی انجام گردید و مشاهده گشت با افزایش دمای اولیه بتن تازه به دلیل افزایش سرعت هیدرالوسی و گیرش سریعی بتن انرژی افزایش پیدا می‌کند و بین سه تا چهار می‌گردد. به طور تقریبی به افزایش درجه 10 درجه سانتی‌گراد به دلتا اولیه بتن، اسلامی حدود 2 سانتی‌متر کاهش می‌یابد این کاهش باعث افزایش درصد هواييت بتن نيز گردد.

<table>
<thead>
<tr>
<th>جدول 2: نتایج آزمون اسلامی</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>آزمون</td>
<td>T=10</td>
<td>T=20</td>
</tr>
<tr>
<td>100</td>
<td>16/6</td>
<td>16/6</td>
</tr>
<tr>
<td>80</td>
<td>19/4</td>
<td>19/4</td>
</tr>
<tr>
<td>60</td>
<td>27/1</td>
<td>27/1</td>
</tr>
<tr>
<td>40</td>
<td>33/1</td>
<td>33/1</td>
</tr>
<tr>
<td>25</td>
<td>35/2</td>
<td>35/2</td>
</tr>
</tbody>
</table>

4-2- آزمایش مقاومت فشاری

نتایج مقاومت‌های 7، 14، 28، 42 و 91 روزه آزمون‌های مکعبی 15 و 100 سانتی‌متری آزمایشگاهی و مقاومت‌های 28 روزه نمونه‌های استوانه‌ای 7/5 سانتی‌متری کارگاهی برای 4 مخلوط از جدول (3) و نمونه (2) مشاهده می‌شود(نتایج آزمون‌های 10 سانتی‌متری اعمال ضریب 0/97 به 15 تکیه شدند).

<table>
<thead>
<tr>
<th>جدول 3: نتایج مقاومت فشاری (بر حسب مگاپیواسکال)</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>سن آزمون</td>
<td>عمل آوری</td>
<td>شرایط</td>
</tr>
<tr>
<td></td>
<td>T=10</td>
<td>T=20</td>
</tr>
<tr>
<td>7</td>
<td>16/6</td>
<td>16/6</td>
</tr>
<tr>
<td>14</td>
<td>19/4</td>
<td>19/4</td>
</tr>
<tr>
<td>28</td>
<td>27/1</td>
<td>27/1</td>
</tr>
<tr>
<td>42</td>
<td>33/1</td>
<td>33/1</td>
</tr>
<tr>
<td>91</td>
<td>35/2</td>
<td>35/2</td>
</tr>
<tr>
<td>100</td>
<td>38/3</td>
<td>38/3</td>
</tr>
</tbody>
</table>

در ارتباط با مقاومت فشاری در نمونه‌های عمل آوری شده در آزمایشگاه (در داخل حوضچه آب با دمای 22 درجه
دانیاراد با افزایش دمای اولیه بین سرعت هیدرولیس و کسب مقاومت‌های اولیه افزایش می‌یابد ولی به دلیل سرعت بالا، مشخصات هیدرولیس، فرودیتی برای پاش شدن کنوانسیون این مصرف‌نامه و تجربه آن حذف زمان معابت از هیدرولیس کامل زمان گسترش با ایجاد تخلخلی زیست در بین می‌گردد.

لذا در مقایسه با دماهای دومه، بین میان مقاومت‌های پایین تری در سنین بالا تراحتی است.

همان‌گونه که در نمودار (2) مشاهده می‌گردد تا سن 7 روز مقاومت‌های T=30 در بین سه طرح دیگر بیشترین مقرار را داراست. در فاصله سنین 14-16 روز این روند تغییر کرده و بین T=20 تا T=30 می‌توان حدس زد که در همین فاصله سنین بین 30 تا T=20 برای مدتی بیشترین مقاومت را در بین سه طرح دیگر داشته است که به پس از مدتی مقدار 20 از آن بیشتر گرفته است. مقدار T=20 در سنین بالا، از آن سن بین 10 تا 30 روز بین T=10 تا T=40 مقدار کمتری داشته و این سن به بعد بین T=40 تا T=100 کسب مقاومت‌هایی دوچندان نسبت به سه طرح دیگر نمی‌کند.

هرچنین در حال کسب مقاومت است اجتنابکه در سن 42 روز بین T=10 تا T=30 مقدار مقاومت بسیار نزدیک به می‌دارند و نهایتا در سن 91 روز بین T=10 تا 30/4 مگاپاسکال بیشترین مقدار مقاومت و T=30/33 مقدار 9/12 می‌باشد. همان طور که مشاهده می‌شود در این آزمایش، با سنجش‌ها آب و سیال مصرف شده و شرایط تغییر اخیر آوری بین از دمای 20 درجه سانتی‌گراد در سه تا 40 درجه سانتی‌گراد حدود 5/3 مگاپاسکال کاهش می‌کند و از آن 26 درجه سانتی‌گراد حدود 3/5 مگاپاسکال کاهش می‌کند در سنین اولیه.

مقدمه در مورد نمونه‌های کارگاهی تها توانسته در سن 28 و 38 روز می‌توان مقاومت‌های این اجاع گذشته است. در این سن بیشترین مقاومت شاری را دارای T=10 تا 30 تا T=40 مقدار نزدیک به می‌دارند و تا T=40 مقدار مقاومت را دارای T=10 تا 30 تا T=100 می‌باشد. در این سن نسبت به نمونه‌های آزمایشگاهی، را می‌توان در شرایط عمل ایفای و توان به سخت رجوع کرد. باید توجه داشت که دمای‌ها به هنگام عمل اوری این نمونه‌ها به طور متساوی در شیب روز می‌بود. این دمای بالا در زمان عمل اوری باعث افزایش آهنگ کسب مقاومت این نمونه‌ها نسبت به نمونه‌های آزمایشگاهی کشته چنانکه تا سن 42 و 6 روز توانسته اند به پیش بیشتری از مقاومت خود دست دستی کند. ولی به دلیل رطوبت رسانی کمتر در طول مدت عمل اوری، این نمونه‌ها به مقاومت‌های کم‌تری نسبت به نمونه‌های عمل آمده در آزمایشگاه دست یافته اند. (3).
نمودار ۱- تغییرات مقاومت فشاری با افزایش سن نمونه‌ها در دمای‌های مختلف

بتای برای این طور به نظر می‌رسد که هر دو عامل دمای اولیه و ناپایداری تاثیرگذار در آهنگ و میزان کسب مقاومت‌های نهایی بنت‌هاست. لذا در سنگ‌شکل واریزی مقاومت بنت در سینه‌های مختلف، این دو عامل و تاثیرات آنها نیز بهتر است مورد توجه قرار گیرد.

نمودار ۲- تغییرات مقاومت فشاری نمونه‌های آزمایش‌گاهی و کارگاهی نسبت به دمای اولیه بین در سینه ۲۸ روز

نتایج آزمایش مقاومت کششی (کشش برزیلی)
نتایج مقاومت کششی ۲۸ روزه آزمایش‌های استانداردی ۳۰×۱۵ از آزمایش‌گاهی و همچنین مغزه‌های ٥/٧ سانتیمتری جداول کارگاهی در جدول (۴) و نمودار (۴) مشاهده می‌شود.
جدول 4: نتایج آزمایش مقاومت کشنده بوزنی

<table>
<thead>
<tr>
<th>سن آزمون</th>
<th>عمل آوری</th>
<th>شرایط</th>
<th>فشار</th>
<th>زمان طرح و عوامل مقاومت کشنده (mpa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T=40</td>
<td>2/40</td>
<td>آرایشگاهی</td>
<td>28 روزه</td>
<td>100 %</td>
</tr>
<tr>
<td>T=30</td>
<td>2/64</td>
<td>آرایشگاهی</td>
<td>28 روزه</td>
<td>100 %</td>
</tr>
<tr>
<td>T=20</td>
<td>2/86</td>
<td>آرایشگاهی</td>
<td>28 روزه</td>
<td>100 %</td>
</tr>
<tr>
<td>T=10</td>
<td>2/64</td>
<td>آرایشگاهی</td>
<td>28 روزه</td>
<td>100 %</td>
</tr>
</tbody>
</table>

نمودار 3: تغییرات مقاومت کشنده نمونه‌های آزمایشگاهی و جدول در دماهای مختلف

مقاومت کشنده و فشاری بین در ارتباط مستقیمی با یکدیگرند لذا به همان دلیل که در مورد مقاومت فشاری بین ذکر گردید، در سال 87 روز مقاومت کشنده نمونه‌های T=20 پیشترین مقدار و نمونه‌های T=40 کمترین مقدار را دارا بود.

در مورد مقاومت کشنده چندین نکته قابل تأمل است. اول آنکه اهمیت فشاری مقاومت کشنده با افزایش عمر بین نسبت به مقاومت فشاری آن که درست است بنابراین نسبت مقاومت کشنده به فشاری با افزایش عمر بین کاهش می‌یابد. همچنین مقاومت کشنده در برای عمومی بسیار حساس است (اثرات جمع شدگی غیر یکنوخت سیبیار حادرویی کشنده نسبت به فشار آن از گذشته)

بنابر این می‌توان گفت که علت کاهش مقاومت T=10 نسبت به T=20 در سن 28 روز در هر دو سری نمونه‌ها به خصوص نمونه‌های کارگاهی (رشد کندتر مقاومت کشنده نمونه‌های T=10 T=20 در مقایسه با مقاومت نمونه‌های کارگاهی T=20) مقاومت فشاری آنها داشته. (در حالیکه مقاومت فشاری 28 روز نمونه کارگاهی T=10 بیش از T=20 بود.)

ترکم مناسب در بین نازه هنگام قرار گیری در قالب در بهره مقاومت کشنده بین بسیار موتر است. از انگا که نمونه‌های آزمایشگاهی ضمن داشتن شرایط عمل آوری مناسب تر از تراکم پهنه‌ای نزی بروخوردار بوده اند. لذا مقاومت کشنده بزرگتری نسبت به نمونه‌های کارگاهی دارا می‌باشد.

همچنین در متابع ذکر شده است که به دلیل حساسیت مقاومت کشنده در برای عمل آوری نسبت نشادن کشنده به فشاری در بینی که در هوا عمل آمده نسبت به بینی که در شرایط استاندارد عمل آوری شده کمتر است. ولی در این آزمایش چنین تبعیضی ای حاصل نگشت و نسبت مقاومت کشنده به فشاری در نمونه‌های عمل آمده در
آزمایشگاه کمتر از ین نسبت در نمونه‌های عمل آمده در کارگاه بود. (آزمایشگاه‌ها استفاده کارگاهی که از توانایی اتمی‌کننده دمای اولیه را تبادلی در کاهش مقاومت فشاری نمونه‌های کارگاهی نسبت به مقاومت کششی آنها دارد که این امر باعث مقاومت کششی به مقاومت فشاری این نمونه‌ها در مقایسه با ین نسبت در نمونه‌های آزمایشگاهی می‌گردد.

نمودار 4: نسبت مقاومت کششی به فشاری در نمونه‌های آزمایشگاهی کارگاهی در شرایط مختلف.

۵-۶-۵ آزمایش تعیین مقاومت ویژه الکتریکی

در این آزمایش از آزمون‌های مکعبی میلی‌متری با شرایط عمل آمده آزمایشگاهی و مغزه‌های ۲±۰.۰۱۰۰۰ میلی‌متری به کار گرفته شد. تعداد آزمون‌های ساختمان شده در این آزمایش برای هر مخاطب با وسیله نامیده ماهیت‌هایی بوده که در سال۷۴، ۷۶، ۷۸، ۷۱ و ۹۱ روز، آزمون‌های آب خارج و پس از انجام آزمایش، تحت آزمایش مقاومت فشاری قرار گرفتند.

برای تعیین مقاومت الکتریکی آزمون‌های ساخته در استانداردهای مختلف دستورالعمل خاصی وجود ندارد. از این رو برای انجام این آزمایش از وسیله و روش خاص استفاده شد.

برای انجام این آزمایش از یک دستگاه تعیین کننده مقاومت الکتریکی با فرکانس ۱ kHz و ضریب نهایی مΩ مخصوص به همراه دو صفحه مسی استفاده شد.

این آزمایش در سال‌های ۲۸ و ۹۱ روز بر روی نمونه‌های ۱۰ و ۱۵ سانتی‌متری آزمایشگاهی و در مورد جداول به دلیل بهبودهای موجود فقط در سال ۷۸ روز بر روی مغزه‌های ۵۷ سانتی‌متری انجام گرفت که نتایج این آزمایش در جدول (۴) نشان داده شده است.

جدول ۴: نتایج آزمایش مقاومت ویژه الکتریکی

<table>
<thead>
<tr>
<th>شرایط عمل اوری</th>
<th>سن ویژه الکتریکی (W·cm)</th>
<th>شرایط عمل اوری</th>
</tr>
</thead>
<tbody>
<tr>
<td>T=۴۰</td>
<td>۱۸۷۳</td>
<td>۱۸۰۸</td>
</tr>
<tr>
<td>T=۳۰</td>
<td>۱۸۷۳</td>
<td>۱۸۰۸</td>
</tr>
<tr>
<td>T=۲۰</td>
<td>۱۸۷۳</td>
<td>۱۸۰۸</td>
</tr>
<tr>
<td>T=۱۰</td>
<td>۱۸۷۳</td>
<td>۱۸۰۸</td>
</tr>
</tbody>
</table>
نمودار 5- تغییرات مقاومت ویژه الکتریکی نسبت به دمای اولیه بتن و زمان.

نمودار 6- تغییرات مقاومت ویژه الکتریکی نسبت به زمان و دمای اولیه بتن.

همانگونه که در جدول (۴) و نمودارهای (۶ و ۷) مشاهده می‌گردند در نمونه‌های آزمایشگاهی در سی ۲۸ روز نمونه‌های T=۲۰ درایی بیشترین مقادیر و پس از آن به ترتیب نمونه‌های T=۳۰ و T=۱۰ و در نهایت نمونه T=۴۰ درایی کمترین مقادیر مقاومت ویژه الکتریکی است. ولی تفاوت بین مقاومت نمونه‌های T=۲۰، T=۳۰ و T=۱۰ در این سن به نسبت کننده زیاد نمی‌باشد.

در مورد نمونه جدول این اختلافات بیشتر بود چنانچه نمونه‌های T=۲۰ و T=۳۰ درایی بیشترین نسبت مقاومت و پس از آن نمونه‌های T=۱۰ به دلیل درجه هیدراسیون ضعیف تر نسبت به دو نمونه قرار داشت و کمترین مقاومت در نمونه T=۴۰ به دلیل بالاتری ضعیف تر اجزای بتن و نفوذ پذیری زیاد آن قرار داشت.

علت اختلاف بیشتر، بین نتایج نمونه‌های جدول دو می‌توان ناشی از تفاوت در شرایط عمل آوری آنها نسبت به نمونه‌های کارگاهی و هیدراسیون سریعتر این نمونه‌ها نسبت به نمونه‌های آزمایشگاهی دانست.

این آزمایش در سی ۲۸ روز بر روی نمونه‌های آزمایشگاهی انجام گردید و باز نمونه T=۲۰ درایی بیشترین مقدار

۵۷
مقدار مقاومت را دارا بود.

در سن 91 روز روند تغییر کرد و نمونه $T=10$ دارای بیشترین مقدار مقاومت و پس از آن نمونه‌های $T=20$, $T=30$, $T=40$, $T=50$, $T=60$ و $T=70$ نمونه در حدود 1000 آهم-سانتیمتر بود.

به نظر می‌رسد که سرعت کند هیدراسیون و تکامل ساختمان نمونه در $T=10$ باید کم مقاومت الکتریکی واقعی آن در سن 91 روز گردد.

اگر چه به نظر می‌رسد که این آزمایش در مورد بیشتر که در دماهای اولیه متفاوت ساخته شده اند، بهتر است در سنین بالای 90 روز انجام گیرد.

نمودار 7- تغییرات مقاومت ویژه الکتریکی نسبت به تغییرات دما و سنی نمونه در سنین مختلف در نمونه‌های آزمایشگاهی و جداول کارگاهی

**5- نتیجه‌گیری و پیشنهاد**

- به نظر می‌رسد که روابط ارائه شده در برابر دما، بر اساس مقادیر داخلی و در عمل در دما محیط اطراف ساخت بین و شرایط محیطی ساخت آن، در دما تعادل آن بسیار نافذ است. لذا بهتر است عمل دما محیط در حین ساخت بین نیز، در این روابط لحاظ شود.

- در سنین اولیه بین پراکنده تایید کرد که بر روی ویرگول‌های بین تنها می‌باشد. همچنین به نظر رسیده که در سنین بالای 90 روز در سنین بالای 90 روز در سنین بالای 90 روز در سنین بالای 90 روز در سنین بالای 90 روز دمای مورد نظر نسبت به دو بین $T=20$, $T=30$, $T=40$, $T=50$, $T=60$ و $T=70$ نمونه در حدود 1000 آهم-سانتیمتر بود.

- به نظر می‌رسد که سرعت کند هیدراسیون و تکامل ساختمان نمونه در $T=10$ باید کم مقاومت الکتریکی واقعی آن در سن 91 روز گردد.

با افزایش دما، مقاومت فشاری در سنین اولیه آزمایش می‌باید لیا افزایش عمر بین این روند تغییر می‌باشد و این شانه‌های روند کنر هیدراسیون این بین و کسب مقاومت‌های پیشر در سنین بالا تر آن می‌باشد.
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Simple Beam with Third-Point Loading.


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CD03
Construction Technology and Implementation Methods
EFFECT OF CASTING TEMPERATURE ON THE RISK OF CRACKING IN THE MASS CONCRETE COLUMNS CONTAIN SILICA FUME

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ABSTRACT

In this paper the effect of different casting temperatures on cracking potential of mass high strength columns were studied. Four mix proportions with 0.3 water-cementing ratio were made and two different casting temperatures, 26\(^\circ\)C and 40\(^\circ\)C were applied to simulate moderate and hot climate conditions. Cement was replaced by silica fume at 0\%, %5, %8 and %11. Temperature rising of the mixtures, due to hydration, after casting was monitored till 7 days. Furthermore, compressive strength of the specimens was determined at the ages of 1 to 91 days. Thermal analysis of 4 large columns, 600×600×200 to 1800×1800×2000 mm, was carried out by finite element method \([3-7]\). In this regard, two demolding time, 24 and 48 hours were selected to clarify the effect of ambient temperature on risk of cracking. The results declare that tensile stress of the 600 mm columns with 24 hours demolding time, irrespective of casting temperature, was higher than tensile strength and will be cracked. However, 48 hours demolding time induced lower tensile stress and diminished risk of cracking. Silica fume has no considerable effect on risk of cracking.

Keywords: mass concrete, casting temperature, risk of cracking, silica fume, demolding time

1. INTRODUCTION

Heat evolution during hydration of cementing materials in high strength mass concrete lead to thermal stress, which in-turn induce thermal cracking in the body of structures\([1-2]\).Therefore, this is a task of researchers to clarify thermal behavior of mass structures made by high strength concrete to develop a convenient design method to control crack potential \([5]\). Huge foundations, pile, columns of bridges, thick walls and tunnels lining are examples of mass structures which thermal cracks were observed. In large structures, due to low thermal diffusion properties of concrete, heat spreading is very slow \([6-8]\). In this regard, concrete temperatures in excess of 65\(^\circ\)C have been reported \([8]\). The high temperature can adversely affect the performance of the concrete. Thermal cracking will occur when thermal stresses exceed the tensile strength of concrete \([9]\). The geometry of a member, water cement ratio, cement content and type of supplementary cementing materials govern the magnitude of heat lost to the environment and lead to high thermal
Cement replacement with different pozzolan is known as an important way to diminish temperature rising \[8\]. Effect of silica fume on hydration heat is influenced from water cementitious material and also superplasticizer content. It was observed that in high water cement ratio, about 0.5, silica fume behave as cement \[4\]. However, as water cement ratio decrease silica fume have no considerable effect on hydration heat development. On the other hand, at water cement ratio of 0.4 silica fume diminish hydration heat \[5\]. Slag-blended cements assist in reducing hydration temperatures of concrete with nominal strength of 100, 80, 60 and 40 Mpa \[9\]. Casting temperature and demolding time are the other important parameters which may affect risk of cracking in silica fume specimens which are studied in the present paper.

2. MATERIALS AND TESTING METHOD
Crushed stone, with 19 mm maximum nominal size, in two ranges of 5-10 and 10-19 with relative density at saturated surface dry of 2.61 were used. Fineness modulus of sand and relative density was 3.4 and 2.6 respectively. Absorption value is 1.9 and 2.1 for fine and coarse aggregate. The cement used was Portland cement Type 2, with a specific gravity of 3.12 and 3750 cm$^2$/gr surface area. A commercial carboxylic type plasticizer, (Gelenium 110M), was used to maintain workability of fresh concrete. Silica fume, made by Semnan Ferro Alley factory, was used at 0%, 5%, 8% and 11% (by weight) as partial replacement of cement. Chemical properties of silica fume are given in Table 1. Mix proportions of the concrete are given in Table 2. Water-cementing material ratio is 0.3.

<table>
<thead>
<tr>
<th>Composition</th>
<th>Percent</th>
</tr>
</thead>
<tbody>
<tr>
<td>Al$_2$O$_3$</td>
<td>0.5-1.7</td>
</tr>
<tr>
<td>SiO$_2$</td>
<td>85-95</td>
</tr>
<tr>
<td>Fe$_2$O$_3$</td>
<td>0.4-2</td>
</tr>
<tr>
<td>C</td>
<td>0.6-1.5</td>
</tr>
<tr>
<td>CaO</td>
<td>2-2.3</td>
</tr>
<tr>
<td>MgO</td>
<td>0.1-0.9</td>
</tr>
</tbody>
</table>

For all mixtures a cylindrical (150×300mm) specimen was molded just after casting and was put in a semi adiabatic box for monitoring the hydration temperature rising. Temperature rising was monitored via a Data Logger at 1 minute interval till 6 days. Guardian, 4C-Heat and 4C Temp&Stress soft wares were used for determination of heat properties and thermal analysis. Consequently, risk of cracking in large columns was determined. The columns size was 600, 1000, 1400 and 1800 mm, and height of all columns was 2000 mm. Two climate zones, moderate with air temperature of 20 to 30°C and hot with 30 to 45°C, were chosen. Wind velocity was 5m/sec. Sixteen four analyses were carried out for
determination of cracking risk (tensile stress/tensile strength). Furthermore, compressive strength of the cube specimens (100×100×100 mm) accordance to standard condition was also measured at the ages of 1, 2, 3, 7, 14, 28 and 91 days.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement (kg/m³)</td>
<td>540</td>
<td>513</td>
<td>496.8</td>
<td>480.6</td>
</tr>
<tr>
<td>Water (kg/m³)</td>
<td>164.7</td>
<td>164.7</td>
<td>164.7</td>
<td>164.7</td>
</tr>
<tr>
<td>Fine agg. (kg/m³)</td>
<td>865</td>
<td>859</td>
<td>856</td>
<td>853</td>
</tr>
<tr>
<td>Coarse agg. (5-10 mm)</td>
<td>-</td>
<td>203</td>
<td>202</td>
<td>202</td>
</tr>
<tr>
<td>Coarse agg. (10-19 mm)</td>
<td>-</td>
<td>626</td>
<td>623</td>
<td>621</td>
</tr>
<tr>
<td>Micro silica (%)</td>
<td>0</td>
<td>5</td>
<td>8</td>
<td>11</td>
</tr>
<tr>
<td>Super plasticizer</td>
<td>3.24</td>
<td>7.5</td>
<td>7.5</td>
<td>7.5</td>
</tr>
<tr>
<td>Water/Cementitious material</td>
<td>0.3</td>
<td>0.3</td>
<td>0.3</td>
<td>0.3</td>
</tr>
<tr>
<td>Slump</td>
<td>150</td>
<td>150</td>
<td>150</td>
<td>150</td>
</tr>
</tbody>
</table>

*Aggregate in saturated surface dry condition

3. TESTING RESULTS

Casting temperatures for moderate (M) and hot climates (H) is given in Table 3. As it is shown, for moderate climate casting temperature is between 26.5 to 28 °C. However, casting temperature for hot climate is between 36 to 41 °C. Casting temperature was adjusted based on trial mixtures and the proposed equation in [1].

<table>
<thead>
<tr>
<th>S0</th>
<th>SF5</th>
<th>SF8</th>
<th>SF11</th>
<th>S0</th>
<th>SF5</th>
<th>SF8</th>
<th>SF11</th>
</tr>
</thead>
<tbody>
<tr>
<td>Casting Temp. (M)</td>
<td>27.5</td>
<td>27</td>
<td>26.5</td>
<td>28</td>
<td>36</td>
<td>40.5</td>
<td>41</td>
</tr>
</tbody>
</table>

After casting, temperature rising was monitored via a semi adiabatic box. Heat development parameters were also calculated from the following equation using 4C-Heat software:

\[ Q(M) = Q_o \cdot \exp\left(-\left(\frac{\tau_c}{M}\right)^\alpha\right) \]  

(1)

Where,
- \( Q(M) \) Correspond to heat value at maturity \( M \),
- \( Q_o \) Is final heat value and
- \( \tau_c \) And \( \alpha \) are constants.

Using the above parameters and the measured strength properties in predetermined casting and curing and also ambient temperature, temperature regime, peak temperature, stress and risk of cracking were calculated by 4C Temp&Stress.
3.1. Moderate Casting Temperature

Peak temperature of different mixtures, due to hydration of cementing materials, is shown in Figure 1. As it is shown, higher peak temperature belonging to the columns with larger size. In the specimens without silica fume peak temperature change between 65 °C to 85°C for 600 and 1800 mm columns size respectively. Silica fume diminished peak temperature not so effectively. Risk of cracking for 24 and 48 hours demolding time are also demonstrated in Figures 2 and 3 when respectively. It is shown that risk of cracking of the specimens was changed in the same manner with peak temperature. Demolding time of 24 hours lead to this fact that only 600 mm column can be conservatively. However, 48 hours demolding time decreased risk of cracking and 1000mm column will also be considered without cracking.

![Figure 1. Peak temperature versus concrete mixtures (Moderate)](image1)

![Figure 2. Risk of cracking versus mixtures (Moderate) – remolding time: 24 hours](image2)

3.2. High Casting Temperature

Temperature rising, risk of cracking at demolding time of 24 and 48 hours were shown in Figures 4, 5 and 6 when casting temperature was about 40 °C. Due to high casting temperature peak temperature values increased more than moderate one. However, a comparison of risk of cracking in moderate and hot casting
temperature declare that, due to lower thermal gradient in hot casting lead to lower risk of cracking. As it is shown, 48 hours demolding was also diminished risk of cracking. Silica fume have no considerable effect on cracking potential, just similar to moderate casting temperature.

Figure 3. Risk of cracking versus mixtures (Moderate) – remolding time: 48 Hours

Figure 4. Peak temperature versus concrete mixtures (Hot)

Figure 5. Risk of cracking versus concrete mixtures Remolding time: 24 hours
4. CONCLUSIONS
From the present study the following conclusions can be drawn:
- Hydration peak temperature is highly affected by size of structures.
- Higher casting temperature led to higher peak temperature.
- Silica fume had no considerable effect on hydration peak temperature of the mixtures.
- High casting temperature (40 °C) led to high peak temperatures however, due to low thermal gradient risk of cracking decreased.
- It was concluded that demolding time of 48 hours in both casting temperatures diminished risk of cracking.

REFERENCES
1. Neville, A.M. “Properties of Concrete”.
2. ACI Committee 207, 1996, “Mass Concrete (ACI 207.1R-96),” American Concrete Institute.
EVALUATION OF THE FLUIDITY AND MECHANICAL PROPERTIES OF LIGHT-WEIGHT SELF-COMPACTING CONCRETE CONTAINING EXPANDED POLYSTYRENE (EPS)

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ABSTRACT
This paper presents the results of an experimental investigation on the effects of expanded polystyrene polymeric beads on the properties of light-weight self-compacting concrete in fresh and hardened states. Since the aim of this study is to produce structural lightweight self-compacting concrete (with compressive strength above 17 MPa), EPS beads were partially substitute for aggregates by 10, 15, 22.5 and 30 percentages by volume. Fluidity and mechanical properties of self-compacting EPS concrete was compared with ordinary self-compacting concrete with slump flow of about 65cm. The slump flow was kept in allowable range and the effects of EPS were evaluated on the other fluidity parameters such as V-funnel and L-box. The results obtained, showed that with increasing in EPS volume, V-funnel time increased while blocking ratio decreased. At the stage of hardened concrete, compressive strength (at different ages), tensile strength, ultrasonic pulse velocity (UPV) and water absorption were studied.

Keywords: light-weight self-compacting concrete, expanded polystyrene, fluidity, mechanical properties

1. INTRODUCTION
Light-weight concrete with density varying between 1400 to 2100 kg/m\textsuperscript{3} in contrast with normal concrete with density about 2400 kg/m\textsuperscript{3} has been used for structural purposes for so many years. Light-weight concrete has been center of attention because of its low density leading to the decrease in the area-sectional which results in decreasing the final cost of structure. Light-weight concrete used in this study is made by using porous fine aggregates with a low density. Water absorption is the most significant demerit of many of these light-weight aggregates. From a fresh state point of view, increasing the amount of water in matrix (associated with water absorption of this aggregates) can cause increase in the amount of structure self-weight. Also, this water absorption cause decreasing in slump flow and to solve this problem additional water required. In term of hardened state, as the time passes, the evaporation causes rapid loss of surface bleeding which results rapid drawdown in pore water level. This in turn, makes an increase in pore water pressure, which tends to bring the neighboring solid particles closer which resulted that shrinkage cracking appears [1,2]. Different studies were
shown that because of interesting in properties such as hydrophobia, closed cell nature (prevention from going paste or water in to light-weight structure) and non-absorbent, employing the light-weight expanded polystyrene beads as suitable ultra light-weight aggregates in mortar or concrete for structural and non-structural applications is increasing [3,4]. These properties can overcome disadvantages discussed above.

Self-compacting concrete (SCC) was first produced in Japan in 1980s [5] with problems in consolidation of the normal concrete was to achieve concrete with favorite compaction and high durability. In fact, SCC has a very high fluidity, which removed many of the problems associated with normal concrete such as segregation, bleeding, absorption, permeability and etc. In addition, without any vibration placement in complex or dense reinforced formworks, filled it and covered the space around the bars. Compared to normal concrete, reduction of the harmful effects of sound in urban environments and the industrial process costs are additional advantages [6]. Fillingability, passingability and resistance to segregation are three key properties that SCC must comprise at fresh state [7]. The first two properties are achieved by employing a high-range water reducing. In order to avoid the segregation of coarse aggregates, the plastic viscosity of SCC should be increased. For these purpose, three methods were suggested. The First is using high powder content. Employments of some mineral admixtures such as fly ash, ground blast furnace slag, silica fume, limestone powder or quartzite powder is a possible way for the first method [8,9]. The second is employing a viscosity-modifying agent (VMA). Finally, the last one is a combination of the first and second methods.

Because of self-weight of light-weight aggregates, for compaction of concrete may results in increasing in segregation and bleeding problems. Noticeable problem associating with unsuitable vibration is inefficient dissipation of light-weight aggregates. This condition raises light-weight aggregates to the surface of concrete and forms a weak layer. As the SCC which dose not needs any vibration, it looks like that these problems can be kept at the lower level. But because that SCC must be compacted by its own weight (without any compaction) and the weight of self-compacting light-weight concrete reduces in comparison to SSC, so the balance between compaction and weight of SCC must be established. The major objective of the present study is to provide information about this balance in light-weight self-compacting concrete. To attain this aim and because of the interesting property of EPS as ultra light-weight aggregates, on this study,

2. EXPERIMENTAL PROGRAMME
2.1. Materials
Ordinary Portland cement (opc) meeting the requirements of ASTM C 150 were used for preparation of the self-compacting EPS concrete specimens in all compositions. Commercial dry uncompacted silica fume (SF) was used as a cementations material. The chemical compositions of OPC and SF are given in Table 1.


<table>
<thead>
<tr>
<th></th>
<th>Cement</th>
<th>Silica fume</th>
</tr>
</thead>
<tbody>
<tr>
<td>SiO₂</td>
<td>21.46</td>
<td>91.7</td>
</tr>
<tr>
<td>Al₂O₃</td>
<td>5.55</td>
<td>1</td>
</tr>
<tr>
<td>Fe₂O₃</td>
<td>3.46</td>
<td>0.9</td>
</tr>
<tr>
<td>CaO</td>
<td>63.95</td>
<td>1.68</td>
</tr>
<tr>
<td>MgO</td>
<td>1.86</td>
<td>1.8</td>
</tr>
<tr>
<td>SO₃</td>
<td>1.42</td>
<td>0.87</td>
</tr>
<tr>
<td>K₂O</td>
<td>0.54</td>
<td>-</td>
</tr>
<tr>
<td>Na₂O</td>
<td>0.26</td>
<td>0.1</td>
</tr>
<tr>
<td>LOI</td>
<td>-</td>
<td>2</td>
</tr>
</tbody>
</table>

Table 2: Grading and physical properties of expanded polystyrene

<table>
<thead>
<tr>
<th>EPS type</th>
<th>Type I</th>
<th>Type II</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sieve size (mm)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>9.5</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>4.75</td>
<td>96</td>
<td>2</td>
</tr>
<tr>
<td>2.36</td>
<td>2</td>
<td>0</td>
</tr>
<tr>
<td>1.18</td>
<td>2</td>
<td>0</td>
</tr>
</tbody>
</table>

Physical properties

| Mostly beads size (mm) | 2.36 | 4.75 |
| Specific gravity      | 0.025| 0.018 |

To mix the self-compacting EPS concrete and for curing the specimens, potable water specified by ASTM D 1129 was used. Natural river sand (0-4.75mm) with fines modulus of 2.83 and crushed gravel with 12.5mm were used. The coarse and fine aggregates have specific gravities of 2.71 and 2.6 and mean water absorption of 0.8% and 0.58%, respectively. Sika Viscocrete-1 as a third generation of super-plasticizer and meets the requirements for super-plasticizer according to SIA 162 (1989) was used in all mixtures. This type of modified polycarboxylate based has been used by other researchers as VMA [10]. Solid content, PH and specific gravity of VMA were provided by its manufacturer to 35.7%, 6.5 and 1.08, respectively. Expanded polystyrene equally replaced by fine and coarse aggregates by using two types of commercial EPS with different specific gravity and grading which were used to prepare self-compacting EPS concrete. Type 1 with mostly 2.36 mm beads size and type 2 with mostly 4.75 mm size replaced fine sand and gravel, respectively. As seen, to prevent grading disturbance of coarse and fine aggregates, mostly beads size was selected in the grading range of aggregates, respectively. The EPS beads properties and grading details are presented in table 2.

2.2. Mixture Proportion of Self-Compacting Eps Concrete

The mixing sequences were as follows. Coarse and fine aggregates, EPS and some mixing water (25%) were initially homogenized for 1min in rotary planetary mixer.
Thereafter, binder materials including cement and SF were added. Finally, the remaining water and VMA (according to Table 3) were introduced to the wet mixture, while mixing was going on for 4 minutes. This optimum time is required to disperse VMA and stabilize viscosity. The water/cement ratio and binder content of the mixtures were maintained at 0.53 and 418 kg/m$^3$ throughout this study, respectively. In order to keep slump flow in allowed ranges, VMA was used in different dosages. The VMA was adjusted between 0.4 to 0.63% by weight of binder content and EPS beads replaced aggregates with 10, 15, 22.5 and 30 percentages by volume to produce structural self-compacting EPS concrete with compressive strength above 17 MPa and density ranging 1700-2100 kg/m$^3$. Fluidity and mechanical properties of self-compacting EPS concrete was compared with ordinary SCC that designed for slump flow about 65cm. The details of the mixtures including Silica fume which was replaced in 10% of the cement mass are presented in Table 3.

<table>
<thead>
<tr>
<th>Mix No.</th>
<th>Cement (Kg/m$^3$)</th>
<th>SF (Kg/m$^3$)</th>
<th>Water (Kg/m$^3$)</th>
<th>Gravel (Kg/m$^3$)</th>
<th>Sand (Kg/m$^3$)</th>
<th>% volume of EPS</th>
<th>%VMA of binder</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>380</td>
<td>38</td>
<td>196</td>
<td>822</td>
<td>925</td>
<td>0</td>
<td>0.62</td>
</tr>
<tr>
<td>2</td>
<td>380</td>
<td>38</td>
<td>196</td>
<td>689</td>
<td>792</td>
<td>10</td>
<td>0.55</td>
</tr>
<tr>
<td>3</td>
<td>380</td>
<td>38</td>
<td>196</td>
<td>623</td>
<td>726</td>
<td>15</td>
<td>0.50</td>
</tr>
<tr>
<td>4</td>
<td>380</td>
<td>38</td>
<td>196</td>
<td>524</td>
<td>626</td>
<td>22.5</td>
<td>0.45</td>
</tr>
<tr>
<td>5</td>
<td>380</td>
<td>38</td>
<td>196</td>
<td>424</td>
<td>527</td>
<td>30</td>
<td>0.40</td>
</tr>
</tbody>
</table>

The major problem of light-weight concrete is desirable dispersal of aggregates in matrix. As shown in Figure 1, the specimens of self-compacting EPS concrete containing different percentages of EPS beads show desirable dispersal of EPS up to 22.5% compare to 30% in concrete specimens. Hence, a replacement up to 22.5% volume of EPS seems not to need any innovations. So, the beads can be used the way that already been explained. This should be qualified by more experiments. To assess the effect of workability loss where produced in sites, all of the fresh-state properties were tested 30 min after mixing. Some portion of the concrete mixture was used for fresh properties experiments and the remaining part was poured into the moulds in one layer and without any compaction other their self-weight to assess mechanical properties of self-compacting EPS concrete. After 24 h casting, they were demolded and curing was conducted according ASTM C 511 and then kept in the curing environment until the date of testing.

![Figure 1. Desirable dispersal of EPS in self-compacting concrete specimens](image-url)
2.3. Concrete Tests

Mechanical properties of hardened concrete are strongly related to fresh properties. Several experiments were conducted in each mixture to assess the most important features of fresh properties of light-weight self-compacting concrete. (i) Slump flow test is primarily to assess workability, filling ability and consistency of concrete without any obstructions. According to EFNARC, a SCC must have a slump flow ranging between 650-800mm. (ii) V-funnel flow test should be used to assess the ability to achieve smooth flow through narrow spacing without blockage. This test measured variable for describing the cohesion, viscosity and fillingability of SCC. Some researchers believe that V-funnel times represent the flowability and stability or segregation resistance of SCCs through V-shaped funnel test [11,12]. According to EFNARC, a stable and flowable SCC must has a V-funnel flow time varying between 6–12 s. (iii) L-box evaluates fillingability and passingability of SCC and also representative yield stress of the materials. The passing ability, on the other hand, shows the compatibility between the size of the coarsest particles of the concrete and the gap between the reinforcing steel bars in the structure to be cast [10]. According to EFNARC, a SCC must have a blocking ratio varying between 0.8-1. Fluidity is defined either qualitatively as the ease of placement or quantitatively by rheological parameters [9]. As previously expressed, ease of placement can defined by fillingability, passingability that are two of the three key properties of fresh concrete, were evaluated simply by experiments discussed before. From the quantitative view, Plastic viscosity and yield stress are two most common rheological parameters were assessed as Bingham equation. If the viscosity is too low, an increase of shear rate is recommended to avoid segregation. On the other hand, if viscosity is too high, a low shear stress would be necessary [13]. So, for self-compactibility, concrete should have an adequate plastic viscosity together with a low yield stress approaching to the behavior of a Newtonian fluid [9]. The measurement of yield stress and plastic viscosity requires a rheometer, and, where not available, alternative simpler tests such as slump flow, V-funnel times... can be used [9]. Moreover, these experiments are not costly and they can easily be carried out in sites. So, in this study the above tests were conducted to evaluate the fluidity of self-compacted EPS concrete. The water content in a mixture can be classified into free water and the bound water. Free water is the interstitial water existing between fines and aggregates. It disperses and lubricates the solid particles in concrete to create fluidity and plasticity of concrete. Therefore, it is the quantity and quality of free water that determines much of the rheological behavior of fresh concrete [14]. As mentioned above, hydrophobic is one of the main features of EPS beads, so increasing replacement of EPS in concrete can cause an increase the free water in matrix which affects the fluidity of self-compacting EPS concrete. On the other hand, increase in EPS in matrix can change many things such as size, kind and configuration of aggregates, decrease internal friction and, above all, decrease the weight of matrix in ratio with the condition without EPS. So, it has been tried to estimate these affects on fresh self-compacting EPS concrete by means of the above experiments and obtain the desirable EPS percentage. For each concrete mixture, the average compressive
strength of three 100mm cubes at the age of 7, 28 and 60 days were obtained. Compression test loading was done by a system with the maximum capacity of 3000 KN and a loading rate of 0.25 N/mm²s per specimens. 15×30 cylindrical was taken from the mixture to measure tensile strength. Water absorption tests were conducted on three 100 mm cube specimens according to the ASTM C 642 at 28 days. For the determination of water absorption, because of the temperature suggested by ASTM (100-110°C) EPS beads initially shrink and finally evaporate, saturated surface dry cubes were placed in an oven at 60 °C until a constant mass was achieved [15]. Before measuring compressive strength of the specimens at 28 days, the UPV measurements were conducted to measure the time needed for propagation of a sonic waves according to ASTM C 597.

3. RESULTS AND DISCUSSIONS
3.1. Fresh Concrete Tests Results
The first part of the results was attributed to the fresh properties of self-compacted EPS concrete. In Figure 2 the common effects of density and EPS percentage on slump flow and V-funnel time shows. Since Slump flow is not a suitable factor to exhibit the fresh behavior exactly, so the slump flow was kept in allowed range specified by EFNARC and other parameters effective on fluidity were evaluated. Segregation and bleeding were visually checked during the slump flow test and were not observed in any of the mixtures with the exception of mix No.5 in which segregation of aggregate near the edges of the spread-out concrete was observed. In other words, for light-weight self-compacting concrete with slump flow over than 680 mm, segregation symbols often appears. It can be seen from Figure 2, slump flow values were 640-700mm, while mix No. 5 has lower contents of VMA, The biggest slump flow attributed the maximum percentage of EPS in volume and mix No. 1 has minimum slump flow. Therefore, slump flow increased with the increase in EPS percentages. It was due to decreased in internal friction with an increase in EPS percentages. This, in turn, shows a better flowability in self-compacting EPS concrete in compare to SCC with no EPS. But, this is not enough for evaluating the fluidity of self-compacting EPS concrete, so other experiments must be conducted to assess other parameters such as viscosity, cohesion, segregation, blocking and etc. The time measured via the V-funnel flow was in the range of 6–15 s. As observe in Figure 2, an increase in EPS percentage caused an increase in V-funnel time, while slump flow has target range 60-70 cm. The V-funnel times for all mixtures were within the EFNARC range of SCC, apart from mix No. 5 which exceeded upper limit of V-funnel time. Felekoglu and his colleague [16] reported that the V-funnel time will decrease with increasing in water/powder values or free water content increase. But the results showed in self-compacting EPS concrete increasing in free water content, produced by increase in the hydrophobia EPS beads in matrix will increase V-funnel time. On the other hand, internal friction of matrix was decreased with increasing in the amount of EPS in concrete due to smooth surface of EPS compare to aggregates.
In the other words, limiting the coarse aggregate content by replacing them with EPS will cause the relative distance between coarse aggregates to be increased. Therefore, the frequency of colliding between aggregates decrease and finally, the energy dissipation decrease. So, it seems with an increase in EPS content, the V-funnel time was decreased. But an increase in EPS percentage caused increasing V-funnel time. The reason probably is the lower self-weight of self-compacting EPS concrete compare to SCC. An increase in EPS in matrix decrease self-weight and a decrease flowability of matrix in the funnel lead to a decrease the V-funnel time. So in self-compacting EPS concrete, self-weight is a major parameter to characterize the fresh-state behaviors while in SCC, free water is the major case. In the base of EFNARC range for V-funnel time varying between 6-12 s, mix No.5 is rejecting. The results show that Light-weight self-compacting concrete with density higher than 1900 kg/m³ satisfies the fresh-state behaviors of concrete related to viscosity, cohesion and segregation and lower amount of density were rejected. In the blocking ratio point of view L-box test was performed in all mixtures. Results of blocking ratio of all the different mixes are presented in Figure 3. Blocking ratio vary between 0.9-0.78 for self-compacting EPS concrete. As the effect of EPS, blocking ratio decreased with the increased in EPS percentages. So, in the base of EFNARC, mix No. 5 is rejecting. Aggregate blocking is more probable in mixtures where blocking ratio is lower than 0.8. But in self-compacting EPS concrete, it did not seem that blocking ratio lower than 0.8 for mix No. 5 are pertinent to aggregates blocking between gaps. When aggregates replaced by EPS with mostly beads size lower than G_max the blocking must be solved. Like V-funnel time, probably the lower self-weight of self-compacting EPS concrete compare to SCC is the reason. Fewer Self-weight of self-compacting EPS concrete, decreasing the flowability of the fresh concrete in horizontal section.
Figure 3. Variation of density with blocking ratio

3.2. Hardened Concrete Tests Results
The second part of results was attributed to the hardened properties of self-compacted EPS concrete. These results are reported in Table 4.

<table>
<thead>
<tr>
<th>Mix No.</th>
<th>Density (Kg/m³)</th>
<th>Compressive strength (MPa)</th>
<th>split tensile strength (MPa)</th>
<th>UPV (Km/s)</th>
<th>absorption</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>7 days (28 days</td>
<td>60 days</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>2413</td>
<td>32.5</td>
<td>47.3</td>
<td>5.37</td>
<td>4.808</td>
</tr>
<tr>
<td>2</td>
<td>2195</td>
<td>26.2</td>
<td>33.2</td>
<td>4.17</td>
<td>4.545</td>
</tr>
<tr>
<td>3</td>
<td>2039</td>
<td>21.1</td>
<td>25.1</td>
<td>3.69</td>
<td>4.315</td>
</tr>
<tr>
<td>4</td>
<td>1906</td>
<td>18.7</td>
<td>21.9</td>
<td>3.14</td>
<td>4.042</td>
</tr>
<tr>
<td>5</td>
<td>1702</td>
<td>14.8</td>
<td>17.1</td>
<td>2.69</td>
<td>3.724</td>
</tr>
</tbody>
</table>

3.3. Compressive Strength
Standard cubes were tested for compressive strength after 7, 28 and 60 days of curing. All of the mixtures represented a continuous increase in compressive strength with age. Similar to EPS concrete, the rates of strength development for self-compacting EPS concrete was greater initially and lower as the curing ages increased. The results show that the compressive strength decreases with increasing EPS content at all ages. This is due to lower strength of EPS compared to fine and coarse natural aggregates. On the other hand, bond between EPS and paste or mortar are fragile due to its smooth surface. According to Figure 4, mix No.1 developed highest compressive strength up to 47.3 MPa While mix No. 5 shows lower about 17.1 MPa at the age of 28 days. It can be observed for replacement aggregates by 30% EPS in volume, compressive strength decreased about 64% and replacement aggregates with more than 30% cause to leave structural condition.
Because of this study purpose was produced structural self-compacting EPS concrete; replacements above 30% are not conducted. Mix No.5 had structural strength, but in fresh state, it was rejected. This means that concrete in fresh state is more remarkable than hardened one. Variation of compressive strength with different percentage of EPS by volume or density is represented in Figure 4 and 5, respectively. Density is one of the most important factors in concrete. In self-compacting EPS concrete, with increase in EPS volumes decreasing in density were shown. Also, as shown in Figures 4 and 5, approximately linear relationships were evaluated between EPS volume and density with Compressive strength. This relationship was confirmed by other researcher for light-weight EPS concrete containing SF [15].

![Figure 4. Variation of EPS vol. with compressive strength](image1)

![Figure 5. Variation of density with compressive strength](image2)

### 3.4. Tensile Strength

Tensile strength is one of the most important fundamental properties of concrete. In Table 4, the influence of EPS content on splitting tensile strength is presented. With increasing amount of EPS content or decreasing the density, splitting tensile strength also decreased. The relationship between compressive strength and splitting tensile strength for self-compacting EPS concrete is presented in Figure 6. It can be seen that with an increase in compressive strength, the splitting tensile
strength increases. Against to SCC in which failure is typically brittle and with separation of concrete in two pieces, in self-compacting EPS concrete failure is more gradual and the samples do not separate. This mode of failure was reported earlier for light-weight EPS concrete by K. Ganesh Babu and colleague [15].

3.5. Ultrasonic Pulse Velocity and Relationship with Compressive Strength

Ultrasonic pulse velocity (UPV) is one of the non-destructive methods for assessing the quality and homogeneity of the in situ concrete. In Table 4 UPV values for all mixtures was given. It can be observed from this table, when compared to the control mixture, use of EPS generally decreased the UPV. The variation of compressive strength with UPV is presented in Figure 7.

\[
F_t = 0.4352 F_{cu}^{0.6439} \\
\tau = 0.99
\]

Figure 6. Relationship between splitting tensile strength and compressive strength

\[
UPV = 1.061 \times \ln(F_c) + 0.784
\]

Figure 7. Relationship between UPV and compressive strength

With an increase in compressive strength, UPV was increased. The results of this study were suggested by Eq. (3) for the Relationship between UPV and compressive strength of self-compacting EPS concrete.

\[
UPV = 1.061 \times \ln(F_c) + 0.784 \tag{3}
\]

3.6. Absorption

Water absorption is one of the major parameters of durability of concrete which
related to its porosity. Water absorption is a measure of the portion of the total volume of concrete occupied by pores. According to table 4, maximum water absorption was detected with mixture with higher percentages of EPS. This can be expected that the specimens with high percentage of EPS will include to more amounting of free water. By the time when this free water vaporized will lead to an increase in pores which resulted to an increase in the amount of concrete water absorption.

4. CONCLUSION
The behavior of self-compacting EPS concrete was represented in two parts. In the first part, it was shown that light-weight self-compacting concrete with slump flow over than 680 mm, segregation symbols often appears. V-funnel time increased with increasing in the EPS percentages, while slump flow was kept in allowed ranges specified by EFNARC. But L-box was decreased with increasing in EPS volume. This was probably attributed to the lower self-weight of self-compacting EPS concrete compare to SCC.

On the other hand, Light-weight self-compacting concrete with density higher than 1900 kg/m$^3$ satisfies the fresh-state behaviors of concrete related to viscosity, cohesion and segregation and lower amount of density were rejected. The second part of this study was considered the hardened state of self-compacting concrete. The results show that compressive and tensile strengths and absorption increased with increasing EPS content, while UPV decreased.

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PRACTICAL APPLICATION OF INNOVATIVE TECHNOLOGY IN FLOOR CONSTRUCTION

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ABSTRACT
The main objective of this paper is to show the benefits of using innovative technology i.e. post-tensioning and concrete core activation for the more widespread practical applications in concrete buildings. In consideration of the spans up to 20.00 m and heavy superimposed loads structures built using unbonded post-tensioning systems provided many advantages compared to conventionally reinforced concrete structures. The advantages of using post-tensioning with unbonded tendons, structural behavior, concerning constructive aspects and economy will be illustrated by the presentation of several commercial building and parking structures. Reductions in concrete and reinforcing steel quantities are two of the advantages that directly impact overall construction costs. Concrete core activation is a heating and cooling system based on simple physical principles to create a low-cost system. Concrete possesses a large capacity to store heat, and water functions well as a heat conductor.

Keywords: unbonded post-tensioning, free tendon layout, floor construction, fire protection, fire resistance, concrete core activation

1. INTRODUCTION
Slabs supported on columns are defined as flat slabs. For the purpose of this section flat slabs may be of uniform thickness or they may incorporate drops (thickenings over columns). Figure 1 illustrates typical arrangements of slabs in floor construction.

- **Flat Slab**–Economical for spans between 6.0 m and 9.0 m. Loads light to medium.
- **Flat Slab**–Economical for spans between 6.0 m and 9.0 m. Loads light to medium.
- **Continuous Band**–Effective for large spans, 8.0 m to 12.0 m. Loads light to medium.
- **Beam and Slab**–One span is excessive, up to 20.0 m and the other does not exceed 8.0 m to 9.0 m. Loads medium to heavy.
- **Ribbed One Way**–Recommended where spans are predominantly in one direction. Loads medium to heavy.

Figure 1. Typical arrangements of slabs in floor construction
2. UNBONDED MONOSTRAND SYSTEM
The unbonded tendon as the name implies is never bonded to the concrete. The strand is carefully coated with grease which gives a low frictional resistance and contains anti-corrosive additives. Separating the grease from the concrete is an extruded plastic tube (Figure 2).

![Figure 2. Classical anchorage of monostrand for flat slab [5]](image)

The tendons are individually anchored at each end and stressed one at a time. Strands are stressed and held by wedges fitting neatly inside a single anchor specially designed to retain the bursting forces and distribute the prestressing forces evenly into the concrete (Figure 2).

2. CONSTRUCTION PROCEDURE FOR UNBONDED POST-TENSIONING
In slabs with unbonded post-tensioning, the operations are normally carried out as follows:
1. Erection of slab supporting formwork
2. Fitting of end formwork; placing of stressing anchorages
3. Placing of bottom and edge reinforcement
4. Placing of tendons
5. Placing of top reinforcement and tendons
6. Concreting of the section of the slab
7. Removal of end formwork and forms for the stressing block-outs
8. Stressing of cables according to stressing program
9. Stripping of slab supporting formwork

3. DESIGN OF POST-TENSIONING FLOORS
Flat slabs should be analyzed using a proven method of analysis, such as grillage (in which the plate is idealized as a set of interconnected discrete members), finite element, yield line or equivalent frame. Appropriate geometric and material properties should be employed.
The design of post-tensioning floors differ from that of reinforced concrete in that forces are applied to the structure which have a positive effect on behavior. Prestress applied eccentrically to the slab produces deflections which are opposite in direction to those produced by self weight and variable loads, so the designer can vary the prestress to balance out any chosen proportion of these loads “load-balancing method” (Figure 3).

Due to the additional stiffness of the slab achieved by prestress, the thickness of the slab may be reduced without affecting performance or compliance with standards. Post-tensioning automatically allows for wider spacing of columns and the introduction of additional stiffening elements increases this facility still further. The limit states are placed in two categories [1]:

Ultimate Limit States (ULS) are those associated with collapse or with other forms of structure failure.

Serviceability Limit States (SLS) correspond to states beyond which specified service requirements are no longer met (deformation, cracking of concrete). For prestressed slabs the control of deflection is a main concern.

4. FREE TENDON LAYOUT
A reversed parabola is the most common gradient as it provides uniform distributed counterbalance. However the tendon layout is not that much of importance for floor slabs as for bridges. Tests have shown that there is no much difference for tendons that are supported only by two chairs over the support and are placed horizontally for most of the span [1]. This method “free tendon layout” spares chairs and reduces field labor costs and is less prone to field errors [2].

Figure 3. Illustration of transverse effect of prestress
The design should be oriented to maximize site efficiency for the indirect cost (less labor, formwork systems, cycle redundancy, etc) and accounts for almost 60% of the savings. The use of the free tendon layout concept, with very few control points and less saddles, is mandatory. This also forces the software to be able to model explicitly complex tendon shapes, not just parabolas (Figure 4).

There are substantial differences between the North American and the European practice with regard to the amount and the placement of mild reinforcement. The European Code does not permit any section without bottom reinforcement [1]. A further reduction is possible using the so called design concept “reinforced concrete with tendons”. The tendons utilize the permissible deflections. The ultimate limit state and crack control is utilized by mild reinforce only [2].

A defect of tendons is especially in the case of fire not relevant for the bearing capacity [3].

5. FIRE PROTECTION AND RESISTANCE

Research Report “Fire Resistance of Fiber-Reinforced, Reinforced and Prestressed Concrete”, was published by the Austrian Federal Ministry of Transportation, Infrastructure, and Technology in its series of publications on "Road Research" in 2004 [4].

When concrete is exposed to fire temperatures, constraining forces are generated within the reinforced concrete structure and the strength of the composite materials, i.e. concrete and steel, is gradually reduced. Explosive spalling of the inner concrete cover reduces the load-bearing capacity of the structure. Further investigations within the research program concerned the residual strength
of concrete and steel after a fire, the behavior of monostrands in the event of a fire, and the determination of the load-bearing capacity of a structure.

5.1. Behavior of Monostrands in Case of Fire

In case of fire monostrands are normally better protected (by a second reinforcement layer) than the reinforcing steel. In case of steep temperature gradients the protection layer may spall off more rapidly and the monostrand gets directly exposed to fire.

As a consequence, the corrosion protection is destroyed. If the concrete is not spalling off, the monostrand is durably protected by the concrete; it is heated slowly and the following processes can be observed:

The HDPE-sheath can resist temperatures ranging from 120°C to 140°C for a short duration and can resist temperatures of 100 °C for a longer duration. Higher temperatures will lead to a slow degradation (decompensation).

If for instance the prestressed strand is deflected inside the structure, it exerts lateral forces to the plastic sheathing which may result in perforation of the plastic sheathing even at temperatures of only 75°C (softening point).

The agent for corrosion protection (grease) remains unchanged up to a temperature of 180°C. If temperatures exceed this value, the presently used greases get destroyed. Because of the temperature increase the corrosion protection mass expands causing pressure in the plastic sheathing. Test results show pressures up to 9 bar inside the plastic sheathing, without causing any spalling of the concrete [4].

It can be stated that a long-term durability of monostrands after fire exposure is provided by the used corrosion protection systems up to maximum temperature of approximately 150°C in case of linear prestressing tendon layouts and of approximately 100°C in case of deflected prestressing tendon layouts. Even if the corrosion protection system is destroyed a short-term use of the monostrands is possible under the above described conditions [4].

In order to document the development of strength with continually increasing temperatures on a single test sample and in the full range of possible temperatures, a sample of prestressing steel with Ø9.4 mm (Figure 5) was put into a wire welding machine and exposed to a temperature increase to 950 °C over a length of 220 mm [4].

![Figure 5. A sample of prestressing steel, temperature increase to 950 °C [4]](image-url)
5.2. Fire Protection-Concrete Cover
Minimum concrete cover shall be provided in order to ensure an adequate fire resistance.
Reinforced concrete with tendons behaves like reinforced concrete. A defect of tendons is especially in the case of fire not relevant for the bearing capacity.

Nominal cover     2.0 cm  
Minimum cover of prestressing steel     3.0 cm

Unbonded post-tensioning und fire resistance class REI 90 [6]  
Prestressing steel (monostrand) in second reinforcement layer

Nominal cover     3.5 cm  
Minimum cover of prestressing steel     4.5 cm

The values can vary according to the standards of the various countries.

6. PRESTRESSED SLABS–BENEFITS FOR INVESTORS, ARCHITECTS, ENGINEERS AND CONTRACTORS
The application of unbonded post-tensioning systems provides many advantages compared to conventionally reinforced concrete structures.

Prestressing offers:
- Greater resistance to cracking and water seepage
- Slabs may be designed to be waterproof without the need for expensive membranes. This is particularly important in trafficable roof structures
- Increased movement joint spacing, up to 50-60 m

Increased design flexibility:
- Longer spans providing larger column-free areas
- Reduced deflection under maximum design load

Significant savings in material:
- Thinner slabs result in a lighter structure
- Reduces column and foundation loads
- Reduces building height can result in significant cladding savings in high-rise structures
- Less weight to be considered along the height of the building is of an advantage for earthquake design.

Speedier methods of construction:
- Simpler and more rapid fixing of tendons and reinforcing steel, no complex fixing details
- Fast formwork turnaround. Stressing is carried out 3-4 days after concrete is poured, after which, formwork and props may be removed, which also provides earlier access to other trades.
6.1. Concrete Core Activation in Floor Construction

Concrete core activation is a heating and cooling system based on simple physical principles. Concrete possesses a large capacity to store heat, and water functions well as a heat conductor.

The principle or the essence of concrete core activation is not new; by bringing the mass of a building up to a certain temperature, one takes care of the heating or cooling in comparison with the surrounding temperature. It uses the storage capacity of the concrete floor layers; night air and cool earth energy, or low-level heating energy can all be stored in the concrete floor to create a low-cost system [9].

The activated layers absorb the heat from the room in order to cool the room during operating periods. The heat is subsequently released during the time remaining. The complete concrete floor construction is cooled or heated through an integrated piping system, which is evenly woven, before the concrete is casted (Figure 6).

The system is simple to install, from the assembly of the water supply pipes onwards. Pre-fabricated modules are installed to the concrete layer in externally supplied reinforcement casings. The cooling and heating functions are both possible using the same pipe distribution system and this is fully integrated into the ceilings making it completely invisible (Figure 6).

If the daytime sees high temperatures, then the system is subjected to its greatest loads. The cooling floor reduces the comfortable temperature range to between 21°C und 26°C. During the night time, the water circulating through the system releases the heat back out through the floor [7].

Water reliably transports the cooling and heating energy, meaning that the temperature is not prone to swings. Water is for heating and cooling, air is just for ventilating - the energy-saving slogan is certainly justified.
The harsh construction site environment, particularly during the structural build phase, as well as the decades of use to which the pipes are subsequently exposed, require a highly durable raw material.

The cross-linking has numerous advantages: it results in a material which is dimensionally stable, non-sensitive to stress cracks and highly durable.

The concrete core activation system takes care of the basic load, and in the case of optimally configured buildings, the entire cooling and heating requirements. Under these circumstances radiator installations beneath the windows and false ceilings for ventilation channels become entirely unnecessary [7].

7. COMPLETED STRUCTURES
The benefits of using post-tensioning for the more widespread practical applications in concrete buildings and the advantages of using post-tensioning with unbonded tendons, concerning constructive aspects, economy and structural behavior will be illustrated by the presentation of several commercial building and parking structures.

8. CONCLUSIONS
The unbonded post-tensioning enabled the designer to limit deflections under service load conditions, so the designer could vary the prestress to balance out any chosen proportion of self weight and variable loads. Due to the additional stiffness of beams and slabs achieved by prestressing, the thickness of beams and slabs could be reduced. Post-tensioning allowed also for wider spacing of columns. Particularly with regard to the low ambient temperature post-tensioning with unbonded tendons offered a special advantage. Stressing procedure could take place without grouting operation.

The concrete core activation system takes care of the basic load, and in the case of optimally configured buildings, the entire cooling and heating requirements. Under these circumstances radiator installations beneath the windows and false ceilings for ventilation channels become entirely unnecessary.

Owners, contactors and architects benefited from concrete core activation system and from thinner beams and slabs as well as from a weight reduction in floor constructions. Moreover, post-tensioning allowed earlier stripping of formwork shortening overall construction time. For high durability tendons provided crack-free beams and slabs. In addition, Post-tensioned beams and slabs offered increased seismic resistance.

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EXPERIENCE GAINED DURING DESIGN AND CONSTRUCTION OF THE JEGIN RCC DAM IN IRAN

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ABSTRACT
Construction of the Jegin RCC large dam in Iran has been completed in January 2007. Jegin dam with 78 meter height and 400,000 cube meter in concrete volume is located in a remote area at the South-east of the Country, in a region with very high temperatures and extremely difficult working conditions. Although the original design was already a straight gravity RCC dam type, when performing the definitive construction design some improvements were introduced in order to achieve an “RCC- friendly” design. As a consequence the new design, together with the optimization of the construction processes, enabled the Contactor to place the RCC rapidly and efficiently, which led to a better execution quality, a schedule reduction and lower cost. This paper deals with the original and actual design criteria and construction of RCC works including development of mix design and full scale trails. RCC placement, compaction and quality control procedures and other relevant aspects are also discussed.

1. INTRODUCTION
The Jegin Roller-Compacted Concrete (RCC) dam is the first large RCC dam in Iran; a smaller RCC dam that is essentially the core of a spillway was completed in 2002 on the Karkheh River in southwest Iran. Jahgin is 78 m high, 220 m long at the crest and was contain approximately 400000 m³ of concrete of which some 270000m³ was RCC.

The dam is in southern Iran some 350 Km east of Bandar Abbas and some 107 Km north of Jask that is on the coast (see Figure 1). The conditions at the Jegin site are very challenging, probably as challenging as at the site of any RCC dam completed to date. There is a total lack of water and throughout the year the temperatures are high, particularly in the summer when they can be in excess of 50 °C for significant periods. During 2002 there was no rain at all on the site and the maximum temperature was 54 °C. There were also significant floods; the design flood is in excess of 10 730 m³/sec.

Prior to the start of the trial mix programme, an extensive search was made for sources of supplementary cementitious materials that were reasonably close to the Jegin site, two natural Pozzolans were found together with an air-cooled blast furnace slag and a copper slag.

In addition it was ascertained that a low lime fly ash from India was being
imported into the Country. None of the Iranian materials had ever been used in concrete before let alone in a structure as important as Jegin dam. The sources of the Iranian supplementary cementitious materials together with the two cement suppliers, both of whom added natural pozzolans in their cements, are shown in Figure 1.

2. CONSTRUCTION MATERIALS

2.1. Portland Cements
Two ASTM Type-II Portland cements were investigated during the trial mix programme; the preferred cement from Hormozgan and alternative cement from Kerman (see Figure1).

2.2. Pozzolans
An extensive search was made in southern Iran for suitable pozzolans. The following five possible materials were located;
- a natural Pozzolan from Khash;
- a ground slag from Esfahan (it was originally thought that this was a ground-granulated blast-furnace slag (GGBFS), but it was eventually ascertained that the slag was air cooled rather than granulated);
- an imported low lime fly ash from near Mumbai in India;
- a natural pozzolan from Sirjan;
- a ground copper slag from near Jiroft;

In addition, some fine river sand was also investigated, to see if it could be used as 'fine' in the mixes to reduce the quantity of natural pozzolan which might be required. The location of two cements and various potential pozzolans relative to the Jahgin site are shown in Figure 1.


Figure 1. Location of potential cementitious materials sources in southern Iran
2.3 Aggregates
The sandstone aggregates were obtained from gravel borrow pit in the reservoir area. Considering the Jegin dam site condition, it has been decided to transport RCC with vacuum chute system. In order to avoid segregation of the aggregate, 75mm maximum size aggregate given in the primary study has been changed by engineer to 38mm. It's to be mention that, the difference in cementitious material requirements for mixture with maximum size aggregate from 38mm to 76mm is less in RCC than in conventional concrete. An aggregate plant was designed to crush the oversize material and four size of aggregate were used; a partially crushed 20 to 5mm material and two fine aggregates, the first crushed and second uncrushed (5).

3. TRIAL MIX PROGRAMME–STAGE 1
The trial mix programme at Jegin was carried out in two stages. The objective of the first stage was to assess the performance of all the potential cementitious materials to see which could achieve the strengths required at Jegin dam. Three of the five pozzolans were considered to be 'base' materials: the natural pozzolan from Khash; ground slag from Esfahan and, the low-lime fly ash from India.

3.1. Optimum Workability/Water Content
After gradation has been optimized for each of the combinations of the base cement and the base supplementary cementitious materials, the RCC was visually optimized by using a "Standard" set of mixture proportions:-
- 100 + 100 (cement + supplementary cementitious materials) in the case of the natural pozzolan:
- 50 + 150 in the case of the ground slag;
- 120 + 100 in the case of the low-lime fly ash
In all three cases the optimum workability was in the range of a Loaded Ve Be (ASTM C1170) time of circa 14 to 18 sec with an acceptable range (for the laboratory tests) of between 10 and 25 sec. The optimum water contents commensurate with these workabilities were:
- 140 Kg/m\(^3\) for the Khash natural pozzolan (for the 100 + 100 mixture proportions);
- 135 Kg/m\(^3\) for the Esfahan ground slag (for the 50 + 150 mixture proportions)- this was later found to be a little low;
- 127 kg/m\(^3\) for the Indian low-lime fly ash (for the 120+100 mixture proportions).

3.2. Mix Proportions
Two RCCs were to be used in the dam body at Jegin:
- RCC1 was to be used near the upstream face of the dam where the dynamic loading was greatest. This was to have a characteristic cylinder compressive strength of 200 kg/cm\(^2\) at the design age of 365 days: a target (average) strength of 220 kg/cm\(^2\) was defined
- RCC2 for the majority of the dam body; this was to have a characteristic
cylinder compressive strength of 120 kg/cm² at the design age of 182 days; a
target (average) strength of 145kg/cm² was defined.

Three sets of mix proportions were designed to obtain the requirements of RCC as
shown in Table 1. Most of the mix proportions have a total cementitious content of
195 kg/m³. The results of the stage-1 programme showed that there were
essentially only two pozzolans which were satisfactory for the RCC Jahgin, both in
terms of their strength development and also in terms of practically. These were the
natural pozzolan from Khash and the Indian low-lime fly ash. The Sirjan natural
pozzolan had a very similar performance to the Khash natural pozzolan, but there
were potential supply and transportation problems.

<table>
<thead>
<tr>
<th>Mix No*</th>
<th>Cement + natural Pozzolan + water</th>
<th>Mix No.</th>
<th>Cement+ground slag+water</th>
<th>Mix No.</th>
<th>Cement + low-lime fly ash + water</th>
</tr>
</thead>
<tbody>
<tr>
<td>HK.Mix1</td>
<td>70 + 125 + 139</td>
<td>HE.Mix1</td>
<td>30 + 165 + 139</td>
<td>HI.Mix1</td>
<td>45 + 150 + 127</td>
</tr>
<tr>
<td>HK.Mix2</td>
<td>95 + 100 + 140</td>
<td>HE.Mix2</td>
<td>40 + 155 + 140</td>
<td>HI.Mix2</td>
<td>70 + 125 + 128</td>
</tr>
<tr>
<td>HK.Mix3</td>
<td>120 + 75 + 142</td>
<td>HE.Mix3</td>
<td>50 + 145 + 142</td>
<td>HI.Mix3</td>
<td>95 + 100 + 129</td>
</tr>
<tr>
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<td>145 + 50 + 144</td>
<td>HE.Mix4</td>
<td>70 + 125 + 144</td>
<td>HI.Mix4</td>
<td>120 + 75 + 131</td>
</tr>
<tr>
<td>HK.Mix5</td>
<td>170 + 25 + 146</td>
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<td>90 + 105 + 146</td>
<td>HI.Mix5</td>
<td>145 + 50 + 134</td>
</tr>
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<td>195 + 0 + 146</td>
<td>HE.Mix6</td>
<td>110 + 85 + 146</td>
<td>HI.Mix6</td>
<td>170 + 25 + 137</td>
</tr>
<tr>
<td>HK.Mix7</td>
<td>240 + 0 + 150</td>
<td>HE.Mix7</td>
<td>130 + 65 + 150</td>
<td>HI.Mix7</td>
<td>150 + 30 + 135</td>
</tr>
</tbody>
</table>

*The first letter of the Mix. No. refer to cement (H = Hormozgan) and the second the pozzolan
(K = Khash natural pozzolan), E = Esfahan ground slag, I = Indian low-lime fly ash

3.3. Development of Strength with Age of the Stage–I Mixes
3.3.1. Khash Natural Pozzolan
Six mixes were designed containing the Khash natural pozzolan (and Hormozgan
Type–II cement), all with a total cementitious content of 195 kg/m³. The natural
pozzolan content varied from 0 Kg/m³ in HK. Mix6 (the H refers to the cement, Hormozgan type II, and the K to the supplementary cementitious material, Khash
natural pozzolan) to 125 Kg/m³ in KH. Mix 1. The development of strength with
age of the HK series is shown in Figure 2. All the mixes are developing strength
much in line with expectation and they might be expected to continue to do so.

![Figure 2. Development of strength with age of the Stage-I HK (Hormozgan cement +
khash natural Pozzolan) series of mixes](image-url)
A comparison was also made between the HK Series and the KK series of mixes (Kerman Type – II cement and Khash natural pozzolan). The strengths of the mixes containing the two different cements are very similar and it was determined that the two cement might be interchangeable.

On the basis of the data, it has been concluded that the Khash natural pozzolan could successfully be used as a cementitious material in the RCC at Jegin Dam.

### 3.3.2. Indian Low–Lime Fly Ash

Seven mixes have been designed containing the Indian flash. The development of strength with age of the HI series (Hormozgan Type – II cement and Indian fly ash) is shown in Figure 3.

![Figure 3. Development of strength with age of the Stage-I HI (Hormozgan cement + Indian low-lime fly ash) series of mixes](image)

The development of strength of HI.Mix2, HI.Mix3 and HI.Mix4 is very similar to the pattern found at Olivenhain dam in the USA (Pauletto & Dunstan 2003), where RCCs with cylinder compressive strengths of 40 kg/cm² to 70 kg/cm² at an age of 7 days achieved, strengths of 120 to 180 kg/cm² at 91 days and 230 to 330 kg/cm² at an age of one year.

As with the HK Series, apart from some scattered results with H.Mix4, HI. Mix6 between 91 and 182 days, all the mixes are developing strength in line with expectations.

A comparison was also made between the HI Series and the KI Series of mixes (Kerman Type-II cement and Indian fly ash). As with the KK Series the mixes containing the two different cements are developing strength along similar lines.
The two type – II Portland cement (Hormozgan and Kerman) seem to have similar characteristics when used in association with both the supplementary cementitious materials at Jegin.

It would be possible to design an RCC containing the Indian fly ash (it has very good fresh properties) to achieve the strength requirements at Jegin, but given that the material has to be imported, it was concluded that it would only be sensible to use the material if there was a serious problem with the supply of the Khash natural pozzolan.

3.3.3. Esfahan Ground Slag
Initially it was thought that the Esfahan slag was granulated when discharged from the blast furnace. It is now understood that the material is air cooled. No ground air–cooled blast-furnace slag has been used in an RCC dam to date.

Seven mixes were designed containing the Esfahan GBFS (ground-granulated blast-furnace slag) but unfortunately the performance was found to be very poor. On the basis of the data to date and considering the difficulty to mill the material and the 1450 km it would have to be transported (see Figure 1), it has been concluded that the use of the Esfahan GBFS as a cementitious material in the RCC at Jegin would seem to be unlikely and it is no longer being considered for use in the dam.

3.4. Supplementary Stage 1 Programme
Following the 91 days results of the stage 1 trial mix programme, a supplementary programme was designed so as to have a preliminary look at the RCC1 mix and the leveling concrete, together with some potential retarders. Only Khash natural pozzolan was used as a pozzolan in this Series.

3.4.1. RCC1
Three rcc1 mixes were tried in the supplementary programme: 175+50 (cement + pozzolan), the expected mix; 160 + 65 (a lower-strength mix); and, 190 + 35 (a higher- strength mix). All had a total cementitious content of 225 kg/m³. The mixes were designed to achieve target strength of 225 kg/cm² at the age of one year. To achieve such strength with a 100 percent Portland cement RCC, the same cement content of 225 kg/cm² would be required (HK. Mix6 and HK.Mix7) and as there would be little strength gain after an age of 56 days.

It is expected that RCC1-3 (190+35) almost achieve the target cylinder compressive strength of 225 kg/cm² at the design age of 365 days, and the supplementary Stage 1 programme showed that it was possible to achieve the specified strength even without an admixture, at a rather high cementitious content.

3.4.2. Leveling Concrete
Three leveling concretes were designed for the supplementary programme: 160+120+195 (cement + pozzolan + water) the expected mix; 145+135+195 (a lower-strength mix); and, 175+105+195 (a higher-strength mix). All the mixes had a total cementitious content of 280 kg/m³, and all were designed for a slump of 35
10 50mm and for a 182 day cylinder compressive strength of about 145kg/cm². The total cementitious content is necessary rather high, because of the very high water demand of the Jahgin aggregates.

### 3.4.3. Retarder
Admixtures from five or six different suppliers tested during the supplementary programme. After a number of preliminary tests, two retarders were chosen for future study: Sitka Retarder and Forsook Conplast R. After an extensive further study Conplast R was chosen for use in the RCC for Jegin.

### 4. TRIAL MIX PROGRAMME – STAGE 2
#### 4.1. Aggregate
The Stage 2 aggregate was obtained from the same source as the Stage 1 aggregate, but the shape was rather better, this was a function of the development of the new aggregate plant.

#### 4.2. Optimization of Mixture Proportions for Stage 2 Trial Mix Programme
Following the stage-I trial mix programme, it was agreed that the natural pozzolan from khash be used in the RCC for the Jegin. The RCCs containing Khash natural pozzolan will have the properties, both fresh and hardened, required for the dam. It was decided that the following mixes be studied in detail in the Stage-II trial mix programme.

The necessary dosage of retarder to produce an initial set of 15 to 20 hours at different temperature; an RCC2, and RCC1 and a leveling concrete with both the Khash natural pozzolan and the Indian fly ash as detailed in Table 2; Grout Enriched Vibratable RCC for the interface (upstream and downstream faces and against the abutment).

It has been agreed that the khash natural pozzolan should be the preferred option and that the Indian fly ash should be a back up.

Three batches, from each of which eight cylinders were manufactured, were made for each of the mixes in Table 2.

The cylinders are being tested at ages of 7, 14, 28, 56, 91, 182, 365 and 1000 days. The development of strengths with age of specimens manufactured (in Stage–II trial mixes) with Hormozgan Type–II cement and khash natural pozzolan as well as Indian fly ash are shown in Figures 4 and 5.

#### 4.3. Recommended Mixture Proportions Used in Jegin Dam
Three mixes were required for the main dam at Jegin:

- RCC1 designed for required cylinder compressive strength of 200 kg/m² at the design age of a year;
- RCC1 designed for required cylinder compressive strength of 120 kg/m² at the design age of a 182 days;

A leveling concrete placed against the foundation as a platform for the RCC having a required cylinder xcompressive strength of 200 kg/cm² at the design age of a year.
Table 2: Suggested mixture proportions for the concrete studied in the Stage-II trial

<table>
<thead>
<tr>
<th>Mix</th>
<th>Cement (kg/m³)</th>
<th>Pozzolan (kg/m³)</th>
<th>Slump (mm)</th>
<th>L. V Be time (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Khash natural pozzolan</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>HK.RCC.1-1</td>
<td>150</td>
<td>75</td>
<td>10 - 15</td>
<td></td>
</tr>
<tr>
<td>HK.RCC.1-2</td>
<td>165</td>
<td>60</td>
<td>10 –15</td>
<td></td>
</tr>
<tr>
<td>HK.RCC.1-3</td>
<td>180</td>
<td>45</td>
<td>10 - 15</td>
<td></td>
</tr>
<tr>
<td>HK.RCC.2-1</td>
<td>90</td>
<td>105</td>
<td>12 - 18</td>
<td></td>
</tr>
<tr>
<td>HK.RCC.2-2</td>
<td>105</td>
<td>90</td>
<td>12 - 18</td>
<td></td>
</tr>
<tr>
<td>HK.RCC.2-3</td>
<td>120</td>
<td>75</td>
<td>12 - 18</td>
<td></td>
</tr>
<tr>
<td>HK.Levelling.1</td>
<td>135</td>
<td>145</td>
<td>10 - 40</td>
<td></td>
</tr>
<tr>
<td>HK.Levelling.2</td>
<td>150</td>
<td>130</td>
<td>10 - 40</td>
<td></td>
</tr>
<tr>
<td>HK.Levelling.3</td>
<td>165</td>
<td>115</td>
<td>10 - 40</td>
<td></td>
</tr>
<tr>
<td>HK.Levelling.4</td>
<td>165</td>
<td>85</td>
<td>10 - 40</td>
<td></td>
</tr>
<tr>
<td>HK.Levelling.5</td>
<td>180</td>
<td>70</td>
<td>10 - 40</td>
<td></td>
</tr>
<tr>
<td>HK.Levelling.6</td>
<td>195</td>
<td>55</td>
<td>10 - 40</td>
<td></td>
</tr>
<tr>
<td>Indian low-lime fly ash</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>HI.RCC.1-1</td>
<td>95</td>
<td>125</td>
<td>10 - 15</td>
<td></td>
</tr>
<tr>
<td>HI.RCC.1-2</td>
<td>110</td>
<td>110</td>
<td>10 –15</td>
<td></td>
</tr>
<tr>
<td>HI.RCC.1-3</td>
<td>125</td>
<td>95</td>
<td>10 - 15</td>
<td></td>
</tr>
<tr>
<td>HI.RCC.2-1</td>
<td>70</td>
<td>125</td>
<td>12 - 18</td>
<td></td>
</tr>
<tr>
<td>HI.RCC.2-2</td>
<td>85</td>
<td>110</td>
<td>12 - 18</td>
<td></td>
</tr>
<tr>
<td>HI.RCC.2-3</td>
<td>100</td>
<td>95</td>
<td>12 - 18</td>
<td></td>
</tr>
</tbody>
</table>

Figure 4. Development of strength with the age of the RCC-1, RCC-2 mixes containing Khash natural pozzolan

Figure 5. Development of strength with the age of the RCC-1, RCC-2 mixes containing low - lime Fly ash
In order to decide the final mixes used in the dam body, it was of interest to compare the results from the cofferdam RCC with the strengths of RCC2-3 from the Stage-II programme and with HK.Mix3 from the Stage-I programme (see Table 3). As can be seen in the Table 5, RCC2-3 has higher long term strength than HK.Mix3 (because of water reduction of the retarder). However the strengths of the specimens manufactured from the RCC placed in the coffer dam are significantly higher than those of RCC2-3, by about 15% at the design age of 182 days.

<table>
<thead>
<tr>
<th>Cylinder compressive strength (kg/cm²)</th>
<th>7 day</th>
<th>28-day</th>
<th>91 day</th>
<th>182 day</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cofferdam</td>
<td>88</td>
<td>129</td>
<td>179</td>
<td>182</td>
</tr>
<tr>
<td>Stage-II RCC2-3</td>
<td>64</td>
<td>108</td>
<td>142</td>
<td>164</td>
</tr>
<tr>
<td>Stage-I HK.Mix3</td>
<td>73</td>
<td>104</td>
<td>127</td>
<td>145</td>
</tr>
</tbody>
</table>

### 4.3.1. Optimization of Mixture Proportions Of RCC2

In the stage –II of the trial mix programme, three mixes were tried, RCC2-1 (90+105), RCC2-2 (105+90) and RCC2-3 (120+75), all with Hormozgan Portland cement, khash natural pozzolan and Fosroc Conplast R as a retarder/water reducer, together with a free water content of circa 130 kg/m³.

The results of compressive testing of three mixes (see Figure 4) have been compared to the results of testing of the HK series from Stage-I of the programme presented in Figure 2. There is a reasonable correlation, so much so that the best-fit relationships did not change significantly at any age. At the design age of 182 days, to achieve the design strength of 145 kg/cm², the cement and natural pozzolan contents required were almost exactly those of RCC2-2 (and HK.Mix3). Given the significant difference between the strengths achieved in the field and the laboratory with RCC2-3 (120+75), it might be expected that there could be a similar difference with RCC2-2 and thus a reasonable margin between the required strength and that achieved during the full-scale placement. It has been agreed that RCC2-2 will be used in the dam itself subject to a satisfactory full-scale trials.

### 4.3.2. Optimization of Mixture Proportions Of RCC1

None of the three mixes, RCC1-1 (150+75), RCC1-2 (165+60) or RCC1-3 (180+45) tried in Stage-II of the trial mix programme achieved the design strength at an age of 182 days and as there does not seem to be significant increase in strength between the ages of 182 and 365 days, none of the mixes is likely to achieve the design strength at the design age at 365 days.

The strengths of the stage-II RCC1 mixes, all of which had a total cementitious content of 225 kg/m³, have been plotted in Figure 4. The results are rather scattered (as might be expected with a set of data being the average of three RCC2 mixes. In the Figure it can again be seen that none of the mixes (apart from the all-Portland cement mix) achieved the design strength (225 kg/cm²) at an age of 182 days. In order to achieve the required strength of RCC1, the total cementitious content will
have to be increased. There is a very reasonable correlation between the Portland cement content of the all cement mixes and their long-term strength (see Figure 2), e.g. HK.Mix6 (and KK.Mix6) (cement content=195 kg/m³) with a long-term cylinder compressive strength of circa 200 kg/cm² and HK.Mix7 (cement content=240 kg/cm³) with a long-term cylinder compressive strength of circa 245 kg/cm². In order to achieve the design strength of 225 kg/cm², it was decided that the mixture proportions of RCC1 be 160+90+130 (cement + pozzolan + water).

Coincidently this has the same natural pozzolan content as RCC2. Thus in order to change from RCC2 to RCC1, a further 55 kg/m³ of cement is added to RCC2. Test results obtained from leveling concrete at Stage –II indicate that a leveling concrete having mixture proportions of 185+105+150 (cement + pozzolan + water) could be used against the foundation.

5. COFFER DAM
5.1. Preliminary Full-Scale Trial
In early March 2003, preliminary full–scale trial was conducted on the site (see Figure 12). The trial showed that an RCC containing Hormozgan Portland cement and Khash natural pozzolan was cohesive, and could be placed in very high air temperatures without segregation or any other major problem. 40 days after completion of the trial, 100-mm diameter cores were extracted from the trial section. In spite of the very early age and the small diameter the cores through the suitably-retarded RCC were very good. Following the successful cores, placement of RCC started in the cofferdam (see Figure 13), placement of 20000 m³ was used to train all the personal involved with the project, to refine the construction methodology and to further prove the maximum proportions of the concretes used in the dam itself (3).

The final full-scale trial was conducted at the top of coffer dam using the final mixtures proportion for RCC1, RCC2 and GEVR. The extracted 45 days cores are shown in Figures 8 and 9.

Figure 6. Photograph of the initial full Scale trial at Jegin dam
Figure 7. Photograph of RCC placement in coffer dam at Jegin with chute in background
6. DESIGN OF THE DAM

Before construction of main dam and in agreement with all parties involved in Jegin dam, a number of modifications were made to the original design of the dam so as to make construction easier and thus more rapid and of a higher quality. There were two main changes: first, all the galleries were made either horizontal or vertical. The latter were shafts and were separated in to 'man access' or 'equipment access'. The second main change was to move the bottom outlets to the left abutment, and to construct them within traditional concrete blocks, thus separating them from the RCC placement in the main body of the dam. The bottom outlets discharged on the concrete steps before joining the river downstream of the main stilling basin, see Figure 10.

A further change to the construction procedure was to remove the traditional facing concrete and replace it with GEVR (grout-enriched vibratable RCC). This had been tested successfully in the coffer dam. The dam thus relied on the RCC itself for impermeability (5).

7. CONSTRUCTION OF DAM

7.1. Concrete Production

The geometry of the dam site and the relatively reduced space to locate the installations were the additional difficulties for the design of the plants. Figure 11 shows the final layout of the concrete and cooling plants at right abutment at Jegin dam site. The supply of the aggregate from the main stockpiles was done by 25-tonne dump trucks. Those trucks downloaded on receiving hoppers with a capacity from 4 to 6 hours average production. The hoppers were connected with the batching plant by automatic control systems that could feed the material into the
weighing hoppers. The concrete plant had a nominal capacity of 2*160 m³/hour, and consisted in two plants each with two 2,250 m³ capacity horizontal twin-shaft bath type mixers. The cooling plant combined with a water chiller plant to cool the mixing water to 4 degrees centigrade and the flak ice plant with total capacity of 180 tone of ice per day. The cementitious materials were transported from the cement factories to the site by road, and the natural pozzolan from Khash to the site were deliver by big-bag. In order to have a storage for cementitious materials to supply the production of more than one week, four large 800-tonne silos and eight further small 60 tone silos were erected closed to the concrete plants. A combination of both pneumatic transportation systems and screw conveyors is used to feed the receiving hoppers of each mixer.

7.2. RCC Transportation
The transportation systems selected for the transportation of concrete was a combination of 45 degree inclined steel chute and trucks on the dam. Still chute was built up in such a way that it could be removed as the dam was raised. As shown in the Figure 12, the chute was supported on the excavation on the right abutment. A surge hopper at the top of the chute regulated the flow of concrete out of mixers. The chute ended in a swinging element that moved to change its loading position from one truck to the other. Figure 12 shows a view of the chute that has been used for the transportation of roller compacted concrete in Jegin dam (1). The chute has worked successfully, and the high-paste concrete in the RCC mix has not shown any segregation. The flexible cover on the top of the steel chute has been working as a protection against solar radiation, and thus avoiding excessive drying of the mix in its way down to the placement.

![Figure 11. View of vacuum chute used at Jegin dam and general view of site lay out](image)

7.3. RCC Placement
The RCC was transported from the delivery point of the steel chute to the point of placement on the lift by dump trucks. Dozer type D4 was then used to spread the RCC and single-drum 11 tones vibratory roller were used for compaction (see
Figure 12). However in areas where the access for the larger roller was difficult or closed to the forms, the compaction was done by small double-drum 3.5 tones roller.

A special type of cantilever formwork was used to form the faces of the dam. GEVR (grout-enriched vibrated RCC) was used against the forms instead of the conventional immersion vibrated concrete.

The great advantage of the GEVR is that just a water/cementitious material grout applied onto the surface before the RCC is spread on top of it, and then consolidated by immersion vibrators.

Transverse joints were formed inserting galvanized sheets into the fresh fully compacted RCC that are left in place acting as joint inducers. PVC membranes were inserted in the GEVR in the upstream face of the dam and downstream spillway sections to seal the transverse joints. The surface of compacted RCC was continuously cured by means of low-pressure water jets creating a thin nebula on top of the surface. Due to the extreme high temperatures on site, this was a major activity at Jahgin. Depending on the time elapsed between consecutive layers and weather conditions, different steps of treatments of the horizontal joints were defined. Cleaning of the surfaces was done by brooms and high capacity vacuum trucks.

Figure 12. View of spreading and compaction of RCC in Jegin dam

Conclusions

- A suitable supplementary cementitious material has been found in Iran for use in the RCC dam at Jegin;
- In spite of the Khash natural pozzolan having never been used in concrete before, suitable concretes containing the material have been designed and tested in the laboratory;
- Both the construction methodology and the RCC have been successfully tested at full-scale.
- In spite of the very challenging conditions at the Jegin dam site, a suitable methodology has been developed for the construction of the dam and a very forgiving RCC designed.
- The plant layout and the construction methods have been optimized and
adopted according to the changes incorporated to the design.

- We believe that with these changes and an efficient management of all activities involved, a successful RCC experience have been achieved in our project.

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APPLICATION OF SELF CONSOLIDATING CONCRETE IN CONSTRUCTION INDUSTRY

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ABSTRACT
Self consolidating concrete (SCC) is a novel concrete which uses in many structures in all over the world. It is a concrete which flows under own weight and completely fills the mold, passes from congested reinforcement without any bleeding or segregation. This type of concrete contains high range water reducing admixture and often viscosity modifying admixture also more fine aggregate compared to conventional concrete. It can reduce labor requirement, increase strength, durability and productivity, eliminate noise and hazard. For these good specifications, SCC uses in improper reach concreting, dense reinforcement elements, thin layer concrete construction and so on. Mahab Ghodss consultant engineering applied SCC in some Dams and tunneling projects. This article presented some executive experience of three projects which some structure of them constructed by SCC successfully. Finally some recommendation for mix design and whole scale construction SCC in site are proposed.

Keywords: self consolidating concrete, concrete, mix proportion, rheological tests

1. INTRODUCTION
Self Consolidating Concrete (SCC) is a concrete that able to flow under its weight and completely fill the formwork, even in the presence of dense reinforcement, without the need of any vibration, while maintaining homogeneity [1]. It was first developed in Japan in 1986 [2]. SCC is used mainly for repair applications and for casting concrete in restricted areas [3]. Though showing good performance, SCC used in Japan, America and Europe and many countries in buildings, bridges, tunneling and other applications [4]. SCC can accelerate placing and reduce labor requirement, increase strength, durability and productivity, eliminate noise and hazard, but little increasing in material cost [3]. Mahab Ghodss consultant engineering as a pioneer in application of novel technology uses SCC in some project such as Karoon III Dam and power plant, Resalat tunnel and Gotvand dam and power plant. In this article specification of these SCC concrete are presented.

2. APPLICATION OF SCC IN KAROON III DAM AND POWER PLANT
Project of Karoon III Dam and power plant sited in 28 Km east of Izeh city. The main objects of this project are:
- Supply 4172 Gw.hr power in year
Supply agricultural water for 120 Km² land
- Flood control of Karoon river

SCC is used in entrance structure of orifice in this dam. This structure located out of reach of cable crane and because the level of around blocks is lower than this block, pumping concrete rate is low and so, it might create cold joint in the structure. Also this structure had congested reinforcement. For whole above reasons, this structure must construct with SCC. Figure 1 shows the plan of this structure.

The cement used was type II. Maximum size of aggregates was 19 mm, sand to total aggregates ratio, water to cement ratio and paste amount were 0.58, 0.42 and about 300 respectively.

A high range water reducing admixture which is type A, F according ASTM C494 [5] with density of 1050 also was used. The mixture proportions are presented in Table 1.

<table>
<thead>
<tr>
<th>Description</th>
<th>Coarse Agg.</th>
<th>Fine Agg.</th>
<th>Cement</th>
<th>Water</th>
<th>HRWR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit</td>
<td>Kg</td>
<td>Kg</td>
<td>Kg</td>
<td>m³</td>
<td>% by weight of cement</td>
</tr>
<tr>
<td>Amount</td>
<td>775</td>
<td>1080</td>
<td>380</td>
<td>0.160</td>
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</tr>
</tbody>
</table>

For shortening concrete placing area and avoiding large movement of concrete in the forms, temporary construction joints were applied.

V- Funnel and T-50 tests were drawn for fresh concrete according EFNARC [1]. Figure 2 and 3 shows the V-Funnel and T-50 tests apparatus. Compressive strength of 28 days concrete was tested according ASTM C39 [6]. Table 2 shows average of test results. The orifice concreting is shown in Figure 4.
Figure 2. V-Funnel tests apparatus

Figure 3. T-50 tests apparatus

Table 2: Average of test results

<table>
<thead>
<tr>
<th>Description</th>
<th>T-50</th>
<th>V-Funnel</th>
<th>Compressive strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>Amount</td>
<td>3.5</td>
<td>9</td>
<td>35</td>
</tr>
</tbody>
</table>

Figure 4. Orifice concreting
3. APPLICATION OF SCC IN RESALAT TUNNEL

Resalat tunnel is a key project which relating east and west section of Resalat highway. In this project B section of lining was built lastly and it hadn't proper access. Construction of this section with conventional concrete might cause many voids and this structure would be very weak. So we had to create SCC to build this section. Figure 5 shows the section of the tunnel lining.

In situ piles were constructed by SCC because of improper reach, congested reinforcement, and lack of vibration and also long height of concrete shooting. The SCC pile is shown in Figure 6.

Maximum size of aggregates was 20mm and sand to total aggregates ratio, volume of paste and water to cement ratio were 0.69, about 360lit/m³ and 0.42 respectively. High range water reducing admixture, viscosity modifying admixture were used for achieving SCC with good rheology. Also for increasing stability of fresh concrete and resistant against segregation and bleeding inert filler was added to mixture. For compensating shrinkage strain of concrete an expanding admixture also added to SCC mixture. The mixture proportions are shown in Table 3.

<table>
<thead>
<tr>
<th>Description</th>
<th>Coarse.Agg</th>
<th>Fine.Agg</th>
<th>Cement</th>
<th>Water</th>
<th>Filler</th>
<th>HRWR</th>
<th>VMA</th>
<th>EX.A</th>
</tr>
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<tbody>
<tr>
<td>Unit</td>
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<td>Kg</td>
<td>Kg</td>
<td>Kg</td>
<td>Kg</td>
<td>Lit</td>
<td>lit</td>
<td>lit</td>
</tr>
<tr>
<td>Amount</td>
<td>530</td>
<td>1200</td>
<td>400</td>
<td>167</td>
<td>100</td>
<td>3.6</td>
<td>2.0</td>
<td>2.7</td>
</tr>
</tbody>
</table>
Slump flow and V-Funnel tests were drawn for fresh concrete according EFNARC[1]. The Result of tests is presented in table 4.

<table>
<thead>
<tr>
<th>Description</th>
<th>Slump flow</th>
<th>V-Funnel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Amount</td>
<td>680</td>
<td>9</td>
</tr>
</tbody>
</table>

### 4. APPLICATION OF SCC IN GOTVAND DAM AND POWER PLANT

Gotvand dam and power plant project located in 25 Km north of shooshtar city. The major objects of this project are:
- Supply 4250 Gw.hr power in year.
- Supply agricultural water
- Flood control of Karoon river

SCC is used in some structure of this project such as around of steel lining, around of penstocks, around of spiral cases, paving of seal beam and etc. In this article concreting around of steel lining is described.

In this structure because of erection of steel pipes and multiplicity of stiffeners, there is no proper reach to molding. In such cases application of conventional concrete may cause poor concrete with many entrapped air and voids. For preventing this disease, SCC is used to concreting the structure.

Maximum size of aggregates was 19mm and sand to total aggregates ratio, volume of paste and water to cement ratio were 0.58, about 310lit/m³ and 0.43 respectively. A high range water reducing admixture also was used to increase filling ability of concrete. The mixture proportions are shown in Table 5.

Because fine aggregate used in this project had proper grading, additional fillers or mineral additive wasn't required.

The slump flow test [1] and visual inspection were drawn for fresh concrete. The average of slump flow test was 675mm. Slump flow spread is shown in Figure 7.
Table 5: Mixture proportions

<table>
<thead>
<tr>
<th>Description</th>
<th>Coarse Agg.</th>
<th>Fine Agg.</th>
<th>Cement</th>
<th>Water</th>
<th>HRWR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit</td>
<td>Kg</td>
<td>Kg</td>
<td>Kg</td>
<td>$m^3$</td>
<td>% by weight of cement</td>
</tr>
<tr>
<td>Amount</td>
<td>796</td>
<td>1082</td>
<td>380</td>
<td>0.164</td>
<td>1</td>
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</table>

5. SUMMARIZE
According to large scale SCC experience, the following recommendations are proposed:

- SCC used where it is required such as improper reach to formwork, congested reinforced concrete, thin layer concreting and so on.
- For decreasing loss of workability by time, water to cement ratio must select more than 0.4 but for increasing workability, it doesn't require increasing free water and it can accomplish by use of chemical admixtures.
- The sand used in SCC has lower fineness modulus than conventional concrete.
- Total volume of aggregates, maximum size of aggregates, average diameters of aggregates and sand grading are the significance parameters in SCC mix design.
- SCC requires curing much than conventional concrete to prevent plastic shrinkage, loss of strength and cracking.
- SCC is more sensitive to aggregate grading especially fine aggregate, amount of cement and mineral additive of filler and amount of HRWR.
- Application of SCC in construction requires more strength and no leaking formwork than conventional concrete.

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REFERENCES
فرزین خورشکی آرماتور در عربه تعدادی از بلهای راه‌های واقع در ناحیه کویری ایران

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چکیده
در سال‌های اخیر تعداد زیادی از سازه‌های دریا در مناطق مختلف کشور در اثر عوامل گوناگون دچار آسیب‌پذیری و یا خرابی ورود برده‌ها در بین از جمله رایج ترین این خرابی‌ها در سازه‌های بتن مسلح بوده و یکی از مهم‌ترین مشکلاتی است که امروزه مهم‌دانی عمارت‌های نگهداری سازه‌های بتن مسلح با آن مواجه می‌باشند. در سازه‌های بتنی که چندین سال زمان مصرف می‌گذارد نیاز به نشان و سرعت خورشکی آرماتور یکی از پارامترهای موتور در مقاومت‌سازی سازه می‌باشد.

در این تحقیق به منظور آرزیابی خرمشدی آرماتورها در بلهای راه‌های واقع در ناحیه کویری، آزمایش‌های مقاومت فشاری، تعمیر عمل کرمان‌سون، برویلوی بودن کاتر، پانل‌سازی خورشکی آرماتور و سرعت خورشکی آرماتور برون‌لایه این سازه‌ها در بهینه‌سازی سازه‌های دریایی و جاذبه‌های ناحیه کویری، آزمایش‌های مقاومت فشاری، تعمیر عمل کرمان‌سون، برویلوی بودن کاتر، پانل‌سازی خورشکی آرماتور برون‌لایه این سازه‌ها در بهینه‌سازی سازه‌های دریایی و جاذبه‌های ناحیه کویری صورت می‌گیرد.

کلیدواژه‌ها: خورشکی آرماتور، نیم بیل، سرعت خورشکی آرماتور، کرمان‌سون، بودن کاتر

1. مقدمه
در سال‌های اخیر با مطالعات مختلف دنیا و در دانشگاهیان اردشیر مینودر و در پانل‌سازی کاتر، پانل‌سازی خورشکی آرماتور به منظور محاسبه عملکرد آرماتورها در تعمیر طول عمر مفید سازه‌های بتن مسلح اهمیت دارد.

در این مقاله خرمشدی آرماتور در عربه بلهای راه‌های اردشیر مینودر با عرض تقریبی 30 سال مورد بررسی قرار می‌گرفت. با انجام مشاهده‌های زمینه‌ای مشابه وارده از آزمایش‌های مقاومت فشاری، عمل کرمان‌سون، برویلوی بودن کاتر و پانل‌سازی در نیم بیل و سرعت خورشکی آرماتور، عمل خرابی در هر ده‌هها امکان‌پذیر گردید. بر اساس سوالی‌ها و شواهد و مقالات منتشر شده، آسیب‌پذیری‌های تراشی از نظر بودن کاتر در تراشی چندی و حاصله خلیج فارس می‌باشد، این
در حالی است که در تحقیق حاضر به این نوع آسیبدیدگی در تابعه کوربی ایران پرداخته شده است.

2- خورش‌گذی فولاد در بین

خورش‌گذی فولاد در بین عناصر به‌ویژه نیکل و پتاسیم که در بین‌های این ماده هستند، به طور مداوم در شرایط متابولیک و در دارای آکسیژن محسوب شده باشند. خورش‌گذی خاوین شد. نگاهی به میلکرد در Zn بیشتر نسبت به نیکل می‌باشد. این شیء می‌باشد که در این خورش‌گذی خوانده می‌شود. نیکل از این اکسید در حفاظت از آنزیم‌ها در نزدیک سطح فولاد، خورش‌گذی را توقف می‌کند. این لاثی محافظ را کاهش می‌دهد (به طور عمده به دلیل کربانتاسیون) و یا حضور میوشای کردن سطح pH می‌رسد.

1- خورش‌گذی ناشی از کربانتاسیون

زمانی که میلکرد اطراف میلکرد از 1/511 به‌ویژه باشند، این محافظ را روی روتی میلکرد سالم پایی میاند و شیمی‌های pH از خورش‌گذی خوانده می‌شود. زمانی که pH از این مقادیر وسیب به میلکرد نشان دهنده‌ها در بین‌های این ماده می‌باشد. این شیء می‌باشد که در این خورش‌گذی خوانده می‌شود. pH ناشی از این طریق مبتنی بر سطحای این ماده و به شکل جهتی این کربانتاسیون شده باشد.

2- خورش‌گذی ناشی از غرفه‌بودن کر

خورش‌گذی ناشی از غرفه‌بودن کر در ایست اصلی خروشگری در نقاط مختلف دنیا شناخته می‌شود. حضور بیان در سازه بینی، زمانی که اکسیژن و کربانتاسیون به دسترس باشد می‌توان باعث خروش‌گذی میلکرهای فولادی‌گردد.

روش اول کر در دانش بنیان (میلکرد‌های افزودنی) می‌باشد. روش دوم، نفود بیان از محیط پیرو亚洲 به داخل بیان (آب، دریا، منجمد، بیجا...) می‌باشد. بیان کر در دانش بنیان توانایی روی صورت وجود داشته باشد، بیان کربانتاسیون باعث شیب‌های اضافی می‌شود که در نتیجه هیچ نتایجی بر فرآینده خروش‌گذی فولاد در بین ندارد و یا به صورت آزاد باشد.

روش ارزیابی سازه

بررسی وضعیت یک سازه بین مسلح ولیون گام برای پایداری و ترمیم آن می‌باشد. برای تعیین علل خرابی و گستره‌ای آن، این بررسی مقدار خروش‌گذی و ارزیابی وضعیت کنونی سازه ضروری می‌باشد. یک بررسی دقیق شامل دو مرحله انجام می‌شود. مرحله اول ارزیابی و مطالعه اولیه سازه بر اساس مشاهدات ظاهری است که باید ماهیت خرابی‌ها
را مشخص نمایید و پایه ای برای مطالعات دقیق تر پاشد. در حالی که استفاده از آزمایش‌های الکتروشیمیایی در مرحله بعد برای مطالعه وضعیت دقیق میکروژده و پویا سرعت کوردنگی ضروری است.

الف- مشاهدات ظاهری

خراش‌های ناشی از خوردنگی با استفاده از مشاهده ظاهری برنامه مناسبی به‌عنوان قابل تشخیص می‌باشد. طبقه بندی خراش‌ها با توجه به گسترش ترک ناشی از خوردنگی تعیین شده است. در پروسه حاضر که شامل بررسی حدود 30 بیل رادیولوژی در ناحیه کورنی ایران بود مساحت در حدود 800 متر مربع از ارزیابی ظاهری شد. در این محل‌ها ترک‌های موی سطح در نواحی و روزه‌های آرام‌تر از بین اصلی جدا شده است. قابل شناسایی می‌باشد. این خراش‌ها بیشتر در نواحی که بین در معرض رطوبت قرار دارند، رویت می‌شوند. متوسط ضخامت پوشش بینی عرضه بل‌ها در حدود 0.3-0.5 میلی‌متر می‌باشد.

ب) آزمایش‌های انجام شده

بر پایه مشاهدات ظاهری، عرضه بل‌ها به دو دسته بدون خوردنگی (a) و خوردنگی زیاد (b) تقسیم شدند. در ادامه نتایج آزمایش‌های انجام شده ارائه می‌شود.

ی- 3- مکانیق فشاری

برای ارزیابی مشخصات سازه ای مصالح موجود، منفه‌های با قطر 10 سانتی‌متر از سازه تهیه شده و مورد آزمایش قرار گرفتند. متوسط مقاومت فشاری منفه‌ها در دو سطح (a) و (b) در جدول (1) آرایه شده است. این مقادیر متوسط 3 نمونه آزمایش شده می‌باشند. با توجه به نتایج مقاومت فشاری می‌توان به طور تقریبی نسبت اب‌ به مواد سیمانی در بین را در بین 4/3 تا 6/7 ارزیابی نمود. این میزان نسبت آب به مواد سیمانی برای اجرای این‌های فنی زیاد به نظر می‌رسد و قطعاً ملاحظات دوم و پایایی بین موردنعایت قرار نگرفته است.

جدول 1: نتایج مقاومت فشاری منفه‌ها

<table>
<thead>
<tr>
<th>سطح خوردنگی</th>
<th>متوسط مقاومت فشاری منفه‌ها (kg/cm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>210</td>
</tr>
<tr>
<td>b</td>
<td>194</td>
</tr>
</tbody>
</table>

2- 3) عمق کربناتسیون

کربناتسیون باعث کاهش pH محیط، احیال ظاهری و یا خواص کم‌مدل کلر مقیم و ایجاد کلر آزاد می‌شود. کلر آزاد شده می‌تواند وارد قسمت‌های کربنات سده بین شود اما پدیده موجب افزایش غلظت کلر آزاد در ضخامت بین یو آرام‌تر شده و ممکن است کلر آ در در میکروژده و به‌سیار بحرانی‌پسند بدن‌های سازده‌ها ناخواسته کلر که کربناتسیون در آنها رخ داده است نسبت به سازده‌ها که فقط از یکی معضل رنج می‌برند، بیشتر در معرض خوردنگی قرار دارد [4, 5].
عمق کرنیاتاسیون را می‌توان با پاشیدن محلول فلقتالین بر روی مقطعی بن تنشخیص داد. در مناطقی که تحت تاثیر کرنیاتاسیون نیوزه، مقدار pH بین تقسیمی 8/5 و جفتالین در این منطقه به رنگ ارغوانی در می‌آید.

جدول 2: نتایج آزمایشهای عمق کرنیاتاسیون

<table>
<thead>
<tr>
<th>حالت خوردار (mm)</th>
<th>متوسط عمق کرنیاتاسیون (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a)</td>
<td>20</td>
</tr>
<tr>
<td>(b)</td>
<td>50</td>
</tr>
</tbody>
</table>

متوسط عمق کرنیاتاسیون سطوح بین مسلم در جدول شماره (2) نشان داده شده است. مطابق جدول (2) در سطوحی از ناحیه خورداری آرام‌تر مشاهده شده است. (ب). متوسط عمق کرنیاتاسیون 50 میلیمتر بوده که با توجه به اینکه متوسط ضخامت یوشش بینی روی آرام‌تر در بالا 30 میلیمتر می‌باشد سی توان تیز نتایج گرفته که آرام‌ترها در این ناحیه تحت تاثیر بیشتر کرنیاتاسیون قرار گرفته است. در ناحیه (a) متوسط عمق کرنیاتاسیون 2 میلیمتر و مساوی مقدار یوشش بینی روی آرام‌تر می‌باشد. در این ناحیه آرام‌ترها در آستانه قرارگیری در معرض پدیده کرنیاتاسیون هستند.

3-3- پروفیل یون کلر

برای تعیین مقدار کلر در بتن، از یک عدد مختلف عضو پیل‌ها نمونه برداری انجام شد. میزان کلر کل (Total) موجود، طبق استاندارد ASTM C114 پروفیل‌های یون کلر سطوح (a) و (b) در شکل (1) به طور نمونه نشان داده شده است. همانگونه که دیده می‌شود مقدار یون کلر در نزدیک آرام‌تر (عمق 20 میلیمتر) در سطح (a) گزارشی که مقدار کلر در نزدیک آرام‌تر به مقدار کلر خودکی تبلور اثر یون کلر سیار کم است. در حالتیکه در سطح (b) مقدار کلر در نزدیک آرام‌تر به مقدار کلر خودکی تبلور اثر یون کلر سیار کم است و با توجه به میزان عمق کرنیاتاسیون و مقدار یون کلر در نزدیک آرام‌تر می‌توان نتیجه گیری نمود که کرنیاتاسیون در ترکیب با یون کلر منجر به کاهش مقدار کلر آستانه خودکی و توسعه وکستر خراش‌ها شده است.

شکل 1- پروفیل یون کلر تهیه شده در دو سطح (a) و (b)
بررسی خوردگی ارومیان در عرضه تعداید از پله‌های راه‌اهن... / ۶۵

در اساس پروپیل‌های ارائه شده به نظر می‌رسد مقادیر کلی اولیه در دست حدود ۲/۰۰۲ درصد وزنی قابل شده و در محدوده مجاز می‌باشند. این در حالت است که کل‌های موجود در تیم اعیانی از مخرب پیرامون وارد باشد است و به نظر می‌رسد مشا آن کل‌های موجود در شن‌های روان و انصرف در منطقه کوبوری و خصوصاً در قسمت و موافقه که با شریف و خلوتی همراه است می‌باشد. علاوه بر آن افت‌مقدار کل در ترکیبی سطح خارجی که موجود در شکل (۱–۰) ملاحظه می‌شود به دلیل پدیده کنترالاتوریون می‌باشد که موجب تجزیه نمک فریب و کم شدن کلی مقد و آزاد شدن آن و حرکت نمک‌ها به سمت داخل بین می‌شود.

۳- آزمایش پتانسیل نیم بیل

آزمایش تیبین پتانسیل نیم بیل، آزمایشی غیر مخرب و استاندارد می‌باشد که به طور گسترده در ارزیابی وضعیت خوردگی مواد استفاده قرار می‌گیرد. این آزمایش مطابق با استاندارد ASTM C876 یا آزمایش نیم بیل (انجام شده است.

پایه توجه داشته که حضور اکسیژن، غلظت کل و مقاومت الکتریکی بین تأثیر زیادی بر روی تغییرات های ناشی از پتانسیل نیم بیل دارد. در این آزمایش میزان بررسی خوردگی طبق الکترود Ag/AgCl ایمن می‌باشد، این آزمایش می‌باشد: ۱- پیانسیل: با احتمال بیش از ۹۰ درصد هیچ خوردگی آزمایش در زمان آزمایش وجود ندارد. ۲- pNشیل: >۲۳۳ mV. ۳- pNشیل: <۲۳۳ mV. فعالیت خوردگی ناشاکس است.

در شکل (۳) به عنوان نمونه، نتایج آزمایش پتانسیل خوردگی ارومیان برای دو سطح مذکور از شکل داده شده است. در شکل (۲) منفیترین پتانسیل نیم بیل در ناحیه با خوردگی زیاد (شکل (۳)) -۳۰۰ میلی ولت است. در صورتی که در ناحیه در ناحیه بدون خوردگی (a) این مقادیر از ۴۰۰ میلی ولت می‌باشد، با توجه به توصیه‌های ارائه شده در قبیل، در سطح (b) آن احتمال بیش از ۹۰ درصد، خوردگی رخ داده است. در حالتی که در شکل (a) فعالیت خوردگی نامشخص است.

شکل ۲ نتایج آزمایش پتانسیل خوردگی ارومیان

سطح با خوردگی زیاد

سطح بدون خوردگی
نتایج آزمایش تعیین سرعت خوردگی آرام‌تر

اندازه‌گیری با دستگاه کلاپلیاس یک روش پلاژیاسیون سریع برای تعیین میزان خوردگی در سازه‌های بین مصالح به‌حساب می‌آید. این روش با سه شکل اندام‌گیری، نیمی، نیمی در محیط‌های مرطوب و نیمی در محیط بی‌مرطوب بعلت تمیز، کم‌آسمان و هم‌نیم‌آسمان سرعت زیادی در بستر اورود میزان خوردگی آرام‌تر در بین تسویه پایه است. با استفاده از دستگاه کلاپلیاس، پتانسیل نیمی، نیمی و سرعت خوردگی آرام‌تر قابل تعیین می‌باشد.

جدول 3: تخمین مقدار خوردگی قابل مشاهده از روی قرانته‌های انجام شده با دستگاه کلاپلیاس

<table>
<thead>
<tr>
<th>مقدار خوردگی (µA/cm²)</th>
<th>مقدار اندازه‌گیری شده (µA/cm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.15</td>
<td>0.15</td>
</tr>
<tr>
<td>0.15 - 0.25</td>
<td>0.15</td>
</tr>
<tr>
<td>زیاد</td>
<td>زیاد</td>
</tr>
</tbody>
</table>

نتایج آزمایش تعیین سرعت خوردگی آرام‌تر به طور نمونه در شکل (۱) نشان داده شده است. همانطور که در این شکل دیده می‌شود، نقشه خطوط هم‌مرز سرعت خوردگی در سطح مورد نظر (۳ و ۱) قابل مقایسه با نتایج به دست آمده از پتانسیل نیمی بیل راهشه در شکل (۲) است. نکته‌ای در سطح (۱) بین از ۶ میکرو آمپر به سانتی‌متر مرغوب می‌باشد که طبق جدول (۲) مقدار خوردگی در این سطح زیاد است و آزمایش نیمی پیل نیز احتمال خوردگی را پیش از ۰.۱۵ درصد نشان می‌دهد. این در حالیست که نتایج سرعت خوردگی در سطح (۳) نمونه‌ای از آماری‌پذیر نیمی بیل در محدوده عدم قطعیت قرار دارد.

پتانسیل نیمی بیل در محدوده عدم قطعیت قرار دارد.

شکل ۲: نتایج آزمایش سرعت خوردگی آرام‌تر

شکل ۱: مسیر خوردگی

شکل ۳: تابع بین نیمی بیل
نتایج گیری

1- ارزیابی خرابی‌سازی‌های بتن مسلح به منظور ترمیم آن باز در یک نظام منطقی اجرای شود. یک ترمیم با کیفیت نیازمند شناخت علت وقوع خرابی، مشاهدات کامل آسیب‌ها، مشخصات دقیق محل‌های آسیب دیده است.

2- با توجه به دست آمده عامل اصلی وقوع خوردگی در بتن مسلح عرشه بل‌ها وقوع پدیده کریستالاسیون می‌باشد.

3- مقدار پتانسیل نیم بیل تنا احتمال خوردگی را نشان می‌دهد و حتی در برخی از حالت‌ها اضافه از آن روش یا محدودیت‌های مهم‌است. در حالی‌که با استفاده از روش سرعت خوردگی می‌توان مقدار خوردگی را در نواحی خوردگی مشخص کرد و برخورد شده و یا حتی نواحی مشکوک به خوردگی ثبت نمود.

4- پدیده کریستالاسیون در ترکیب با پون‌کل بلند به توسه و کنترل و سعی خرابی‌ها و خوردگی‌های مهم‌تر در سازه‌های بتن مسلح اهمیت دارد. همان‌گونه که پدیده شده در سطوح دارای شدت خرابی زیاد خوردگی تحت اثر تواوی کریستالاسیون و پون کل رخ داده است. در حالی که در سطوح دارای شدت خوردگی کم خوردگی نتیجه ناگهانی از پدیده کریستالاسیون محتمل می‌باشد.

5- بر اساس مطالعات انجام شده در قالب این تحقیق مشخص می‌شود که خوردگی ناشی از نفوذ پون کل در کشور محدود سواحل و جزایر خلیج فارس و نم‌می‌شود و در نواحی مرکزی کشور خصوصاً در نواحی کویری این نوع خرابی وجود دارد. بدیهی است اجرای سازه‌ها و ابندی نتیجه کویری و نسبت آب به مواد سیمانی زیاد موجب پدیده کریستالاسیون و تشدید فاصله‌کردن خوردگی و آسیب‌دیدگی پทน خواهد شد.

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چکیده
از انتخاب و راه‌گیری سیستم‌های تولید انرژی توسط یک کشور به سه منظور عمده می‌باشد: اقرار با نیازهای روندی و سیره‌ای رشد بهزیستی، ضد اقتصاد و کاهش مصرف انرژی رو به حال واقعیت مواجه می‌شود. در این پژوهش، تکنیک‌های مختلف از نظر استانداردهای جهانی بررسی شده و در نهایت انتخاب سیستم‌های تولید انرژی به‌منظور کاهش مصرف انرژی و کاهش تولید کهکشانی آلودگی محیط زیست ارائه شده است.

۱- تامین ضوابط آئین نامه‌ای
مطابق مبحث ۹ مقررات ملی ساختمان هوای سرد به وضعیت اطلاع می‌شود که برای ۳ روز متوالی شرایط زیر برقرار باشد.
الف - دمای هوا متوسط هوا در شیب زمین روز کمتر از ۵ درجه سلسوس باشد.
ب - دمای هوا برای بیشتر از نصف روز از ۱۰ درجه سلسوس زیادتر نباشد.
پ - به منظور اطمینان کافی از وضعیت کهگیلویه بین دئر اثران دمای بین در طول شیب زمین روز ۱۰ در نقطه متفاوتی مختلف سازه تبیه می‌شود.
برای رسیدن به جد مطلوب هنگام اختلاف بین از آب گرم استفاده شده و مطابق آئین نامه، از تمسی
مستقيم آب گرم با سیمان در صورتی که دمای آن بیش از ۷۰ درجه سلسیوس باشد در حین ریختن مصالح در
مخلوط کن، کنترل لازم به عمل می‌آید.

انتقال نسبت آب به سیمان با توجه به روند کسب مقاومت در دمای محيط بوده و کمتر از ۰/۴ می‌باشد.
عمل آوردن بین تا حداقل به مدت ۴۴ ساعت و تا رسیدن به مقاومت ۵ مگاپاسکال ادامه می‌یابد.
عمل آوردن بین با استفاده از پوشش (رول‌های یشم شبیه که در داخل نایلون قرار داده می‌شود) و گرم کردن داخل
تنولها با هیتروهای برقی و گازی تا ۱۵ درجه سانتی‌گراد انجام می‌گردد.

مزیت استفاده از هیتروهای برقی و گازی نسبت به گرم کننده‌ها با مواد سوختی نظر نفت و گازوئیل به این علت
است که از احتمال تبخير آب و کریستال شدن سطح بینی در اثر احتراق، جلوگیری به عمل می‌آید.

با توجه به اینکه در سیستم قالب تولید به جهت افزایش سرعت عملیات اجرایی، باز کردن قالب‌ها در مدت زمان
کمتری از زمان تعريف شده در مبحث ۹ مقررات ملی انجام می‌شود، لذا از مواد روان کننده که خاصیت
تبدرک کننده دارد استفاده و برا برداشت بنی بعد از قالب برداری از پایه‌های اطمینان در وسط دهانه‌ها استفاده
شده و دقت لازم به عمل می‌آید تا همیشه پایه‌های اطمینان در دو طبقه متوالی وجود داشته باشد و تا حد امکان
هر دو پایه اطمینان نظیر در دو طبقه، روی هم و در امتدادی واحده قرار گیرند.

بلوک ۹۱ - زمستان ۸۶
۲- تهیه ابگرم پچینگ
برای تهیه ابگرم لازم در پچینگ از تناکر آب و استفاده کردن آن و مجهز نمودن به سیستم گر ماشی بوسیله مشعل‌های گازی، همراه با نصب ترموستات جهت رسیدن دماهی آب بین به ۸۰۰۰ استفاده می‌شود.

چلوگیری از بخ‌زن مصالح پچینگ
با اجرای لوله‌کشی در محل دیوار مصالح پچینگ و نصب گرم کن با مشعل گازو تهی و عبور دادن هوای گرم از
داخل لوله‌ها، از یخ زدگی مصالح پچبینی می‌شود.

جهت حفظ حرارت در محل دیگری مصالح پچبینی، سطح مصالح را با روی‌های یشم شیشه (با کاور نابلست) پوشانیده و بر روی آن جادو برزنتی کشیده می‌شود. در قسمت یکنواحی با گذشتن یک خرک و ایجاد اختلاف ارتفاع، با قرار دادن هیترهای بلندی فن دار، سطح فوقانی مصالح گرم نگه داشته می‌شود.
جهت جلوگیری از افت دما در بتن ریزی جهت حدود ۱۲ ساعت قبل از بتن ریزی با قرار دادن هیترهای برقی فن دار در داخل قالب‌های تونلی و استقرار آنها در قسمت خروجی یا گارد، گرمایش داخل قالب تونلی نامناسب شده و دمای قالب‌ها به حدود ۱۰۰° C رسید.

![عکس](https://example.com/image1)

**خط جهت برقی فن دار**

**ساخت قسمت خروجی قالب تونلی**

### ۵- عمل آوری و گنتانل دما بتن

به منظور جلوگیری از افت دما بتن در محل بتن ریزی، مقدار بهره بتن در پنج‌گانه در هر مرحله، به مترمکعب تقلیل و بدين ترتيب مدت زمان حمل مصرف بتن كاهش مي‌يابد.

![عکس](https://example.com/image2)

**عکس از نتايج قبلی و پس از گنتانل دما**

پس از اتمام بتن ریزی، گرمایش داخل قالب‌های تونلی تا مدت ۴۸ ساعت بعد از بتن ریزی ادامه یافته و سطح بتن با پشم شیشه پوشانده و بر روی پشم شیشه نیز همانند پوشش روی مصالح پچینگ، جادر کشیده می‌شود و...
آرمان‌های اجتماعی رونده‌ها یا نام‌های پیش‌نوازه مشهد است. با رعایت موارد فوق ذکر تا دمای 18-15 رطوبت بتن روزه به مدت 8 روز آن هم بدلیل کولاک شدید بود.

1- اقدامات انجام گرفته چهت کاهش مدت زمان اجرای اسکلت بتنی

در راستای تحقق و نیل به اهداف کیفی تعیین شده از طرف شرکت مادر در سال 78 که کاهش مدت زمان اجرای اسکلت بتنی پروسه آذران به میزان 60 درصد نسبت به سال 74 مدت قرار گرفته، از همان ابتدای سال برای هر یک از بلک‌ها، متوقفه نمودارهای پیوستی (صفحات 54 تا 56) جهت مقایسه مدت زمان اجرای شناسنامه مدت زمان اجرای اسکلت تهیه گردید به طوریکه پیش‌ترین مدت مورد رفع به بلک A1 در مدت 18 روز و 3 ماه‌نامه مدت اجرای اسکلت به بلک A3 در مدت 33 روز انجام گرفته است. پیش‌ترین مدت اجرای یک نیم سرم در بلک A1 به مدت 8 روز و با مطمئنی رفع موانع، در بلک B3 به مدت 2 روز رسید.

بطور خلاصه مواردی که در سرعت دادن کار اجرایی اسکلت نقش اساسی داشته‌اند به شرح زیر می‌باشد:
1- افزایش سرعت بتن ریزی با اضافه کردن حجم پاکت به میزان 1/8 مترمکعب

2- کوتاه نمودن مسیر حمل بتن و اضافه نمودن تراک میکسر جهت حمل بتن.

3- افزودن تعداد نفرات به پیمانکار

4- به منظور جابجایی قالب‌های پلاستیکی، میانی و

بکسی و جداسازی مخصوص به همان قالب استفاده می‌شود که باید و بسته کردن مجدد سیم بکسی‌ها باعث کندی عملیات می‌گردد. با تجميع کل سیم بکسی‌ها در قالب تاور و تعمیر از همان قسمت سرعت جابجایی قالب‌ها افزایش یافت.

5- انجام عملیات تمیز کاری و روغن کاری قالب‌ها به هنگام خروج بست به

(PLAT FORM) و

انتقال به نیم ست دیگر (توضیح اینکه قبل از تور کاری، روغن کاری قالب‌ها بیش از اندازه آنها به سطح زمین صورت می‌گیرند و سپس به نیم ست بعدی انتقال داده می‌شود)
1- لوله‌های تولیدی کارخانه زد پلی‌کاری از نوع PVC طولی نسوز که قبل استفاده می‌شود و مقاومت فشاری عمودی کافی داشته با نو کاری از نوع PVC سخت نسوز سفید تولیدی کارخانه ثابت پی ای اولین توضیح شد که این امر باعث کاهش تعداد لوله‌های شکسته شده در مرحله اجرای مش دوم سقف و خنادی گردید.

SHOP DRAWING

2- تهیه نقشه ثابت

3- تهیه شابلون و فنر خم مناسب با سایز لوله جهت ایجاد خم لوله‌های برق دیوارها

خم‌کاری لوله‌ها با استفاده از فنر و شابلون خم
بهترین دیوایر در هوای سرد و اقدامات اصلاحی در جهت کاهش ...

۴- تهیه شابلون جهت ساخت بخش‌های سر لوله‌های برق دیوارها بصورت سری
۵- سوراخ گذری محل ورود لوله‌ها به قطعه‌های گلد و برای به صورت سری
۴- تهیه پلاستیک جهت محل قرارگیری لوله‌های انظار سقف و دیوار و محل باز شو را با توجه به ابعاد آنها و
ایجاد سوراخ‌های لازم در آنها به صورت سری.

استفاده از پلاستیک جهت محل قرارگیری لوله‌های انظار سقف

۷- علامت‌گذاری لوله‌ها با رنگ‌های متنوع برای شناسایی محل مبهم‌سازی مختلفی از جمله روشنایی برای و
تلفن و آنل و درب بار کن برای تسرب در لوله‌های سقف درکارگاه موقت.
۸- کد گذری و دسته بندی لوله‌های خم و غلاف‌های جفت و نگه و قطعه‌های سوراخ‌شده با توجه به تعداد
سایز آنها در نقشه SHOP DRAWING
۹- کد گذری و دسته بندی با توجه به تعداد آنها در نقشه SHOP DRAWING

۱۰- ساخت غلاف فلزی قطعه‌های گلد و برای از لوله سیک نرده ای به قطر 4/16 یا یک جوش 2 عدد میلکرده
نمره 5 بطول 40 سانتی‌متر و بستن آن با سیم ارمنی‌بندی به مش دیوارها، جهت قرارگیری محل دقیق قطعی
کلد و پریز و فاصله از دیواره قالب تولید
11- حضور مستمر ناظم تاسیسات برای در زمان بین ریزی و انورش یک نفر به عنوان کارگر فنی جهت کنترل و ترمیم لوله‌ها در صورت شکستگی لوله‌ها در حین بین ریزی

3- مطالعه و بررسی در سرعت دادن به عملیات مکانیکی
1- لوله‌کشی سیستم فاضلاب و ونت (لوله‌های به قطر 50 تا 63 میلی‌متری) معمولاً با انجام شیارکنی و برخ لواردهای داخلی و خارجی صورت می‌گیرد که موجب تخریب دیوارها و افزایش تخاله می‌شود. در پروژه آذرن که غلاف از لوله PVC از لوله مورد نظر هم‌زمان با انرژی دیوارها در داخل دیوار قرار داده و بعد از اتمام دیوارچینی غلاف را برداشته و برای استفاده مجدد به طبقه دیگر انتقال می‌یابد که بدن تربیت از تخریب دیوار جلوگیری می‌شود.
۲- محل عبور لوله‌های آب شهری به داخل و خارج، مطلق نقشه‌های اجرایی در مشاهده و داخل و خارج به صورت زیرشکل مشخص و از نظر معماری نیاز به اجرای سقف کاذب دارد. در بلوک‌های نیاز A لوله‌های نمابان به مقدار ۲۴۰ متر طول و در بلوک B به مقدار ۱۶۲۰ متر طول می‌باشد. با تغییر نقشه‌های اجرایی و اجرای لوله‌ها بصورت تکرار از هزینه مربوط به خرید مصالح و دستمزد اجرای سقف کاذب شامل (خرید مصالح از قرار ۵۰۰۰ ریال به مبلغ ۵۰۰۰۰ ریال – رایت‌س بنی‌ ۵۰۰۰ ریال – گچ و خاک ۵۰۰۰ ریال – دستمزد آهن کشب به مبلغ ۵۰۰۰۰ ریال) جمعاً به ازای هر متر طول به مبلغ ۱۰۰۰۰۰ ریال، به مبلغ کل ۵۰۰۰۰۰۰ ریال عمده صرفه جویی گردید.

۳- اجرای سایپورت لوله‌ها در دکت‌های ناسانه معمولاً پس از انجام اسکلت انجام می‌شود. در پروژه آذرنعم، عملیات آهن کشب داکتر اسکلت با استفاده از مصالح پرت حاضر نهایت از بشق کشب دیواره‌های خارجی، هم‌زمان با اجرای اسکلت با قرار گرفتن شاک‌که‌های فلزی در داخل بین اجرا می‌شود.

۴- جهت کاهش عملیات آهن کشب در چاهک‌های آسانسور بلوک‌های نیاز A و B، دیواره‌های پوششی در...
طبقات (بک ضلع در بلک A و سه ضلع در بلک B) پلیت گذرانی شده و از حجم عملیات اهمیت کشی به مقدار ۶۰ درصد در بلکهای تیپ B و ۲۰ درصد در بلکهای تیپ A کاسته شده است.

پلیت گذرانی دیوار بینی چاهکهای اسانتور

5- نقشه‌های ناسیسات مکانیکی در رابطه با هماهنگی محل این‌پینگ‌ها (داکتی‌های ناسیسات) با نقشه‌های معمول و سازه و با در نظر گرفتن طول اتصالات مصالح مصرفی در لوله کشی سیستم فاضلاب سروپس‌ها و راپرها در این اصلاح شده و در این رابطه نقشه‌های ازبین SHOP-DROWING تهیه گردید.

6- جهت اجرای پازشهای مربوط به ناسیسات مکانیکی در سقف‌ها در زمان بین رزی سقف، نسبت به ساخت قالب‌های فلزی مربوط به محل عبور لوله با دو سایز بیشتر از قطر لوله‌ها به شکل کوینی اقدام گردید.

برای ایجاد محل پازشهای طبقات در امتداد هم قرار گیرند، مختصات مرکز پازش در سقف قالب تولید سوارخ کاری شده و با قرار گیری بین قالب پازش (مربع شهره ضروری) در سوارخ مذكور و بعد از کبارک اولیه بینی، قالب مذكور برای استفاده در سقف طبقه بعدی برداشته می‌شود.

قالب فلزی محل عبور لوله‌ها از سقف بینی
پژوهش خودگردی آرمان‌های در عرضه تعدادی از یل‌های راه‌آهن واقع در ناحیه کویری ایران

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چکیده
در سال‌های اخیر تعداد زیادی از سازدهای بتنی در مناطق مختلف کشور در آن‌ها عامل کوتاهگی و یا خرابی زودرس شده‌اند. خودگردی میان‌گره‌ها در بتن از جمله رایج ترین این خرابی‌ها در سازدهای بتن مسلح بوده و یکی از مهارت‌های مشکلاتی است که امروزه مهندسان عمران در تغییرات سازدهای بتن مسلح با آن مواجه می‌باشند. در سازدهای بتنی که چندین سال از ساخت آنها می‌گذارد تعیین میزان و سرعت خودگردی آرمان‌ها یکی از پارامترهای مؤثر در مقاومت سازه می‌باشد.

در این تحقیق بر مبنای منظور آرمان‌های خودگردی آرمان‌ها در یل‌های راه‌آهن واقع در ناحیه کویری، آزمایش‌های مقاومت فشار آبی انجام شد. برای این تحقیق، پروپیل یون کلاس، پنتانیل خودگردی آرمان‌ها و سرعت خودگردی آرمان‌ها بر روی عرضه یک انجام شد. در نهایت با مقایسه نتایج و آزمایشات در دو دلفی اصلی خودگردی آرمان‌ها تعیین گردید. بر اساس نتایج بدست آمده نظر می‌رسد در ناحیه کویری ایران مشاهده سلول خاص خرابی‌های مشابه توانایی تولید سیستم‌های خودگردی های کلیدوازه‌ها: خودگردی آرمان‌ها، نیم پیل، سرعت خودگردی آرمان‌ها، کرتوناتاسیون، بون کر

1- مقدمه
در سال‌های اخیر تعداد زیادی از نقاط مختلف دینا خودگردی میان‌گره‌های فولادی در بتن به عنوان دلیل اصلی خرابی‌های زودرس و در برخی موارد نخوردن کلی سازه بتن مسلح شناخته می‌شود. [1] همچنین بررسی سرعت خودگردی آرمان‌های منظور مطالعه علم‌گرای‌اند یا آرمان‌ها در تعبیر طول عمق می‌弗اید سازدهای بتن مسلح اهمیت دارد.

در این مقاله خودگردی آرمان‌ها در عرضه یل‌های راه‌آهن با عمر تقریبی 30 سال مورد بررسی قرار گرفت. با انجام مشاهدات ظاهراً و آزمایش‌های مقاومت فشاری، عمق کرتوناتاسیون، پروپیل یون کلاس، پنتانیل نیم پیل و سرعت خودگردی آرمان‌ها، علت خرابی در هر سه از یل‌ها تعیین گردید. بر اساس سواپس و شواهد و مقالات منتشر شده، آسبی بیدگی‌های ناشی از نفوذ بیون کر در تواجه خشن و حاشیه خالق فشار گزارش شده است. این
در حالی که خورشید گولاندا در بین عناصر به دلیل فوتو یون کلر پدیده کردنی و تولیده کردنی معنی‌دار مصرف شده بودن، خورشید خاوهندش. هنگامی که میلگردار
پنثر می‌گردد که آن را از خورشید حفظ می‌کند. لایه اکسید احتمالاً پوشش مترکم و غیر قابل فتوحی را ایجاد می‌کند
که با معدل ترک حرکت کاتیون‌ها و آنیون‌ها در ترکیب سطح خورشید، خورشید را متوقف می‌کند. لایه
محافظات با کاهش pH محیطی(به طور عمده به دلیل کردنی و اکسید) و یا حضور یون‌های کلر و سواده از بین
می‌رود.[2]

2- خورشید‌کن‌های از کردنی و اکسید
پنثری که pH محیط اطراف می‌گردد از 1/15 تا 1/19 پشت باشد که می‌توانند روی میلگرد سالم باقی مانند و میلگرد را
از خورشید حفظ کند. پنثری که pH از این مقدار کمتر یک می‌شود و حفاظت به عمل آمده از میلگردها از بین می‌رود. کردنی و اکسید در بین دلیل اصلی برای کاهش pH شناخته می‌شود.
پنثری که pH ناشی از آن به طور عمومی از سطح بین آغاز می‌شود و به شکل جهیزه کردنی شده به
سمت مرکز بین پیش می‌رود. سرعت کردنی和 اکسید به عوامل محیطی (رطوبت، دما و غلظت دی اکسید کربن هوا)
و مشخصه‌های (فیتو پدری، قلبیت) بستگی دارد. در یک یا یکی نسبت، سرعت کردنی و اکسید در حدود
1 میلی متر بر سال می‌باشد.[3]

2- خورشید‌کن‌های از نفوذ یون کلر
پنثری که ناشی از یون یکی از دلایل اصلی خورشید در نقاط مختلف دنیا شناخته می‌شود، حضور یون کلر در
سازه بینی، زمانی که اکسیدز و رطوبت لازم در دسترس باشد می‌توان باعث خورشید میلگرد‌های فولادی گردد.
یون کلر به دو روش اصلی وارد می‌شود:
روش اول، یون کلر موجود در مصالح بین سیستم‌های افروندی (الکتریک‌ها، افروندی‌ها نیز).
روش دوم، یون کلر از محیط پیامون به داخل بین (آب دریا، منکسه‌های بزرگ...).
یون کلر در داخل بین می‌تواند به دو صورت وجود داشته باشد، یکی به اندازه و با کاهش شیمیایی مقید شود
که در نتیجه هیچ تاثیری بر خورشید فولاد در بین ندارد و یا به صورت آزاد باشد.
روش ارزيابی سازه
بررسی وضعیت یک سازه بین مساحول لوله‌های جام برازی و مربیم آن می‌باشد. برای تعیین انرژی خرابی‌ها و
گسترش آن، بررسی مقدار خورشید و ارزيابی وضعیت کننده سازه ضروری می‌باشد. یک بررسی دقیق شامل
دو مرحله است. مرحله اول ارزيابی و مطالعه اولیه سازه بر اساس مشاهدات نظری است که با پایه‌های خرابی‌ها

را مشخص نماید و پایه ای برای مطالعات دقیق تر پایش در حالی که استفاده از آزمایش‌های الکتروشیمیایی در مرحله بعد برای مطالعه وضعیت دقیق می‌گردد و بر اثر سرعت خوردنگی ضروری است.

الف- مشاهدات ظاهری

خرابی‌های ناشی از خوردنگی با استفاده از مشاهده ظاهری برنامه‌ریزی شده قابل تشخیص می‌باشد. طبقه بندی خرابی‌ها و تعیین وضعیت ظاهری، محل و علت آن تعیین می‌شود. خرابی‌ها با توجه به گسترش ترکب ناشی از خوردنگی تعیین شده‌اند. در پروژه‌هایی که شامل افراد حساس به نوری‌های ایران‌بود مساحتی در حدود 300 متر مربع از چهار طرفی طراحی می‌شود و محل جدایی‌ها مشخص گردید. در این محل‌ها ترکب‌های موی میزANCHER طبقه در نواحی که پوشش روی آماده از بین اصلی جانشین‌ها است قابل شناسایی می‌باشد. این خرابی‌ها پیشتر در نواحی که بین در معرض رطوبت قرار دارند، روتی می‌شوند. متوسط ضخامت پوشش بینی عرضه بل‌ها در حدود 150-200 میلی‌متر می‌باشد.

(ب) آزمایش‌های انجام شده

بر پایه مشاهدات ظاهری، عرضه بل‌ها به دو دسته بدون خوردنگی (a) و خوردنگی زیاد (b) تقسیم شدند. در ادامه نتایج آزمایش‌های انجام شده ارائه می‌شود.

۱-۳ ماقولات فشاری

برای ارزیابی مشخصات سازه ای مصالح موجود، مفهومی با قطر 10 سانتی‌متر از سازه بهره‌برداری و مورد آزمایش قرار گرفتند. متوسط ماقولات فشاری مفهومی در دو سطح (a) و (b) در جدول (1) ارائه شده است. این مقادیر متوسط 3 تومان آزمایش شده می‌باشد. با توجه به نتایج مقاومت فشاری می‌توان به طور تقریبی نسبت آب به مواد ساین در بین (3-1) تا (3-2) ارزیابی نمود. این انتخاب نسبت آب به مواد ساین برای اجرای اینه فنین که می‌تواند و نهایاً ملاحظات و تابعین تبدیل به آن‌ها قرار گرفته است.

جدول ۱: نتایج ماقولات فشاری مفهومی

<table>
<thead>
<tr>
<th>فشار خوردنگی (kg/cm²)</th>
<th>متوسط ماقولات فشاری مفهومی (kg/cm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>۲۱۰</td>
<td>(a)</td>
</tr>
<tr>
<td>۱۵۰</td>
<td>(b)</td>
</tr>
</tbody>
</table>

۳-۲ عمق کرنتاسیون

کرنتاسیون باعث کاهش pH محیط، احاله‌سازی جامه‌ای، کاهش قدر مقدی در محل جهیزه ای، آزاد شدن کلر مقید و ایجاد کر آزاد می‌شود. کلر آزاد شده می‌تواند وارد قسمت‌های کرنتاسیون نشده بین شود این بکرده موجب افزایش غلظت کلر آزاد در در ضخامت بین پوشش روی آماده شده و ممکن است کر آ در سطح می‌گردد به مقدار بحرانی برساند. بنابراین سازمان‌های حاواری گل که کرنتاسیون در آنها رخ داده است نسبت به سازه‌هایی که فقط از یک معلول رنگ می‌برد، بیشتر در معرض خوردنگی قرار دارد [4].
عمق کربناتاسیون را می‌توان با پاشیدن محلول فلئاتالین بر روی مقطعی یا تحقیق‌داد. در مناطقی که تحت تاثیر کربناتاسیون نیو، مقدار pH بین تقسیم‌بندی بیشتر از 8/5 و فلئاتالین در این مناطق به رنگ ارغوانی در می‌آید.

جدول 2: نتایج آزمایش عمق کربناتاسیون

<table>
<thead>
<tr>
<th>جدول خوردگی (mm)</th>
<th>متوسط عمق کربناتاسیون (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>(a)</td>
</tr>
<tr>
<td>50</td>
<td>(b)</td>
</tr>
</tbody>
</table>

متوسط عمق کربناتاسیون سطح پن مصالح در جدول شماره (2) نشان داده شده است. مطابق جدول (2) در سطحی از بین که خوردگی آرام‌تر مشاهده شده است (b)، متوسط عمق کربناتاسیون 50 میلی‌متر بوده که با توجه به اینکه متوسط ضخامت پوشش بینی آرام‌تر در پیش 50 میلی‌متر بوده، می‌توان تیزی‌های گرفته که آرام‌تریا در این ناحیه تحت تاثیر بیشتر کربناتاسیون قرار گرفته‌اند در ناحیه (a) متوسط عمق کربناتاسیون 30 میلی‌متر و مساوی مقدار پوشش بینی آرام‌تریا بوده در این ناحیه آرام‌تریا در ناحیه آرام‌تریا در این ناحیه آرام‌تر است. قرارگیری در معرض پیشه کربناتاسیون هستند.

3-3 برای تعیین مقدار کلر در بتن، از به‌کارگیری مختلف عرشه پلاک‌های نمونه برداری انجام شد. میزان کلر کل (Total Chloride) موجود، طبق استاندارد ASTM C114 اندام‌گیری شده بر روی پروفیل‌های پون کلر سطح (a) و (b) در شکل (1) به طور نمونه‌دار نشان داده شده است. همانگونه که دیده می‌شود، مقدار پون کلر در نزدیک آرام‌تر (عمق 30 میلی‌متر) در سطح (a) کمتر از مقدار کلر استاندارد (حدود 0/7-0/9 می‌شود) در نتیجه در سطح (a) احتمال خوردگی تحت اثر پون کلر بسیار کم است. در حالیکه در سطح (b) مقدار کلر در نزدیک آرام‌تر به مقدار کلر بسیار نزدیک است. بنابراین در این سطح خوردگی تحت اثر پون کلر بسیار محتمل است و با توجه به میزان عمق کربناتاسیون و مقدار پون کلر در نزدیک آرام‌تر می‌توان نتیجه گیری نمود که کربناتاسیون در ترکیب با پون کلر منجر به کاهش مقدار کلر استاندارد خوردگی و توسه و کسترش خرابی‌ها شده است.

![شکل 1- پروفیل پون کلر تهیه شده در دو سطح (a) و (b)](image1)

![سطح بدون خوردگی](image2)
بررسی خوردوگی اراموتر در عرضه تعدادی از طیف‌های راداهن...

بر اساس پروفیل‌های ارائه شده به نظر می‌رسد مقدار کل اولیه در 170/100 درصد وزنی بین بوده و در حدود مجاز می‌باشد. این در حالت است که کل موجود در عمق 2-3 سانتی‌متر از مخاطب پیرامون وارد بین شده است و به نظر می‌رسد منشا آن کل موجود در شن‌های روان و انصوری در منطقه کوبری و خصوصاً در صورت و مؤثر که با شیوه و رژیم‌های است می‌باشد. علاوه بر این افت قدرت کل در ترکیبی سطح خارجی که مشخصا در شکل (1-b) ملاحظه می‌شود به دلیل یک‌دیده کرنتاسیون می‌باشد که موجب تجزیه نمک فریل و کم شدن گاز می‌باشد و آزاد شدن آن و حرکت نمک‌ها به سمت داخل بین می‌شود.

۳- آزمایش پتانسیل نیم پایل

آزمایش تعبین پتانسیل نیم پایل، آزمایشی غیر مخرب و استاندارد می‌باشد که به طور گسترده در زون‌های وضعیت خوردوگی مورد استفاده قرار می‌گیرد. این آزمایش مطابق با استاندارد ASTM C876-93 نیست. انجام شده است.

به‌این ترتیب داشت که حضور یک‌سیزی، غلظت کل و مقاومت الکتریکی بین تاثیر زیادی بر روی قرانت‌های ناشی از پتانسیل نیم پایل دارد در این آزمایش میزان بررسی خودگی طبق الکترود Ag/AgCl اراموتر چنین می‌باشد: نتایج نیم پایل در ارائه بیش از ۹۰ درصد، هیچ خودگی اراموتر در زمان آزمایش وجود ندارد. 

در شکل (۲) به عنوان نمونه، نتایج آزمایش پتانسیل خودگی اراموتر برای دو سطح مذکور ارائه شده است. در شکل (۲) منفی‌ترین پتانسیل نیم پایل در تابع با خودگی زیاد (مثلاً ۳۲۰ می‌باشد. در حالت در ناحیه در ناحیه بدون خودگی (۴) این مقادیر ۱۴۰-۱۰۰ می‌باشد. با توجه به توضیحات ارائه شده در قبل، در شکل (б)

یا احتمال بیش از ۹۰ درصد، خودگی رخ داده است. در حالت در سطح (a) فعالیت خودگی نامشخص است.

(۳) سطح بدون خودگی

(۴) نتایج آزمایش پتانسیل خودگی اراموتر

(۵) نتایج در سطح (a)
AAD-3

Azmaish تعیین سرعت خوردگی آراماتور

اندازه‌گیری به دستگاه گالواپاس یک روش پلاژرسیون سریع برای تعیین میزان خوردگی در سازه‌های بتن مصالح به‌حساب می‌آید. این روش با بازداری مشکلات اندام‌گیری‌های نیم بیل در محیط‌های مرطوب و یا نیمه مرطوب بعلت کم‌هوایی اکسیژن و همین‌طور سرعت زیاد آن در بست‌آور میزان خوردگی آراماتور را بنت تتوسعه باید است. با استفاده از دستگاه گالواپاس، پتانسیل نیم بیل و سرعت خوردگی آراماتور قابل تعیین می‌باشد.

محدودیت‌های بحرانی برای طبقه بندی ریسک پدیده میزان خوردگی در جدول (۳) نشان داده شده است.[۷]

جدول ۳: تخمین مقدار خوردگی قابل مشاهده از روی قرانت‌های انجام شده با دستگاه گالواپاس

<table>
<thead>
<tr>
<th>مقدار خوردگی ( \mu A/cm^2 )</th>
<th>مقدار اندازه‌گیری شده (( \mu A/cm^2 ))</th>
</tr>
</thead>
<tbody>
<tr>
<td>در حال مقایسه ( \leq 5 )</td>
<td>کم</td>
</tr>
<tr>
<td>( 5 &lt; ) ( \leq 15 )</td>
<td>متوسط</td>
</tr>
<tr>
<td>( 15 &lt; )</td>
<td>زیاد</td>
</tr>
</tbody>
</table>

نتایج آزمایش تعیین سرعت خوردگی آراماتور به طور نمونه در شکل (۳) نشان داده شده است. همان‌طور که در این شکل دیده می‌شود، تاثیر نسبت استحکام خوردگی در سطوح مورد نظر (۳ و ۴) قابل مقایسه با نتایج به دست آمده از پتانسیل نیم بیل (ارائه شده در شکل (۲)) است. حاکم سرعت خوردگی در سطح (۵) بین ۱۶ میکرو آمپر بر سانتی متر مربع بیشتر گزارش شده است و آزمایش نیم بیل نیز احتمال خوردگی را بیش از ۹۰ درصد نشان می‌دهد. این در حالیست که نتایج سرعت خوردگی در سطح (۶) حداکثر ۲/۱۵ میکرو آمپر بر سانتی‌متر مربع است که نشان دهنده مقدار کم می‌باشد. ضمناً مقادیر پتانسیل نیم بیل در محدوده عدم قطعیت قرار دارد.

پتانسیل نیم بیل در محدوده عدم قطعیت قرار دارد.

![Image](image-url)
نتیجه‌گیری
1. ارزیابی خرابی‌های بتن مسلح به منظور ترمیم آن، باید در یک نظام منظمی اجرای شود. یک ترمیم با کیفیت نیازمند شناخت علت وقوع خرابی، مشاهدات کامل آسیب‌ها، مشخصات دقیق محل‌های آسیب‌دیده است.

2. با توجه به دست آمده عامل اصلی وقوع خورده‌گی در بتن مسلح عرشه بی‌پی و نمایندگی گیری ۱-

۳ مقدار پتانسیل نیم پیل تناها احتمال خروج‌گری را نشان می‌دهد و حتی در برخی از حالات استفاده از این روش

۴ مقدار محدودیت‌های همراه است. در حالیکه با استفاده از روش سرعت خودگی می‌توان مقدار خودگی را

۵ کلریون و غلظت بین‌نشین می‌باشد. مقادیر پتانسیل نیم پیل تناها احتمال خروج‌گری را نشان می‌دهد و حتی در برخی از

۶ پیدا کردن‌ود و در برخی از حالات استفاده از این روش

۷ بر اساس مطالعات انجام شده در قابل این تحقیق مشخص می‌شود که خورده‌گی ناشی از نفوذ بیون کلر در

۸ این نوع خرابی‌ها وجود دارد. بدین‌هی است اجرای سازه‌ها و این‌به بینی با کیفیت نازل و نسبت آب به مواد

۹ سیمانی زیاد موجب بروز پیداید کردن‌ود و تشدید فاکتور خودگی و آسیب‌دیدگی بتن خواهد شد.

مراجع


7. برگر، فرهاد، اردبیلی‌رفت، ۱۳۷۵، «بررسی پارامترهای موثر بر غلظت کلر برابری برای شروع خورده‌گی

8. میلک‌دها فولاده در بین‌»، پایان‌نامه کارشناسی ارشد در رشته مهندسی عمران، دانشگاه فنی دانشگاه

برکن.
بحث ریزی در هواهای سرد و اقدامات اصلاحی در جهت کاهش مدت زمان اجرای اسکلت بتنی با سیستم قالب تولید

سید اسدی

چکیده

از نظریات در سیستم قالب تولید همراه همراه موضوع مدیریت زمان مطرح می‌باشد. لذا بتن ریزی در هواهای سرد از اهمیت خاصی برخوردار بوده و با توجه به سرمایه شدید در منطقه آذری‌چایان که معمولاً بتن ریزی در استان بر روی مخزن می‌باشد، با اجرای تجهیزاتی آقای ساختمان و نصب هنرهای برقراری فن دار در داخل تولید و پوشش‌های مناسب و استفاده از مشعل و لوله گرمایشی و آمکان بتن ریزی در هواهای سرد و بالابردن مقاومت بتن می‌سر می‌گردد.

1 - تامین ضوابط آین نامه‌ای مطلق مبوت 9 مقررات ملی ساختمان هواهای سرد به وضعیت اطلاق می‌شود که برای 3 روز منفی شرایط زیر برقرار باشد.

الف - دمای متوسط هوا در شیب‌های روز کمتر از 5 درجه سلسیوس باشد.

ب - دمای هوا برای بیشتر از نصف روز از 10 درجه سلسیوس زیادتر نباشد.

به منظور اطمینان کافی از وضعیت نگهداری بتن در پروژه آذران دمای بتن در طول شب هنگام بتن روز 2 بار در نقاط مختلف سازه ثبت می‌شود.

برای رساد‌دندی دمای بتن به حداکثر هنگام اختلاف بتن از آب گرم استفاده شده و مطابق آین نامه، از تماس
مستقیم آب گرم با سیمان در صورتی که دما آن بیش از ۴۰ درجه سلسیوس باشد در حین ریختن مصالح در
مخلوط کن، کنترل لازم بعمل می‌آید.
انتخاب نسبت آب به سیمان با توجه به روند کسب مقاومت در دما می‌باید و کمتر از ۰/۴ می‌باشد.
عمل آوردن بتن تازه در حال بند ۳۴ ساعت و تا رسیدن به مقاومت ۵ مگاپسکال ادامه می‌باشد.
عمل آوردن بتن با استفاده از بوش (بکارگیری پشم شیشه که در داخل ناپل زده داده می‌شود) و گرم کردن داخل
تولیدها با هیترهای برقبی و گازی تا ۱۵ درجه سانتی گراد انجام می‌گیرد.
مزیت استفاده از هیترهای برقبی و گازی نسبت به گرم کننده‌ها با مواد سوختی نظیر نفت و گازوئیل به این علت
است که از احتمال تبخیر آب و کرکنی شدن سطوح بتن در آت احراز جلوگیری بعمل آید.
با توجه به اینکه در سیستم قالب تولیدی به جهت افزایش سرعت عملیات اجرایی، باز کردن قاب‌ها در مدت زمان
کمتری از زمان تعریف شده در مبحث ۹ مقررات ملی انجام می‌شود، لذا از مواد روان کننده که خاصیت
تندیگر کننده دارند استفاده و برای مراقبت بتن بعد از قالب برداری از پایه‌های اطمینان در وسط دهانه‌ها استفاده
شد و وقت لازم بعمل می‌آید تا همیشه پایه‌های اطمینان در دو طبقه متوالی وجود داشته باشد و تا حد امکان
هر دو پایه اطمینان نظیر در دو طبقه، روی هم و در اتمادی واحد قرار گیرند.
2- تهیه ابگرم بچینگ
برای تهیه ابگرم لازم در بچینگ از نانک آب و استاندارد آن و محیط نمونه به سیستم گر سالش یوسیله مشعل‌های گازی، همراه با نصب ترمومتر جهت رسیدن دمای آب بین بی‌بی به ۸۰۰ سانتی‌متر شد.

مشعل گرمایشی جهت حرارت به قسمت بالای نانک آب بچینگ

جلوگیری از بخ‌زدن مصالح بچینگ
یا اجرای لوله‌کشی در محل دیوار مصالح بچینگ و نصب گرم کن با مشعل گازو ثانی و عبور دادن هوای گرم از

پلک B1 - زمستان ۸۶

مشعل گمایشی بی‌بی مشعل

پلک بچینگ - زمستان ۸۷
داخل لوله‌ها از بخ زدگی مصالح جلوگیری می‌شود.

جهت حفظ حرارت در محل دیوی مصالح بچینگ، سطح مصالح را با رول‌های پشم شیشه (با کاور ناپاپوئی) پوشانیده و بر روی آن چادر برگزتی کشیده می‌شود. در قسمت پایین با گذاشتن یک خرک و ایجاد اختلاف ارتفاع، با قرار دادن هیترهای برقی فن دار، سطح فوقانی مصالح گرم نگه داشته می‌شود.

# تعمین گرماپیش داخل قالبهای تونلی
جهت جلوگیری از افت دمای بنن هنگام بنن ریزی حدود ۱۲ ساعت قبل از بنن ریزی با قرار دادن هنترهای برقی فن دار در داخل قالب‌های تونلی و استقرار آنها در قسمت خروجی با جاده، گرماشی داخل قالب تونلی تامین شده و دمای قالب‌ها به ۱۰۰° می‌رسد.

هیتر برقی فن دار

این‌هم‌اوان‌ها قسمت خروجی قالب تونلی

۵- عمل اوری و گنترل دمای بنن

به منظور جلوگیری از افت دمای بنن در محل بنن ریزی، مقدار بهره بنن در بچینگ در هر مرحله به مترمکعب تقیف و بین ترتیب مدت زمان حمل مصرف بنن کاهش می‌یابد.

پوشش روی بنن توسط پشم شیشه و برزنت

پس از اتمام بنن ریزی، گرماشی داخل قالب‌های تولنی تا مدت ۴۸ ساعت بعد از بنن ریزی ادامه یافته و سطح بنن با پشم شیشه پوشانیده و بر روی پشم شیشه نیز همانند پوشش روی مصالح پچینگ، جاده کشیده می‌شود و
آرمانهای انتظار ریشه دیوآرما به دلیل پوشانده می‌شود. با رعایت موارد فوق الذکر تا دمای ℃ برگزیده و در زمستان سال 86 که متوسط دمای شهر جدید سهند بددمای ℃ رعایت، کل تعطیلی پروژه به مدت 8 روز آن هم بدلیل کولاک شدید بود.

1 - اقدامات انجام گرفته چه تاثیر مدت زمان اجرای اسکلت بتنی

در راستای تحقق و نیل به اهداف کیفی تعبیه شده از طرف شرکت مادر در سال 87 که کاهش مدت زمان اجرای اسکلت بتنی برگزیده، از همین ابتدا سال 87 برای هر یک از بلوق‌ها مطابق نمودارهای پیوستی (صفحات 52 تا 54) جهت مقایسه مدت زمان اجرای اسکلت تهیه گرددید به طوریکه بیشترین مدت زمان اجرای اسکلت مربوط به بلوق A1 در مدت 128 روز و کمترین مدت اجرا مربوط به بلوق A3 در مدت 33 روز انجام گرفته است.

بیشترین زمان اجرای یک می‌نماید در نیمCU به مدت 8 روز و با مطالعه و رفع موانع، در بلوق B3 به مدت 2 روز.

رسید

بطور خلاصه مواردی که در سرعت دادن کار اجرایی اسکلت نقش اساسی داشته‌اند به شرح زیر می‌باشد:
1- افزایش سرعت بتن یاکت با اضافه کردن حجم پاکت به میزان 1/8 مترمکعب

افراشحجم یاکت بتن به مقدار 1/8 مترمکعب

2- کوتاه نمودن مسیر حمل بتن و اضافه نمودن تراک میکسر جهت حمل بتن.

3- افزودن تعداد نفرات به‌پیمانکار

4- با منظور جابجایی قالب‌های کاری، مبایل و باکس‌های مختص به همان قالب استفاده می‌شود که با و بسته کردن مجدد سیم باکس‌ها باعث کننده عملیات می‌گردد. با تجمع کل سیم باکس‌ها در قالب تاور کردن و تمویض از همان قسمت سرعت جابجایی قالب‌ها افزایش یافته.

5- انجام عملیات تمیز کاری و روغن کاری قالب‌ها به هنگام خروج پرموی پلت فرم (PLAT FORM) و انتقال به نیم ست دیگر (توضیح اینکه قیل کار تمیز کاری و روغن کاری قالب‌ها پس از انتقال آنها به سطح زمین صورت می‌گیرد و سپس به نیم ست بعدی انتقال داده می‌شود)
تمیزکاری و روانگی کاری قابل‌ها در روزی بین‌فرم‌ها

6- تعلیم و آموزش کارگران و استادکاران جهت افزایش بینش آنها به احداث ساختمان با دید صورتی
7- تعریف وظیفه و تخصص خاص برای هر نفر (متلا نصب باز شو، نصب قالب‌های کناری و غیره)
8- افزایش ساعت کاری کارگاه (شروع تایم کاری در تاکستان از ساعت 6 صبح تا 20 شب)
9- انتخاب پیمان‌کار جدایان جهت اجرای کلیه عملیات جوش‌کاری در دیوارها و سقف‌ها

2- مطالعه و بررسی در سرعت دادن به عملیات الکتریکی

1- لوله‌های تولیدی کارخانه بزرگ پولی‌کا از نوع PVC طلسمی نسوز که قبلا استفاده می‌شد و مقاومت فشاری عمودی کافی نداشت با نوع پلی‌کای PVC سخت نسوز سفید تولیدی کارخانه تابع بی‌اثنی عضوی شد که این امر باعث کاهش تعداد لوله‌های شکسته شده در مرحله اجرای مش دوم سقف و بتن ریزی گردید.

SHOP DRAWING

3- تهیه نقشه ثابت

3- تهیه شالن و فنر خم مناسب با سایز لوله جهت ایجاد خم لوله‌های برق دیوارها

خم‌گاری لوله‌ها با استفاده از فنر و شالن خم
بنریزی در هواهای سرد و اقدامات اصلاحی در جهت کاهش....

4- تهیه شیلین جهت ساخت بخش‌هایی سر ولوله‌های برق، دیوارها بصورت سری

5- سوراخ کاری محل ورود ولوله‌ها به قوطی‌های کلید و بریز به صورت سری

6- تهیه بیرا سفوم جهت محل قرارگیری ولوله‌های انظار سقف و دیوار و محل باز شویا با توجه به ابعاد آنها و ایجاد سرواخه‌های لازم در آنها به صورت سری.

استفاده از بیرا سفوم جهت محل قرارگیری ولوله‌های انظار دیوار

7- علاطم گذاری ولوله‌ها با رنگ‌های منسوب برای شناسایی محل مستمایی مختلف، با جمله روشانتان بریز برق و تلفن و انتخاب بار کن برای تسرب در ولوله‌های سقف در کارگاه موکت.

8- کد گذاری و دسته بندی ولوله‌های خم و غلاف‌های جفت و نگه و قوطی‌های سیرا، با توجه به تعداد سایز آنها در نقشه جهت جلوگیری از اشتباه نفرات. نفرات می‌توانند در مرحله اجرای SHOP DRAWING تهیه جهت جلوگیری از اشتباه SHOP DRAWING 9- کد گذاری و دسته بندی با توجه به تعداد آنها در نقشه، نفرات بیماری در مرحله اجرای SHOP DRAWING می‌گردد.

10- ساخت غلاف خالی قوطی‌های کلید و بریز از لوله سیک نرده ای به قطر 1/4 اینچ و جوش 2 عدد میلی‌گرم نمره 5 بطول 40 سانتی‌متر و بسندر آن با سیم ارمان‌برندی به مش دیوارها، جهت قرارگیری محل دیق قوطی کلید و بریز و فصله از دیواره قابل توالی.
11- حضور مستمر ناظر تناسبات برقی در زمان بین ریزی و آموزش یک نفر به عنوان کارگر فنی جهت کنترل و ترمیم لوله‌ها در صورت شکستگی لوله‌ها در حین بین ریزی

3- مطالعه و بررسی در سرعت دادن به عملیات مکانیکی

1- لوله کنگی سیستم فاضلاب و ونت (لوله‌های به قطر 50 تا 72 میلیمتری) معمولاً با انجام شیارکنی و برش کاری دیوارهای داخلی و خارجی صورت می‌گیرند که موجب تخریب دیوارها و افزایش نخله می‌شود. در پروژهی آذران یک غلاف از جنس لوله PVC از سایر بزرگتر از لوله مورد نظر همزمان با اجرای دیوارها در داخل دیوار قرار داده و بعد از اتمام دیوارچینی غلاف را برداشت و برای استفاده مجدد به طبقه دیگر انتقال می‌یابد که باید ترتیب از تخریب دیوار جلوگیری می‌شود.
ابتدا از غالبه جهت حلگرزی از تخریب دیوارها در هنگام لوله‌کشی

2- محل عبور لوله‌های آب شهری به داخل و احجام، مطالب نقشه‌های اجرایی در مشاعات و داخل واحدها به صورت زیرسقفی مشخص و از نظر معمای نیاز به اجرای سقف کاذب دارد، در بلوک‌های تیپ A لوله‌های نماین به مقدار ۴۲۳۷ مترطول و در تیپ B به مقدار ۱۶۲۳۰ مترطول می‌باشد. با تغییر نقشه‌های اجرایی و اجرای لوله‌ها بصورت تکرار از هزینه مربوط به خرید مصالح و دستمزد اجرای سقف کاذب شامل (خزید مصالح از قرار هر مترطول به مبلغ ۱۲۰۰۰ ریال - رابطه بیندی ۵۰۰۰ ریال - گچ و خاک ۲۵۰۰ ریال - دستمزد اهیزه کشی به مبلغ ۳۰۰۰۰ ریال) جمعاً به ارزی هر مترطول به مبلغ ۱۰۰۰۰۰۰ ریال، به مبلغ کل ۵۰۰۰/۲۰۰۰/۵۰۶۵/۰۰۰۰۰ ریال صرفه جویی گردید.

3- اجرای سایپورت لوله‌ها در داکت‌های ناسیسات معمول، پس از اتمام اسکلت انجام می‌شود. در پروژه آذرین عملیات آهن کشی داکت‌ها با استفاده از مصالح پرت حاصل از نیشی کشی دیواره‌های خارجی، هم‌زمان با اجرای اسکلت با قرار گرفتن شاخک‌های فلزی در داخل بین اجرای می‌شود.

4- جهت کاهش عملیات آهن کشی در چاه‌کسی آسانسور بلوک‌های تیپ A و B، دیواره‌های پشتی در
طبیعتی (یک ضلع در بلوک A و سه ضلع در بلوک B) پلیت گداری شده و از حجم عملیات اهن کشی به مقدار ۶۰ درصد در بلوک‌های TIP B و ۳۰ درصد در بلوک‌های TIP A کاسته شده است.

پلیت گداری دیوار بتنی چاپک‌های اسناسور

5- نقشه‌های تاسیسات مکانیکی در رابطه با هماهنگی محل اولین‌گهداری (تاسیسات نازی) با نقشه‌های معمولی و سازه و با در نظر گرفتن طول اتصالات مصالح مصری در لوله کشی سیستم فاضلاب سروپس‌ها و رایزنی‌های آب باران اصلاح شده و در این رابطه نقشه‌های ازبینته تهیه گردید. SHOP-DROWING

6- جهت اجرای پاروپاهمیه مربوط به تاسیسات مکانیکی در سقف‌ها در زمان بتن ریزی سقف، نسبت به ساخت قالب‌های قلبی مربوط به محل عبور لوله به سایز بیشتر از قطر لوله‌ها به شکل کوینک اقدام گردید. برای اینکه محل پاروپاهمیه در طیف‌های در انتظار هم می‌باشد، مختصات مرکز پاروپاهمیه در سقف قالب تولید سروآخ کاری شده و با قرار گیری بین قالب پاروپاهمیه (میلیگرد نمره ۱۶) در سوراخ مذکور و بعد از کشیدن سقف، قالب مذکور برای استفاده در سقف طبقه بعدی برداشت می‌شود.
CD04
Seismic Evaluation and Rehabilitation
UPGRADING THE DUCTILITY AND SEISMIC BEHAVIOR FACTOR OF ORDINARY RC FRAMES USING FIBER COMPOSITE SHEETS

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²Faculty of Engineering, University of Bristol, UK

ABSTRACT
The ductility and seismic behavior factor (R) are evaluated for an existing Reinforced Concrete (RC) frame that has been retrofit with web-bonded Carbon Fiber Reinforced Polymer (CFRP) system. For this purpose, firstly using a nonlinear finite element analysis the flexural stiffness of FRP-retrofitted and original exterior and interior joints of the frame are determined. The obtained stiffness is then implemented into another software package in order to analyze the FRP-retrofitted frame using nonlinear static analyses. Then the $R$ factor components including ductility reduction factor and over strength factors are extracted from pushover analyses. The results are compared with the results of the original RC frame and the same frame that has been retrofitted with steel bracings reported by other investigators. The results show that the ductility and the seismic behavior factor of the existing RC frame that has been retrofitted with CFRP sheets are better than the original frame, and upgrade the ductility of the ordinary RC frame to the Intermediate and even Special RC frame.

Keywords: seismic behavior factor (R), nonlinear static analysis, pushover, reinforced concrete frame, web-bonded CFRP-retrofitting, steel bracings

1. INTRODUCTION
Recently, FRP has been utilized for retrofitting or upgrading RC structures. Parvin & Granata [1] indicated that when joints of an RC frame were reinforced with FRP laminates, the moment capacity was increased up to 37 percent. Mahini & Ronagh [2] used a method for strengthening of exterior beam-column joints using web-bonded FRP sheets. They tested the effectiveness of web-bonded CFRP on energy absorption capacity of 1/2.2 scale RC joints, in order to evaluate the possibility of relocating the plastic hinge location away from the column face. Their experimental studies showed that the FRP repairing/retrofitting system can restore/upgrade the integrity of the joint, keeping/upgrading its strength, stiffness and ductility as well as shifting the plastic hinge from the column facing toward the beam in such a way that the joint remains elastic. The practicality and effectiveness of using web-bonded FRPs on plastic hinge relocation has been also reported by Smith and Shrestha [3]. In another experimental study Balsamo et al. [4] evaluated
the seismic behavior of a full-scale RC frame repaired using CFRP laminates. They indicated that the repaired frame had a large displacement capacity without exhibiting any loss of strength, while providing almost the same energy dissipation of the original frame.

In this paper, seismic behavior factors affecting parameters for CFRP-retrofitted ordinary moment-resisting RC frame are evaluated and compared with corresponding original moment resisting and steel-braced RC frames. The $R$ factor components including ductility reduction factor and over strength factor are extracted from nonlinear static analyses of the frames. For this purpose, an eight storey three bay existing RC moment resisting frame which was retrofitted by Maheri & Akbari [5] using steel bracing systems is retrofitted again with web-bonded CFRP method in order to compare their ductility and seismic behavior factor.

2. GEOMETRY AND MATERIAL PROPERTIES OF THE RC FRAME

Figure 1 shows the selected frame of this study. The design dead and live loads are assumed to be 2750\(\text{kg/m}^2\) and 1750\(\text{kg/m}^2\) respectively. The compressive strength, $f'_c$ and tensile strength, $f_t$ of the concrete are taken as 27.46 MPa and 3.668 MPa, respectively. In addition, the elastic modulus of the concrete $E_c$ is taken as 24.63 GPa and the yield stress of steel reinforcement is assumed to be 412 MPa.

Design base shears were determined for a Peak Ground Acceleration (PGA) of 0.3\(g\) . The weight of the system is taken as the dead load plus 20 percent of live load as an estimation of the equivalent earthquake load, based on the Iranian earthquake code [6]. Initial $R$ factor was assumed to be equal to 6 for this system. The moment resisting frame was designed based on "weak beam-strong column" principle using ACI-95 Code [7] and the steel bracings system was designed using
AISC-LRFD Code [8]. Dimensions and flexural reinforcements of the designed beam and column sections are shown in Figure 1. In this Figure, \( \rho_c \), \( \rho_s \) and \( \rho_s' \) are the total steel ratio of column, tensile and compressive steel ratio of the beam respectively. All members and joints reinforcements have been designed to achieve the desirable strength and ductility [5].

3. NONLINEAR FINITE ELEMENT ANALYSIS OF RC JOINTS
The models of typical exterior and interior joints are shown in Figure 2. It can be seen that ten different models have been analyzed by finite element method for both original and retrofitted joints.

The behavior of the RC joint retrofitted with web-bonded CFRP is analyzed using ANSYS software [9]. Both material and geometric nonlinearities are taken into account in the nonlinear finite element analysis by ANSYS. In order to model the characteristics of concrete, ANSYS SOLID65 elements is used. This element is capable of simulating the cracking and crushing of the concrete. Furthermore, to model the longitudinal reinforcement and the FRP composites, LINK8 and SOLID45 elements, are used respectively [1]. The FRP length has been chosen based on the Paulay and Priestly [10] design approach for obtaining the desirable plastic hinge relocation. For verification of the modeling and the analysis for the RC joint, an experimental study conducted on an exterior RC joint by Mahini & Ronagh [2] was selected. Figure 3 shows the “Beam tip load – Displacement” curves extracted from the non-linear FE analysis and the experimental data. Considering satisfactory agreement could be observed between the curves, it is concluded that the presented FE modeling is reliable. The required FRP thickness was obtained using nonlinear finite element analysis according to the desirable plastic hinge relocation. The required FRP thickness in the first level was calculated to be 4.95 mm, decreasing as the height of the frame increases. Other
characteristics of CFRP laminates are given in Table 1. Note that the characteristics given in Table 1 satisfy the consistency conditions which are necessary for a non-isotropic material like ANISO in the analysis as described in reference [9] and stated by Kachlavec et al. [11]. The numerical models of retrofitted exterior and interior joints at seventh floor of the selected frame are depicted in Figure 4. Figures 5 and 6 show the failure mechanism of retrofitted exterior and interior joints (Isosurface style of concrete strain) at seventh level of the selected frame before and after retrofitting by web-bonded CFRP sheets. It can be seen that desirable plastic hinge relocation is achieved successfully thanks to CFRP sheets, as it was already obtained from an experimental study by Mahini & Ronagh [2]. Figure 7 shows the failure mechanism of an exterior RC joint tested by Mahini & Ronagh [2] before and after retrofitting by web-bonded FRP sheets.

Figure 3. “Beam tip load – Displacement” curve for an exterior RC joint from experiment [2] and calculated from FE analysis.

Figure 4. Finite element models of an (a) exterior and (b) interior retrofitted joint.
Table 1: Mechanical properties of CFRP sheets used for FE modeling [9]

<table>
<thead>
<tr>
<th>Modulus of elasticity (MPa)</th>
<th>In fibers direction</th>
<th>Perpendicular to fibers direction</th>
<th>Compressive strength (MPa)</th>
<th>In fibers direction</th>
<th>Perpendicular to fibers direction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic modulus (<em>E</em>)</td>
<td>E_x=240000</td>
<td></td>
<td>E_x=18581</td>
<td>E_x=18581</td>
<td></td>
</tr>
<tr>
<td>Perpendicular to fibers direction</td>
<td>E_y=18581</td>
<td></td>
<td>E_y=18581</td>
<td>E_y=18581</td>
<td></td>
</tr>
<tr>
<td>In fibers direction</td>
<td>σ_x=80</td>
<td></td>
<td>σ_x=80</td>
<td>σ_x=80</td>
<td></td>
</tr>
<tr>
<td>Shear modulus (MPa)</td>
<td>G_0x=12576</td>
<td></td>
<td>G_0x=12576</td>
<td>G_0x=12576</td>
<td></td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>v_x=0.2</td>
<td></td>
<td>v_x=0.2</td>
<td>v_x=0.2</td>
<td></td>
</tr>
<tr>
<td>Tensile strength (MPa)</td>
<td>σ'_x=3900</td>
<td></td>
<td>σ'_x=53.7</td>
<td>σ'_y=53.7</td>
<td></td>
</tr>
<tr>
<td>Shear modulus (MPa)</td>
<td>G_0y=7147</td>
<td></td>
<td>G_0y=7147</td>
<td>G_0y=7147</td>
<td></td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>v_y=0.3</td>
<td></td>
<td>v_y=0.3</td>
<td>v_y=0.3</td>
<td></td>
</tr>
</tbody>
</table>

Figure 5. Failure mechanism of an exterior joint (a) before and (b) after retrofitting by web-bonded CFRP sheets

Figure 6. Failure mechanism of an interior joint (a) before and (b) after retrofitting by web-bonded CFRP sheets

Figure 8 shows the "moment-rotation" curves of an exterior original and FRP-retrofitted beam-column joint at seventh level of the selected frame. In this Figure, $K_{ij}$ is the difference between the two curves.
3. NONLINEAR STATIC ANALYSIS OF THE FRAMES

- **Original Frame**: Nonlinear static analysis (pushover) of each system is carried out using SAP 2000 10.1.0 program [12]. For this purpose, a constant gravity load equal to the total dead load plus 20 percent of the live load is applied to each frame, and an inverted triangular distribution over the height is used as the lateral load pattern. $P - \Delta$ effect is also considered in the analysis. Force-deformation criteria for plastic hinging is defined based on ATC-40 [13] and FEMA356 [14] patterns.

- **Retrofitted Frames**: The analytical models of the retrofitted frames with web-bonded CFRP system and steel bracings are shown in Figure 9. This frame has already been retrofitted using steel bracing system (Maheri & Akbari [5]). In order to model the FRP-retrofitted frame, SAP 2000 Non-Linear Link (NLLink) elements are used, which can simulate the equivalent additional stiffness to the beams provided by web-bonded CFRP sheets on the system. These elements are assumed to be located at a distance of 500 mm away from...
the column face, corresponding to the FRP length. In Figure 9, $K_i$ is the additional rotational stiffness of each retrofitted beam, which is modeled on the original frame with a NLLink element. The "moment-rotation" curve of original and FRP retrofitted joints are extracted from finite element analysis and the differences are used as the rotational stiffness of retrofitted joints.

![Figure 9](image)

**Figure 9. Analytical modeling of the (a) web-bonded CFRP (current study) and (b) steel-braced frame [5]**

The base shear versus roof displacement curves of original and retrofitted (both steel-braced and FRP-retrofitted) frames are shown in Figure 10. In this Figure, X-brace retrofitting systems examined by Maheri & Akbari [5] have been designed based on 50% and also 100% of the lateral loading on the RC frames.

![Figure 10](image)

**Figure 10. Base shear-roof displacement curves of all frames**
4. SEISMIC BEHAVIOR FACTOR AFFECTING PARAMETERS

In forced-based seismic design procedures, seismic behavior factor, \( R \) is a force reduction factor used to reduce the linear elastic response spectra to the inelastic response spectra. In other words, seismic behavior factor is the ratio of the strength required to maintain the structure elastic to the inelastic design strength of the structure. The seismic behavior factor, \( R \), therefore accounts for the inherent ductility and over strength of a structure as well as the difference in the level of stresses considered in its design. Taking into account the above three components, it is generally expressed in the following,

\[
R = R_\mu R_s Y
\]  

(1)

Where, \( R_\mu \) is the ductility-dependent component, also known as the ductility reduction factor, \( R_s \) is the over strength factor and \( Y \) stands for the allowable stress factor. With reference to Figure 11, where the actual "force-displacement" response curve is idealized by a bilinear "elastic-perfectly plastic" response curve, the seismic behavior factor parameters may be defined as:

\[
R_\mu = V_e / V_y, R_s = V_y / V_s, Y = V_s / V_w
\]  

(2)

Where, \( V_e \), \( V_y \), \( V_s \) and \( V_w \) denote the elastic response strength of the structure, the idealized yield strength, the first significant yield strength and the allowable stress design strength, respectively. For structures designed using an ultimate strength method, the allowable stress factor, \( Y \), becomes unity and the seismic behavior factor is therefore reduced to:

\[
R = R_\mu R_s = \left(\frac{V_e}{V_y}\right) \left(\frac{V_y}{V_s}\right) = \left(\frac{V_e}{V_s}\right)
\]  

(3)

The structure ductility, \( \mu \), is defined in terms of the maximum structural drift (\( \Delta_{\text{max}} \)) and the displacement corresponding to the idealized yield strength (\( \Delta_y \)) as:

\[
\mu = \frac{\Delta_{\text{max}}}{\Delta_y}
\]  

(4)

Many investigators have discussed the two main components of \( R \) factor presented in Eq. (3). In particular, the ductility dependent component, \( R_\mu \), has received considerable attention. Ductility reduction factor \( R_\mu \) is a function of both of the characteristics of the structure, including ductility, damping and fundamental period of vibration (\( T \)), and the characteristics of earthquake ground motion. Nassar and Krawinkler [15] presented a relation for \( R_\mu \) in the following form:
\[ R_\mu = [c(\mu-1)+1]^{1/c} \]  

Figure 11. Typical pushover response curve for evaluation of behavior factor, \( R \) \[5\]

Where,

\[ c(T, \alpha) = \frac{T^a}{1 + T^a} + \frac{b}{T} \]  

In Eq. (6), \( \alpha \) is the post-yield stiffness given as a percentage of the initial stiffness of the system and \( a \) and \( b \) are parameters given as functions of \( \alpha \) that can be obtained from Table 2 \[16\].

<table>
<thead>
<tr>
<th>( \alpha ) (%)</th>
<th>( a )</th>
<th>( b )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1</td>
<td>0.42</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>0.37</td>
</tr>
<tr>
<td>10</td>
<td>0.8</td>
<td>0.29</td>
</tr>
</tbody>
</table>

5. DETERMINATION OF SEISMIC BEHAVIOR FACTOR

A number of performance parameters may govern the capacity of a structure. In order to carry out a nonlinear static analysis, one or a number of these parameters should be considered for determination of the displacement limit state \( \Delta_{\text{max}} \). For the medium-rise ductile building considered in this study, the global drift (maximum roof displacement) is commonly used as a failure criterion. In evaluation of the displacement ductility, \( \mu \), the ultimate capacity of each frame is assumed when the global drift has been reached to 1.5% of the system height. This criterion is based on the NEHRP recommendations \[17\] for RC moment resisting frames. The idealized "force-displacement" (obtained based on the FEMA-356 method) and the capacity curves for the FRP-retrofitted frame are shown in Figure
12. In this Figure, \( V_y \) and \( \Delta_y \), are yield strength and yield displacement, respectively, and \( \Delta_t \) and \( V_t \) are the target displacement and its corresponding base shear. To calculate the yield displacement, \( \Delta_y \), and yield strength, \( V_y \), line segments on the "force–displacement" curve were located using an iterative procedure that approximately balanced the area above and below the curve [14]. The effective lateral stiffness, \( K_e \), shall be also taken as the secant stiffness calculated at a base shear force equal to 60% of the effective yield strength of the structure [14]. After calculation of \( \mu \), now \( R_s \) can be obtained from Eq. 5, 6 and Table 2 and \( R_s \) is determined from Eq. 2. The seismic behavior factor parameters of all systems have been presented in Table 3. It can be seen that the ductility ratio of FRP-retrofitted frame is improved in comparison with the original frame, and is very similar to the one obtained for the X-Braced frames. The R factor of FRP-retrofitted frame is improved significantly in comparison with the original frame and is also better than X-Braced frames.

Figure 12. Capacity and Idealized curve of FRP retrofitted frame based on the FEMA356

<table>
<thead>
<tr>
<th>Frame</th>
<th>( \mu )</th>
<th>( R_s )</th>
<th>( R_t )</th>
<th>( R )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Original [5]</td>
<td>2.27</td>
<td>2.4</td>
<td>1.92</td>
<td>4.6</td>
</tr>
<tr>
<td>Xbraced-50% [5]</td>
<td>2.7</td>
<td>2.66</td>
<td>2.97</td>
<td>7.9</td>
</tr>
<tr>
<td>Xbraced-100% [5]</td>
<td>2.84</td>
<td>2.86</td>
<td>3.33</td>
<td>9.5</td>
</tr>
<tr>
<td>FRP Retrofitted</td>
<td>2.83</td>
<td>3.016</td>
<td>3.193</td>
<td>9.63</td>
</tr>
</tbody>
</table>
6. CONCLUSIONS
Conventional retrofitting systems in earthquake-resisting frames have some limitations. For example, conventional steel bracings in RC frames which were considered in this paper for verification of web-bonded CFRP retrofitting are able to dissipate considerable energy by yielding under tension, but they buckle without much energy dissipation in the compression loads [18]. In this paper, an eight-storey frame that was previously strengthened with steel bracings system is selected and retrofitted with web-bonded CFRP. In order to estimate the flexural stiffness of the FRP retrofitting system, nonlinear finite element analysis by ANSYS is employed. The additional flexural stiffness of the FRP joints is implemented into the frame using NLLink elements on the beam end of exterior and interior joints. A systematic evaluation of each system including ductility ratio and seismic behavior factor is made using nonlinear static analysis. Based on the obtained results, it is concluded that the ductility ratio and the seismic behavior factor of the FRP retrofitted RC frame are significantly improved in comparison with the original frame and increased from 4.6 (original frame) to 9.63 (FRP retrofitted frame).

REFERENCES
7. ACI Committee 318. Building Code Requirements for Reinforced Concrete (ACI 318-95) and Commentary (ACI 318R-95). American Concrete Institute, Detroit, Michigan, 1989.
APPLICATION OF EXPANSIVE AGENT (EA) TO PRODUCE POST TENSIONING FORCE IN FRP JACKETS FOR LATERAL RETROFITTING OF RC COLUMNS

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ABSTRACT
Repair and strengthening of RC structures using FRP materials is one of the new and effective techniques ever used. With this respect, strengthening of structural elements by external lateral confinement can lead to increased strength and ductility. In this way, using an efficient and optimized method for lateral confinement is of great importance.

A novel, economic and simple technique for the repair and strengthening of RC members by means of expansive agent (EA) to produce post tensioning force, has been proven to perform well following experimental work undertaken at the University of Sheffield. The aim of the technique is to ensure the enhancement of the member strength and ductility by localized strengthening.

The above technique is equivalent to increasing the effectiveness of the composite confinement and it becomes possible to strengthen large columns with smaller amounts of composites, which are utilized earlier at much higher strengths. In addition, the level of axial strain achieved at failure is improved significantly. The paper will present details of experimental work with different types of confining material (glass and carbon), amounts of reinforcement and levels of initial pre-stressing.

Keywords: strengthening, expansive agent, post tensioning, FRP jacket

1. INTRODUCTION
Since the 1995 Kobe earthquake in Japan, composites have started being used for the repair and strengthening of columns against seismic actions. The composites are applied as external lateral reinforcement and are often used to prevent shear and anchorage/splicing failures, which can result in the enhancement of the ductility of RC elements.

The Japanese philosophy on earthquake resistant design aims to achieve ductility through low ratios of reinforcement and low steel strengths. This results in very large sections, which as a result require little shear reinforcement. However, during seismic violent load reversals, the shear demand can be higher than estimated, for example as a result of vertical accelerations. In addition, splicing of reinforcement causes additional bursting forces, which are difficult to contain with nominal shear
reinforcement. External lateral confinement can address both of the above problems, in addition to providing other benefits.

In Europe and New Zealand, the seismic philosophy for achieving ductility is different, relying more on increasing the non-linear concrete strains through concrete confinement [3]. The advantage of this philosophy is that apart from smaller cross-sections, a significantly higher amount of lateral reinforcement is required. This lateral reinforcement, which is there to confine the concrete, is also beneficial in resisting additional shear and preventing splice and anchorage failures.

The enhancement of concrete ductility by confinement is central to the principles of Eurocode 8. However, researchers dealing with FRP confinement do not always consult the huge wealth of published work, which is derived from the earthquake engineering research. In addition, there is a fundamental difference between mild steel confinement and high strength composite materials.

Mild steel reinforcement attains its yield strength (around 80% of its ultimate strength) at a strain of around 0.002, whilst composites fracture at strains ranging from 0.014–0.02. Unconfined concrete crushes when the lateral strain is at best 0.001. This means, that for low levels of confinement, steel is relatively well utilised (around 50%), whilst composites are at best utilised at 7% of their capacities.

2. MATERIALS
The properties of the fibres and resin used for confinement are shown in tables 1 and 2. Where:

- $E_{frp}$ Young’s modulus of elasticity
- $f_{frpu}$ Ultimate tensile strength and elongation of pultruded laminate
- $T_{GM}$ Glass transition temperature
- $V_f$ Volumetric fibre content

<table>
<thead>
<tr>
<th>Fibre</th>
<th>$E_{frp}$ MPa</th>
<th>$f_{frpu}$ MPa</th>
<th>$\varepsilon_{frpu}$ %</th>
<th>Thickness mm</th>
<th>Density $g/cm^3$</th>
<th>$T_{GM}$ °C</th>
<th>$V_f$ %</th>
<th>Composition</th>
</tr>
</thead>
<tbody>
<tr>
<td>GFRP</td>
<td>65</td>
<td>1700</td>
<td>2.88</td>
<td>0.135</td>
<td>2.6</td>
<td>-</td>
<td>100</td>
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<td>CFRP</td>
<td>240</td>
<td>3900</td>
<td>1.55</td>
<td>0.117</td>
<td>1.7</td>
<td>100-130</td>
<td>100</td>
<td>Uni-Direction</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Epoxy</th>
<th>Colour</th>
<th>Strength MPa</th>
<th>$E_{flexural}$ GPa</th>
<th>Density kg/litre</th>
<th>$T_{GM}$ °C</th>
<th>Coverage/ Thickness</th>
<th>Manufacturer U.K.</th>
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<td>Adhesive</td>
<td>Mid grey</td>
<td>19</td>
<td>9.8</td>
<td>1.535</td>
<td>60</td>
<td>2-4mm</td>
<td>SBD</td>
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<tr>
<td>Primer</td>
<td>Translucent</td>
<td>-</td>
<td>-</td>
<td>1.12</td>
<td>-</td>
<td>4-6 m² /l</td>
<td>SBD</td>
</tr>
</tbody>
</table>

The next important material required for pre-stressing is the Expansive Agent (EA). It is supplied as a powder and the colour is grey when mixed with water [7]. This material is normally used for concrete demolition and is placed in pre-drilled
holes. Figure 1 illustrates the relation between expansion pressure and reaction time for different hole diameters (when used for demolition) over a period of four days. Figure 1 also shows the EA when mixed with water (20% - 23%). In this experimental work the EA was mixed with cement in different proportions.

3. THE EFFECT OF CONFINEMENT STIFFNESS
In order to investigate the effect of confinement stiffness, the EG was confined directly by metal tubes. The tubes were selected to apply different confinement stiffness levels with different materials (steel or copper), thicknesses, diameters as shown in Figure 2 and different EA ratios. The ratio of length (L) to inner diameter (D) of the tubes was set to be around 10. It is obvious that, choosing different thicknesses of confinement materials (t), elastic modulus of the tubes (E) and radius of tubes or jacket (r), can change the confining stiffness (CS) as defined by $2E \frac{t}{ID}$.

![Figure 1. Properties of Expansive Agent (EA)](image1)

![Figure 2. Metal tubes filled with EG](image2)
Thirty six metal tubes were filled and tested with EG levels 5%, 10% and 20% of EA. The EG consisted of Betonamit (EA), 50% cement, the rest of mix ratio completed by sand (was variable) and appropriate water (between 17% to 20% of the weight of mix).

Various methods of sealing the tubes were tried during casting to prevent any leakage due to the high pressure created by the EG. Due to the possibility of welding on the black steel, one end was fixed while the other end remained open until the expansive grout (EG) was poured into the tube. The end was then sealed by fastening a screw. Due to the characteristics of the metals, welding of copper and annealed steel tubes was not a feasible solution. Two solid plugs were therefore fitted with a located pin designed. In addition, a layer of silicon seal was applied to the surfaces of the plugs to prevent any leakages. The pipes were instrumented with 3 strain gauges placed at the mid length and equally spaced along the circumference. Two gauges were used to measure lateral strain and one to measure the axial strain. Since the strain gauge had a length of 15 mm, only two strain gauges, which were fixed one laterally and the other one axially were used for the smaller copper and black steel pipes.

4. EFFECT OF VOLUME OF EG

This sub-phase investigated the effect of 3 different amounts of expansive grout (EG) with the same EA ratio (20%) on steel tube cylinders having the same confining stiffness.

All tubes were instrumented on the outside with three surface strain gauges to measure lateral strain and one strain gauge to measure axial strain. The first sample (S1) was filled with expansive material without any core in the middle. The second sample (S2) had a concrete core with a diameter of 50 mm placed in the middle. Additional strain gauges were placed at the mid height of the concrete core to monitor lateral and axial strain. The third sample (S3) was similar to S2, but the concrete core had a diameter of 66 mm. Figure 3 shows samples S1, S2, and S3. To avoid any leakage from the top and the bottom of the cylinders, two steel plates (with rubber washers) were used.

![Figure 3. The cylinders filled with different volume of EG](image)
When all samples were ready to be cast, as shown in Figure 5-33, the grout was poured in the 7 mm gap provided. The amount of the expansive grout (EG) that was used determined as a function of the amount of expansive agent (EA). Ratios (in weight) of 10%, 20%, 30% or 40% were used. Whilst filling the EG through the gap, vibrating by a smooth vibrator machine took place. The expansive grout was injected using a silicon gun with plastic pipe attachment with a diameter of five mm.

5. SPECIMEN DETAILS
Since the chemical pre-tension (expansive pressure) is caused by the EA reacting against the confining jacket this means that the magnitude of this pre-tension depends on the degree of stiffness of the jacket and percentage of EA. Experiments were conducted by using these two parameters to quantify the amount of pre-tensioning on the jacket. Following that series of testing, concrete cylinders were confined with pre-tensioned composites. To vary the stiffness and strength of the confining jacket, one, two and three layers of carbon and glass sheets were used. Different percentages of EA were mixed with cement (10% and 20%) to achieve different confining pressures. Also, different materials for jacketing (Glass and Carbon) were used to achieve different stiffness and strength.

A total of eighteen 100mm x 200mm concrete specimens were prepared without any pre-tensioning, 36 specimens were prepared with different levels of confinement prestressing and 15 unconfined specimen were tested under compression to determine the plain concrete strength. The concrete consisted of ASTM Type 1 Portland cement, river sand aggregate with a fineness modulus of 2.5 and gravel river aggregate with a maximum size of 10 mm. The water-cement ratio (w/c) was about 0.52 by mass. The average 28-days compressive strength of the concrete specimens was 31 MPa. Concrete specimens wrapped with one layer of Carbon and two layers of Glass without pre-tensioning force were designated as C0101, C0102, G0201 and G0202, respectively, whereas concrete cylinders with the same characteristics but pre-tensioned with 10% of EA and one layer of jacket were designated as C71101, C71102, G72101 and G72102, and with 20% of EA were designated as C71201, C71202, G72201 and G72202, respectively. Similar designation names were used for two and three layers either for non-pre-tensioned or pre-tensioned samples.

6. PREPARATION OF SPECIMENS
For the wrapping of concrete cylinders without pre-tensioning, after applying the epoxy primer on the concrete surface, epoxy adhesive was applied and Carbon/Glass sheets were wrapped around the concrete cylinder until one wrapping layer was completed (with one third overlap). At the same time a special roller was used to help impregnate the fibre with resin and hardener and give a smooth finish. After curing, the strain gauges were glued directly onto the body of the jacket. For the pre-tensioned specimens, a gap is needed between the concrete and FRP for the insertion of the EA. For these experiments, the FRP jacket was pre-manufactured with a diameter 14mm larger than the concrete cylinder. After
curing, the jacket was placed around the concrete and the ends were capped to seal the expansive agent inside. Strain measurements were taken during the expansion phase of the expansive agent for up to four days. The testing procedure followed was then the same as for the unconfined specimen.

7. TEST PROCEDURE AND OBSERVATIONS
The application of displacement to the specimens was controlled manually. The displacement was applied incrementally with each displacement level being held for a few seconds at each 0.1mm increment. All specimens were tested under centric (axial) loading. Failure was always explosive due to the high strain energy stored by the FRP material and it took place around the middle of the cylinder height as shown in Figure 4.

![Figure 4. Failure of the specimen confined with FRP jacket.](image)

8. EXPERIMENTAL RESULTS
The stress-strain diagrams for the specimen tested with one layer of either glass or carbon FRP are shown in Figure 5. Each graph shows the results for the unconfined cylinders as well as for the confined with or without pre-stressing. The right hand part of the graph shows the longitudinal strain whilst the left hand part shows the lateral strain. Strain measurements shown are the average values from the strain gauges and DV devices. The results are in general in good agreement between the two types of measurements, even though strain gauges measure local strains and DV devices integrate the strains over the length. The strain gauges also show the pre-strain that was developed by the EA. The longitudinal strain gauges on the pre-tensioned specimen show contradicting trend. Whilst the Carbon wrapped specimen show the strain to remain compressive, in the case of glass the strain eventually becomes tensile. This is partly because the glass wrapping contains longitudinal fibres as well, which restrain the concrete from expanding and lock some strain in the mid-height. It is also partly to do with the wrapping overlap,
which creates eccentric deformations on the specimen. Eccentric deformations and initial slip of the jacket may explain the tensile strains in the glass.

9. CONCLUSIONS
This paper has shown experimental results from concrete specimen wrapped with glass and carbon FRP. In this investigation EA was used to produce post tensioning force in FRP jackets. It has shown that post tensioning can be used to enhance the load capacity and behaviour of concrete.

REFERENCES
EFFECTS OF FRP WRAPPING ON THE BOND-SLIP BEHAVIOR OF REINFORCING BAR

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ABSTRACT
Fiber reinforced polymer (FRP) composites are being successfully used for strengthening of existing reinforced concrete (RC) structures because of their superior properties. Effects of wrapping with FRP on the strength and ductility of concrete members have been extensively investigated but information about effects of external confinement on the bond-slip behavior is very limited. In this paper the effect of external confinement with CFRP strips and internal confinement with transverse stirrup at beam splice test is evaluated and the results of two confinements are compared with one another. The main examined parameters include concrete cover, development/splice length, diameter of the reinforcing bar, concrete compressive strength and the amount of ordinary transverse reinforcement in the splice/development region. Test results indicated that external confinement with CFRP is more effective than internal confinement on bond strength and bond failure ductility.

Keywords: FRP, Strengthening, Beam splice test, Bond-slip relationship

1. INTRODUCTION
In structural concrete construction, the concrete is reinforced to compensate for its relative weakness in tension. The interfacial action between the reinforcement and the surrounding concrete involve many different mechanisms that collectively establish the phenomena of “bond”. The nature of bond can vary extensively with varying loads, various types of reinforcement, and variety of situations [1]. The main parameters that influence the bond strength between steel reinforcing bars and concrete are well documented in the technical literature. Important among these parameters include concrete cover, development/splice length, diameter of the reinforcing bar, concrete compressive strength, and, for conventionally confined concrete, the amount of ordinary transverse reinforcement in the splice/development region [2]. There are two types of bond failure between reinforcing bars and concrete, and the parameters that influence each of them are well documented in literature. If the concrete cover around the reinforcing bar is large or the concrete is well-confined by transverse reinforcement, or both, bond failure occurs by pullout as a result of the shearing-off the concrete keys between the lugs. On the other hand, if the
concrete cover is small, and the concrete is either unconfined or moderately confined, bond failure occurs by splitting of the concrete around the reinforcing bar. The most common parameters that influence the splitting bond strength include concrete strength, bar diameter, cover and spacing of reinforcement, and area of transverse reinforcement [3, 5].

The amount of confinement is an effective factor in bond strength between steel and concrete. ACI code mentioned this parameter in calculating the development length of reinforcing bar (Ktr) [4, 9].

Concrete confinement becomes particularly important in areas of seismic risk. Concrete confinement in areas of seismic risk reduces bond deterioration under cyclic loading, enhances the energy absorption and dissipation capabilities and consequently improves the chances of the structure or structural components to survive under earthquake loading [2].

In this paper the effect of external confinement with CFRP strips and internal confinement with transverse stirrup on bond strength and bond-slip behavior of reinforcing bar is evaluated. For this purpose two series of the experimental test results have been used. The experimental tests carried out by Harajli and Hamad (2002) [3] which examine external confinement with CFRP strips on beam splice test and the experimental tests carried out by Harajli, Hamad, and Rteil (2004) [2] which examine internal confinement with transverse stirrup on beam splice test are used and the results of two experimental tests are compared with each other. According to the experimental test result, bond stress-slip graphs are presented and compared together.

1.1. Bond Stress

The bond stress around the reinforcement will appear if force or stress in the steel bar or concrete varies from one point to the other. According to Figure 1, the amount of bond stress in the infinitesimal length of reinforcing bar that is displayed with μ, according to satisfying equilibrium bond stress (μ) is equal to:

$$\mu = \frac{d_b}{4} \frac{\Delta f_s}{\Delta x}$$  \hspace{1cm} (1)

Figure 1. Representation of bond stress on the reinforcement

The nature of bond can vary extensively with varying loads, various types of reinforcement, and variety of situations. Conventionally, two broad types of bond are defined: anchorage/development bond and flexural bond [6, 7].
1.2. Anchorage/Development Bond
Anchorage/development bond refers to the interaction between the reinforcement and concrete when an axial tension or compression force has to be transferred to the concrete [1, 7].

\[ \mu_{\text{ave}} = \frac{d_b f_s}{4 l_d} \]  

(2)

1.3. Flexural Bond
Flexural bond refers to a situation where a gradient in bending moment occurs and the force, in the reinforced concrete member, is subjected to change [1, 7].

\[ \mu(x) = \frac{V}{\pi d_t z} \]  

(3)

2. DIFFERENT KINDS OF TESTS TO DETERMINE THE BOND STRENGTH
The bond strength between reinforcement and concrete is usually determined by the following tests [7, 8].

2.1. Pull-Out Test
This test evaluates the bond capacity of various types of bar surfaces relative to specific embedded length. The distribution of tensile stress will be uniform around the reinforcing bar at specific sections and varies along the anchorage length of the bar and a radial distance from the surface of the bar (Figure 2). However, this test does not represent the effective bond behavior in the surface of the bars in flexural members, because stresses vary along the depth of concrete section [6, 7].
2.2. Embedded Rod Test
In these tests, the tensile force is increased gradually and the number of cracks and their spacing and width are recorded. The bond stresses vary along the bar length between cracks. The strain in the steel bar is maximum at the cracked section and decreases toward the middle section between cracks [6, 7].

![Figure 4. Bond mechanism in an embedded bar.](image)

2.3. Beam Splice Test
Tests on flexural members are also performed to study the bond effectiveness along the surface of the tension bars. In this paper the beam splice test is used for studying the effect of confinement on bond-slip response of the reinforcement in the concrete [6].

3. EXPERIMENTAL PROGRAM
3.1. Properties of Test Specimen and Test Variables
The test specimen consisted of beam specimen with spliced reinforcement at midspan. Dimensions of the specimens and test parameters are presented in Figure 1 and Table 1, respectively. Four different sizes of reinforcement were used (16, 20, 25, and 32 mm). The number of splices and the width of the specimens were calibrated to produce six different ratios of minimum concrete cover-to-bar diameter c/db varying between 0.56 and 2.0. The concrete side cover, bottom cover, and 1/2 the clear distance between the spliced bars were kept identical for each specimen. The splice lengths L_s for all specimens were selected at 5db, small enough to produce local bond conditions. A bond-free length, extending outside the ends of the specimen, was secured using polyvinyl chloride (PVC) tubes for slip measurement. The spliced bars consisted of Grade 60 steel. The clear distance between the bar ribs were 6.5, 6.0, 12.0, and 9.0 mm for the 16, 20, 25, and 32 mm bars, respectively. All specimens were confined with ordinary transverse reinforcement. The transverse reinforcement consisted mainly of two (6 or 10 mm in diameter) stirrups or ties, placed within the splice region at 1/4 the splice length from each splice end producing a spacing of transverse reinforcement in each specimen equal to L_s/2 (Figure 1). Each stirrup consisted of two legs for the specimens with two splices or one leg for the specimens with one splice. The area and spacing of transverse reinforcement were selected to produce a practical range of values of the transverse reinforcement parameter A_t /snd_b between 0.011 and 0.12 [2].
3.2. Specimen Casting and Testing
The concrete mixture was designed to achieve a target concrete compressive strength $f'_c$ of 41 MPa. Portland cement, washed sand, and crushed limestone with 10 mm maximum size aggregate were used to prepare the concrete mixture. The cement: sand: aggregate proportions by weight were 0.32:0.5:1.0 with a water-cement ratio (w/c) of approximately 0.45. Each two specimens were cast together. The concrete strengths were determined using three standard 150 x 300 mm cylinders taken from each batch. Actual concrete strengths are given in Table 1 [2].

3.3. Loading Equipment and Arrangement of Strain Gauges
The specimens were loaded with two symmetrical point loads to produce a constant moment region extending a distance equal to 1/2 the member height $h$ outside the splice zone on either end of the splice. Slip of each splice and deflection of the specimen at midspan were measured using mechanical gages [2, 3].
4. DISCUSSION OF TEST RESULTS

The bond stress \( u \) at any load level during load application is calculated using the following bond relation

\[
u = \frac{A_b f_s}{\pi d_b l_s} = \frac{f_s d_b}{4l_s}
\]

Where \( A_b \) is the area; \( l_s \) is the splice length \((5d_b)\) of the bar; and \( f_s \) is the steel stress, calculated using cracked section analysis corresponding to the level of applied load.

All specimens developed clear bottom splitting or side splitting cracks along the splice length and failed in splitting mode. Splitting cracks were preceded by flexural cracks forming simultaneously at both ends of the splice. Following the formation of the splitting cracks, the load resistance dropped suddenly and diminished gradually with increasing load. Because the load resistance and deflection of the specimens are linearly related to the bond strength and bar slip, the general shape of the load-deflection behavior of the specimens was very similar to the bond stress versus slip response [2].

<table>
<thead>
<tr>
<th>Specimen</th>
<th>( U_{\text{max}} )</th>
<th>( \sqrt{f_c} ), (mpa)</th>
<th>Transverse reinforcement</th>
<th>CFRP (Harajli and Hamad 2002)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(plain concrete)</td>
<td>(confined concrete)</td>
<td>One CFRP wrap</td>
<td>Two CFRP wraps</td>
</tr>
<tr>
<td>B1N</td>
<td>1.09</td>
<td>1.13</td>
<td>1.18</td>
<td>1.08</td>
</tr>
<tr>
<td>B1W</td>
<td>1.33</td>
<td>1.34</td>
<td>1.52</td>
<td>1.57</td>
</tr>
<tr>
<td>B2N</td>
<td>0.82</td>
<td>0.90</td>
<td>0.90</td>
<td>1.09</td>
</tr>
<tr>
<td>B2W</td>
<td>1.04</td>
<td>1.10</td>
<td>1.28</td>
<td>1.20</td>
</tr>
<tr>
<td>B3N</td>
<td>1.17</td>
<td>1.39</td>
<td>1.47</td>
<td>1.27</td>
</tr>
<tr>
<td>B3W</td>
<td>0.71</td>
<td>0.91</td>
<td>0.89</td>
<td>0.91</td>
</tr>
<tr>
<td>B4N</td>
<td>0.82</td>
<td>0.77</td>
<td>0.91</td>
<td>0.93</td>
</tr>
<tr>
<td>B4W</td>
<td>0.52</td>
<td>0.64</td>
<td>0.64</td>
<td>0.66</td>
</tr>
</tbody>
</table>

Figure 6, 7, and 8 show variation of the bond-slip response for the same specimen with an increase in the area of ordinary transverse steel, FRP reinforcement, respectively.
In the specimens confined with CFRP, flexible sheets were wrapped around the perimeter of the specimens along the full splice length. All specimens (Table 2)
were confined with either single or double CFRP wraps. The design thickness of the sheets is equal to 0.13 mm (0.005 in.). The modulus of elasticity, the tensile strength, and strain at break of fibers are equal, respectively, to 230,000 MPa, 3500 MPa, and 1.5%.

Comparing the current bond-slip results for concrete confined with transverse reinforcement with those obtained earlier using other types of confinement as shown in Figure 6 to 8; it is interesting to observe that the shape of the bond stress-slip response tends to be intrinsically similar irrespective of the type and amount of confinement used. The corresponding response can be divided into four distinct stages of behavior as shown schematically in Figure 9 and as described previously by Harajli, Hamad, and Karam (2002) [2, 3]:

1) Initial stiff bond-slip response associated with adhesion and friction, which coincides perfectly with bars that undergo pullout bond failure in well-confined concrete; 2) soft pre-splitting response associated with the formation of circumferential tensile cracks in the concrete surrounding the steel bar and their gradual propagation toward the surface until the peak bond strength at which splitting occurs is reached; 3) sudden drop in the bond resistance down to a post-splitting bond strength as a result of a state of dynamic equilibrium between the radial component of the bond force and the post-splitting tensile resistance of the concrete matrix (confined or unconfined) surrounding the bar; and 4) gradually deteriorating bond resistance with increasing slip beyond splitting associated with the progressive widening of the splitting cracks until the bond resistance diminishes completely. It is clear from the comparisons made in Figure 2 to 4 that one of the most evident and important contributions of confinement reinforcement
to the local bond behavior is the significant reduction in bond deterioration following bond failure relative to plain unconfined concrete.

5. CONCLUSIONS
This study concentrated on the experimental investigation of confinement on local bond-slip behavior of reinforcing bar embedded in plain concrete at beam splice tests. The influence of two main parameters on bond slip response was evaluated namely, external confinement with FRP sheet and internal confinement with transverse stirrup.

From the test results, the following conclusions can be drawn.
1. The value of the ultimate bond stress is not influenced by the bond length, but increases as concrete compressive strength increases.
2. The bond stress-slip curves are clearly nonlinear, and have a tendency to become parabolic in form.
3. External and internal confinement causes ductility in the bond-slip relationship.
4. The bond strength due to confinement increases in proportion to the modulus of elasticity of FRP.
5. For small development/splice lengths corresponding to local bond conditions, confining the concrete with ordinary transverse steel increases the bond strength only slightly, but leads to considerable improvement in the ductility of bond behavior in the post-splitting stage.
6. For confined concrete, the shape of the local bond stress-slip response consists of four distinct stages of behavior. The corresponding behavior tends to be identical irrespective of the type of confinement used.
7. Because external confinement with FRP is more effective in restricting the width of the splitting cracks than internal confinement with ordinary steel, for the same area of confinement reinforcement per unit length along the splice and taking into account the relative modulus of elasticity of the material, the increase in bond resistance acquired for concrete confined externally with FRP sheets is considerably higher as compared with concrete confined internally with ordinary steel.

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EFFECT OF TIEROD POSITION IN SEISMIC REHABILITATION OF ANCHORED CONCRETE QUAYWALLS

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ABSTRACT
One of the most common marine structures is anchored concrete quaywall. Obviously, anchoring the wall will reduce the penetration depth, maximum bending moment and lateral displacement of wall in both static and dynamic conditions.

In this research, first, effect of tierod position in anchored concrete quaywalls in primary static condition is considered and compared with two methods: “Free Earth Method” and “Finite Difference Method”. Also, practical curves of "maximum moment" and "tierod axial force" are given versus r (ratio of tierod depth to wall length) in static condition.

Then, dynamic response of different cases (various tierod depths) is illustrated and compared. Practical history curves of “maximum positive and negative moment”, "tierod axial force” and “lateral displacements” are provided via “Finite Difference Method”. Analyzing these charts, influence of tierod position on anchored quay walls in seismic condition is investigated about seismic behavior and rehabilitation of the structure. Suitable tierod position results in better serviceability and lower maximum moment in concrete quay wall profile and therefore economical one.

Keywords: anchored quaywall, seismic evaluation and rehabilitation, tierod

1. INTRODUCTION
Flexible walls include "Anchored Sheet pile walls" or" Anchored Bored/ Continuous/ secant pile walls " are used as quaywalls. Weight of such walls is ignored comparing with lateral forces. When their height from the dredge line is more than about 6 meters, it is common to use an anchorage system [3, 5, and 7].

Using anchorage decreases penetration depth, maximum bending moment and lateral displacement in both static and dynamic conditions. In classical methods which the equilibrium equations are used, the deformation is ignored; so, it becomes necessary to utilize the numerical methods. This will help the designer to evaluate the serviceability of the structure in case of an earthquake. To rehabilitate the structure in seismic condition, the effect of tierod position in anchored concrete quaywalls is evaluated. It can improve the serviceability and also results in more economical construction using concrete wall profiles with more suitable sections [4, 8].
2. STATIC ANALYSIS AND DESIGN

2.1. Free Earth Method

"Free Earth Method" is a simple classic method which is primarily used here to analyze and design the anchored quaywall; assumptions and parameters which are used in this method are shown in Figure 1 [3, 5, 6, 7]. But in such methods some facts are ignored. For example:

- Wall moving tendency toward soil.
- Wall and anchor flexibility.
- Real lateral soil pressure distribution.

So, for more accurate analysis, a numerical method is used via FLAC 2D software as will be described later.

![Figure 1. Free Earth Method [7]](image)

2.2. Matlab Programming

Using Matlab software, a program is developed in order to analyze anchored concrete quaywalls utilizing "Free Earth Method" in sand. "Maximum moment" and "tierod axial force" curves are displayed versus tierod depth ratio to concrete wall length.

Soil properties and model parts properties are showed in tables 1 and 2 respectively.
### Table 1: Soil properties

<table>
<thead>
<tr>
<th></th>
<th>Y</th>
<th>Y_{sat}</th>
<th>φ</th>
<th>C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand</td>
<td>21</td>
<td>25</td>
<td>38</td>
<td>0</td>
</tr>
</tbody>
</table>

### Table 2. Model parts properties

<table>
<thead>
<tr>
<th></th>
<th>E (N/m^2)</th>
<th>A (m^2/m)</th>
<th>P (m/m)</th>
<th>I (m^4/m)</th>
<th>L1 (m)</th>
<th>L2 (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Quaywall</td>
<td>3.20E+10</td>
<td>1.9</td>
<td>4.3</td>
<td>3.67</td>
<td>4</td>
<td>18</td>
</tr>
<tr>
<td>wall</td>
<td>3.20E+10</td>
<td>1</td>
<td>2</td>
<td>0.083</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cable</td>
<td>2.00E+11</td>
<td>0.00277</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

![Concrete quaywall profile](image)

As it is determined in Figure 3 and Table 3, with deepening tierod position its axial force increases and vice versa, maximum moment in concrete quaywall profile decreases. (*r* is the ratio of tierod depth to quaywall length).

![Graphs](image)
Table 3: Matlab program results for three tierod positions (Free Earth Method)

<table>
<thead>
<tr>
<th>l1 (m)</th>
<th>R (l1/L)</th>
<th>M (KN*m)</th>
<th>F (KN)</th>
<th>D (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.09</td>
<td>0.15</td>
<td>3167</td>
<td>518.7</td>
<td>5.59</td>
</tr>
<tr>
<td>8.01</td>
<td>0.29</td>
<td>2024</td>
<td>623.7</td>
<td>5.15</td>
</tr>
<tr>
<td>12</td>
<td>0.45</td>
<td>-1317</td>
<td>791.7</td>
<td>4.31</td>
</tr>
</tbody>
</table>

3. DYNAMIC ANALYSIS WITH “FINITE DIFFERENCE METHOD”

Evaluating the serviceability of the structure in case of an earthquake needs modeling with a strong, accurate and fast numerical method. So, “Finite Difference Method” is used in this research. At first the structure is modeled in static condition by means of “FLAC 2D” software and then the dynamic analysis is done [1, 2].

3.1. “Flac 2d” Software Introduction

This software is a computational one based on explicit “Finite Difference Method” and is equipped with tools which can consider various geotechnical aspects. In this research the effect of tierod position in anchored concrete quaywall is mentioned for economical Seismic rehabilitation; therefore, complicated soil behavior is considered by means of Mohr-Coulomb plastic soil model [1, 2].

3.2. Problem Solution Stages with “Flac 2d” (FDM)

First, general problem environment is created via Finite Difference elements, then Mohr-Coulomb behavior model is selected and material properties are assigned. Before solving problem it is necessary to determine boundary and initial conditions.

Modeling and loading procedure:
- Modeling soil considering initial stress in soil mass.
- Installing main and anchorage walls in soil.
- Positioning tierod between two walls.
- Excavating front of main wall to final dredging level.
- Applying water pressure.
- Solving problem in static condition.

After evaluating wall behavior in static condition, acceleration is applied to the lower model boundary with shape of a sinusoidal wave with maximum amplitude of 0.25g in two seconds duration. The general geometry of model is shown in Figure 4.

Dynamic loading procedure:
- Applying seismic load with \( PGA \) of 0.25g.
- Dynamic analysis of the problem.
4. ANALYSIS RESULTS
In order to find about consistency in primary static analysis results of two methods, Figure 5 is prepared. It is shown that both methods result in similar tierod axial force and maximum quaywall moment based on various tierod positions ("r" is ratio of tierod depth to quaywall length).

Results of dynamic analysis are illustrated in Figures 6 to 9. Lateral displacements are shown in Figure 6. Charts show that lateral displacements in bottom and middle of the quaywall based on different tierod depths are almost similar to each other during time history.
But as the main criteria of serviceability, different displacements of quaywall top that are shown above, have more changes with deepening tierod position, during assigned earthquake. Note that all lateral quaywall top displacements are limited between -0.08m to 0.09m. So, they are not more than allowable displacement of 0.30m due to "OCDI".
As it has been determined in Figure 7, Tierod axial force increases with deepening tierod position in all history duration. Similarly, maximum positive moment in quaywall (Figure 8) increases with deepening tierod position in all history duration. Absolute amount of maximum negative moment in quaywall (Figure 9) in all history duration decreases with deepening tierod position.
5. CONCLUSIONS

Considering the results which were discussed above, in earthquake duration the absolute amount of both positive and negative maximum moments in anchored concrete quaywall is the least when tierod is in depth of about 8 meters. The tierod axial force in depth of 8 meters is between axial forces in two other depths in all history duration.

It is inferred that an appropriate serviceability is obtained due to various tierod positions and all cases satisfy the horizontal quaywall top displacement limit (based on “OCDI”). Hence, to rehabilitate the structure in seismic condition economically, it is logical to select a tierod position which gives lower maximum moment and axial force (so, economical wall profile and anchorage system) in earthquake duration. It is possible to select a suitable tierod position through curves and analyses of this research.

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ACCURACY OF COMMON MACRO-ELEMENT MODELS IN PREDICTING BEHAVIOR OF CONCRETE INFILLS

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ABSTRACT
Reinforced concrete infills improve seismic behavior by increasing lateral strength, initial lateral stiffness, and energy dissipation capacity of buildings, so it is important to implement a model which can predict behavior of infilled buildings correctly. Due to convenience and simplicity in application proposes, modeling of infills with macro element models can be implemented in place of micro element. In this study, two applicable macro-element models namely one-strut and three-strut was implemented for modeling of these infills and accuracy of these models in predicting actual behavior of structure was compared with experimental tests which have been carried out in recent years on concrete and steel frames. The results show that in frames with strong members when the critical mode is failure in infill; three-strut can simulate ultimate strength and initial stiffness better than one-strut model. This paper also indicates that frame weakness can affect dramatically on the concrete infilled frame behavior and interrupt infill performance.

Keywords: concrete infill, macro element model, three-strut model

1. INTRODUCTION
Infilling frame with reinforced concrete wall is one of the strengthening techniques for reinforced medium rise buildings. Reinforced concrete infills improve seismic behavior by increasing lateral strength, initial lateral stiffness, and energy dissipation capacity of reinforced concrete buildings, and limit both structural and nonstructural damages caused by earthquake. Figure 1 shows a schematic view of these infills.

Figure 1. Schematic view of Concrete Infill
The various theoretical models reported in the literature for predicting the seismic behavior of infills can be classified into two categories: (i) micro element-based and (ii) macro element-based models. Theoretical microelement models, such as the finite element model, provide a rigorous analytical approach to evaluate the dynamic response of infills. A number of finite element models have been developed and used to predict the in-plane lateral load behavior of these infills [5]. Macro element modeling offers an alternative approach in which the entire infill panel is represented as a single strut or multi-strut approach [1, 2]. In this way, only the equivalent global behavior of the infill panel is taken into account in an analysis. Thus, for analysis focusing on overall structure response, macro element models can be implemented in place of micro element models. Application of microelement not only has some complexity in modeling, it is also time consuming. Furthermore, in this study two applicable methods, namely one-strut and three-strut were implemented for modeling of concrete infills and accuracy of the results was compared with the experimental test which had been carried out previously on concrete infills.

2. DESCRIPTION OF MACRO-ELEMENT MODELS

2.1. One-Strut Model

Firstly, one-strut model based on FEMA 356 was used for modeling of concrete infill. It is very important to identify the modes of failure or other effects which need to be controlled or avoided. Based on experimental tests only two modes, the corner crushing (CC) and sliding shear (SS), are of practical importance, (Comite 1996). In order to determine the governing failure mode, the capacity of the infill panels in first and second failure mode were estimated. Because of high value of shear strength in RC infills, in most cases corner crushing mode is dominant. The FEMA 356 provisions prescribe a strut with an area equal to the thickness of the masonry infill panel times; the strut width is given by Eqn. 1.

\[
a = 0.175(\lambda h_{col})^{-0.4} r_{inf}
\]

\[
\lambda_i = \left[ \frac{E_{me} t_{inf} \sin 2\theta}{4E_{me} I_{col} h_{inf}} \right]^{\frac{1}{4}}
\]

\( h_{col} \) and \( r_{inf} \) are the height and diagonal length of infill panel respectively, \( E_{me} \) is expected modulus of elasticity of infill materials, \( t_{inf} \) and \( h_{inf} \) are thickness and height of infill panel, \( I_{col} \) is the moment of inertia of column and \( E_{fe} \) is expected modulus of elasticity of frame materials. It is justifiable to assume that the panel properties in the diagonal direction are the properties governing the behavior of the infill panel. Concrete material is modeled using total strain rotating crack model (DIANA 2005) that describes the tensile and compressive behavior using one stress-strain relationship. The concrete in compression is defined using a parabolic stress-strain \((\sigma - \varepsilon)\) relationship as shown in Figure 2 and defined by equations 3 through 6.
Where:
\( f_c \) = the maximum compressive strength based on uniaxial concrete compression test result, \( E_c \) = the initial modulus of elasticity of concrete in compression estimated in unite of kg/cm² as \( E = 45800 \sqrt{f_c} \), \( \varepsilon_c \) = the strain at which \( 1/3 \) of the compressive strength is reached, \( \varepsilon_u \) = the strain at which the maximum compressive strength is reached, \( \varepsilon_u \) = the ultimate strain in compression at which the material has no strength. \( G_c \) = the fracture energy in compression determined to be consistent with the assumed value of \( \varepsilon_u \) per table 1. The tensile behavior of concrete is modeled using elastic with linear softening relationship as shown in Figure 2 where \( f_{ct} \) is the tensile strength of concrete as determined in concrete split tension test. The value
of \( G_f \) is estimated in units of N/m as \( G_f = \alpha f_{ck} \) where \( \alpha = 6.75 \) and \( f_{ck} \) is the characteristic strength in unit of MPa taken as the same as \( f_{ct} \) in this study.

### 2.2. Three-Strut Model

Because concrete infills are strong members, interaction between infill, beam and column are important and need a model to represent characteristics of concrete-infilled frame correctly. Usage of a multi-strut model rather than single strut will better represent the actual stressed area within the infill and also facilitate the modeling of the progressive failure occurring at the corner contact region, not just at the corner points. Use of three-struts for modeling of infills was studied by El-Dakhakhni [2]. Based on research, it is suggested that at least two additional off-diagonal struts located at the points of maximum field moments in the beams and the columns are required to reproduce theses moments as shown in Figure 3.

![Figure 3. Schematic view of three-strut model [2]](image)

It is suggested that the total diagonal struts area, \( A \), is to be calculated by

\[
A = \frac{(1 - \alpha_x) \alpha_x h t}{\cos \theta}
\]  

(7)

Concrete material is modeled using total strain rotating crack model as described by equation 3 through 6.

### 3. MODELLING OF TEST SPECIMENS USING MACRO MODELS

To evaluate accuracy of macro-element models to determine behavior of structures, some experimental study which has been previously conducted including two CICF (concrete-infilled concrete frame) and one CISF (concrete-infilled steel frame) specimens was implemented. Each model has special characteristics which will be discussed shortly. Six CICF specimens were tested at the University of Gazi in Turkey under reversed-cyclic lateral loading by Sinan al"in et al. [3]. The specimens are one-bay two story concrete-infilled concrete frames. In this study,
only the first and second specimens were considered. The first specimen shows poorly lap spliced columns of nonductile RC frame, while providing RC infill walls. The second specimen is similar to the first specimen but longitudinal reinforcements pass continuously along two stories in boundary elements of infills. These two specimens were considered as representatives of common concrete frames. Figure 4 shows these test specimens in testing.

One CISF specimen was tested in the Building and Housing Research Center in Iran by Moghadam and Mohammadi. This specimen was one-bay one story concrete-infilled steel frame. The details of the test can be found in [5]. The load displacement behavior of test specimens was evaluated by using nonlinear push over analysis. Push over analysis simulated the nonlinear lateral load displacement relationship of the test specimen analytically. Analytical model for a specimen is given in Figure 5 as an example.
Columns and beams were modeled based on FEMA 356 provisions. Because there is no special element in the software for modeling lap splices it was modeled as concentrate hinge based on FEMA 356 provisions. Numerical values of the parameters for the concrete compression material model are presented in table 1.

**Table 1: Parameters for concrete compression material model**

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$f_c$ (MPa)</th>
<th>$\varepsilon_c/3$</th>
<th>$\varepsilon_c$</th>
<th>$\varepsilon_u$</th>
<th>$G_c$ (KN/mm)</th>
<th>$h_c$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CICF</td>
<td>298</td>
<td>.000364</td>
<td>.001456</td>
<td>.015</td>
<td>.138</td>
<td>305</td>
</tr>
<tr>
<td>CISF</td>
<td>150</td>
<td>.0003</td>
<td>.0012</td>
<td>.015</td>
<td>.138</td>
<td>305</td>
</tr>
</tbody>
</table>

4. **COMPARASION BETWEEN EXPERIMENTAL AND ANALYTICAL RESULTS**

In the following, load displacement relationship of each specimen by means of one-strut and three-strut models was obtained and compared with experimental tests. Figure 6 shows load deflection relation of first CICF specimen. It was observed using one-strut and three-strut models; both adequately simulate initial stiffness of concrete-infilled frame. It can be seen that strut model can not predict behavior of infill in ultimate load, as well as in the descending segment of backbone curve. The reason for this is in the following. Because of deficiency in lap slice region in the column, this point acts as a fuse and failure occurred in this region. It means the ultimate load is equal to column tensional-force and does not depend on infills strength, so implementing one-strut or three-strut models caused nearly similar results.

![Figure 6. Load-deflection relations for first CICF specimen](image)

Figure 7. Illustrates load-deflection relation of second CICF specimen, it shows that the three-strut model curve has a better coloration with the experimental test. In this case because of continuous longitudinal reinforcement, the column has
higher strength than concrete infills so failure in infills is prior to failure in columns. It means concrete infills act as a fuse and behavior of this element widely affects CICF specimen behavior. The three-strut analytical model adequately simulates the behavior of infilled test specimen until the ultimate load was attained. The displacement corresponding to ultimate load which was predicted by one-strut and three-strut models is the same with experimental tests. But using one-strut model gets a much higher ultimate load than the experimental test and the three-strut model.

![Figure 7. Load-deflection relations for second CICF specimen](image)

From Figures 6 and 7 it is concluded that concrete infills are strong members and can attract large amount of forces in earthquakes. But their performance depends strongly on perimeter beams and columns. For example, premature failure in poor concrete-frame caused by lap splice can have a dramatic effect on infilled-frame behavior and reduced ultimate load of about 100 percent.

Figure 8. Depicted nonlinear behavior of CISF specimen tested in Iranian Building and Housing Research Center by Moghaddam and Ghazimahale. Due to higher tensional strength of steel columns as compared with concrete ones, corner crushing failure occurred in infill. This fact has been reported based on experimental test which was conducted on steel frame [5]. Furthermore, behavior of infill has a main effect on behavior of CISF. It was observed that one-strut model gives a higher strength than three-strut and use of this model in modeling of this element may be non-conservative. Stiffness of three-strut model has a nearly good correlation with experimental test before ultimate load. One-strut model neither attains a much more ultimate force than the experimental test nor does it give an appropriate stiffness before and after the ultimate load.
There were differences between the part of analytical and experimental load-displacement curves at which after the ultimate load was reached. One of the main reasons for the difference between the analytical and experimental initial stiffness is the difference in the method of load application. While cyclic loading was applied during experiments, analytically the load was increased monotonously up to failure. This difference in application of loading affected the initial stiffness of analytical and experimental results.

CONCLUSION

- Analytical studies were performed to understand the effect of one-strut and three-strut proposed models on the behavior of concrete-infill in steel and concrete-frames. This paper shows that in frames with strong members when the critical mode is failure in infill; three-strut can simulate ultimate strength and initial stiffness better than one-strut model.
- Three-strut model can appropriately estimate initial stiffness of infill frame and no matter failure mechanism occurs in frame or infill.
- Displacements corresponding to ultimate load which are predicted by one-strut and three-strut models are the same with experimental tests.
- This study shows that one-strut model based on FEMA 356 can estimate stiffness and strength of concrete infill superior than reality.
- Infills impose shear force to adjacent elements. Therefore, using a model to consider this fact is mandatory. Three-strut model can predict possibility of shear failure in beam and columns adjacent to concrete infills.
- Premature failure in poor concrete-frame caused by lap splice, can affect dramatically the infilled-frame behavior.
- Concrete infills are strong members and can attract large amount of forces in earthquakes. Incorrect modeling can disturb hinge propagation in structural elements and cause unreasonable results.
REFERENCES
DETERIORATION EVALUATION OF COOLING TOWER UNIT 28 IN MOBAREKEH STEEL COMPLEX AND PROVIDING ITS REPAIR PROCEDURES

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ABSTRACT
It is generally supposed that concrete is one of the best durable materials but, it is a fact that all concrete structures will deteriorate with time, though the rate at which they deteriorate varies considerably, as it is affected by many factors. Deterioration will change the performance and appearance of structure, which may affect its performance under normal working conditions.
Cooling tower unit 28 is one of the concrete structures in the Mobarekeh Steel Complex which was observed with various remarkable deteriorations. This concrete structure was directly subjected to circulation of industrial water and under aggressive sulfates, corrosion reinforcement, lack of timely attention and periodical investigations at regular time intervals were found responsible for serious deteriorations. 
In order to have an optimal use of the structure and its stability and to achieve an adequate repair procedure, a thorough and logical investigation of distress causes was carried out. The survey includes information collection and visual sketching of distress locations, several in-situ NDT tests, the determination of various aggressive ions in depth of concrete, and some other laboratory tests on core specimens taken from selected components of the structure. Based on studies carried out, different deterioration mechanisms were determined and then concrete removal methods and appropriate repair procedures were suggested. Finally, durability and the remaining service life of repaired structure is predicted.

Keywords: distress evaluation, concrete, inspection, durability, maintenance strategies

1. INTRODUCTION
Worldwide concrete with Portland cement is the most widely used construction material in buildings and civil engineering structures. There are several reasons such as appropriate resistance to water, easily flexibility on various shapes and dimensions, inexpensive and easy availability of its materials constituents. Over the years, the type and quality of concrete materials and construction methods have varied considerably. In parallel, there has been an increased understanding of the mechanisms underlying the behavior of concrete and its performance in service. It is an unfortunate, but inescapable fact that all concrete structures will deteriorate
with time, though the rate at which they deteriorate varies considerably, as it is affected by many factors. Deterioration changes the performance and the appearance of the structure, and as a final consequence it may affect its safety and behavior under normal working conditions on exploitation. Usually in the past, some maintenance actions were taken into consideration when already visible traces of deterioration were observed. Where periodical inspections were not carried out at regular time intervals, serious damages might be already presented at the first inspection [4, 5, 6].

Cooling tower unit 28 is one of the concrete structures in the Mobarekeh Steel Complex which was seriously damaged. This structure is used for cooling and decreasing water temperature circulated in the steel production lines. The concrete was subjected to circulation of industrial water and under aggressive sulfates, corrosion of reinforcement, without periodical inspections at regular time intervals and thus serious deteriorations were presented. Damages owing to corrosion of reinforcement have caused expansion and eventually resulted in cracking, delaminating and spalling of concrete.

In order to have an optimal use of the structure and its stability and achieving an adequate repair procedure, a thorough and logical investigation of distress causes was carried out. The survey includes information collection and visual surveying and sketching of distress locations, several in-situ NDT tests, determination of various aggressive ions in the depth of concrete, and some other laboratory tests on core specimens taken from selected components of the structure. Considerations and visual investigation and inspections of the structure, surface impairments, NDT tests and collecting core samples for laboratory tests are shown in Figure 1. Based on the results of preliminary inspections and in-situ and laboratory tests, various deterioration mechanisms were determined and then concrete removal methods and adequate repair procedures were suggested. At the final stage of this study, durability and the remaining service life of the repaired structure is predicted [1].

2. OBSERVATIONS, VISUAL SURVEYING AND INSPECTIONS

In order to investigate the performance of the cooling tower structure which is in a propagation phase of deterioration, an assessment of the current condition of the structure is necessary. This investigation is also necessary because of several other factors such as current maintenance, rehabilitation, serviceability conditions changes, investigation of structural stability and its function, and a study of current environmental conditions. Current state could include rapid assessment and visual inspection up to complex considerations which are taken into account in long time planning and performance. The main purpose of visual surveying and investigation is the diagnosing of probable causes of any visual deterioration and ensuring that the structure remains in its integration and satisfactory conditions. An instance of visual surveying of distress locations is shown in Figure 2 [1].

The most important deteriorations which were obviously seen in the exposed concrete in cooling tower unit 28 are longitudinal cracking due to corrosion of reinforcement in beams and columns and also severe sulfate and frost attacks (see Figure 3). Nevertheless, the essential distresses which are manifested in this
structure is classified as follows:

a) Longitudinal cracking in direction of steel bars in beams and columns (Figure 3).
b) Severe corrosion of reinforcement (Figure 4).
c) Severe removing of cement paste and exposed aggregates (Figure 5).
d) Sulfate and frost attack.
e) Delamination of concrete walls.
f) Leakage and efflorescence (Figure 6).
g) Low concrete cover over reinforcement.
h) Spalling of concrete cover due to corrosion.
i) Disintegration and scaling.

3. IN-SITU TESTS

In the first stage of the inspection and evaluation of the structure and according to visual surveying and inspections, various non destructive tests (NDT) comprising the determination of concrete cover, pulse velocity measurement, concrete strength by Schmidt hammer, resistivity measurement, determination of corrosion rate and depth of carbonation (Figure 7) were carried out. In continuing of the completion of quality and quantity of studies in laboratory, 17 core specimens were taken from selected...
beams, columns and concrete walls at different levels of the cooling tower [1].

Figure 2. Schematic and sketching of distress locations in the western view of cooling tower unit 28

Figure 3. Longitudinal cracking and sulfate attack

Figure 4. Severe corrosion of reinforcement
Figure 5. Removing of cement paste and exposed aggregates

Figure 6. Leakage and efflorescence

Figure 7. Determination of carbonation depth
4. LABORATORY TESTS

In the laboratory, compressive strength, water absorption, chloride and sulfate ion profiles and concrete pH in different depths of core samples are implemented. During taking of core specimens it was seen that the cover of concrete in all samples taken from concrete walls and some of those taken from beams is separated due to the corrosion of reinforcement.

Water could be considered as one of the most important reasons which cause impairments in concrete and concrete structures. Water is the initial compound of life and decomposes the most natural materials and also causes most of the difficulties for concrete durability. Water also is one of the reasons of decrease in quality in porosity materials and is responsible for the intrusion of aggressive ions into concrete and is one of the resources for chemical processes causing quality reduction.

One sample from circulated water in the cooling tower unit 28 was taken for chemical analysis. Chloride and sulfate ions contents in this sample were less than the allowable limit which is in the Iranian code of practice. Nevertheless, these ion contents in a constant volume of concrete are more than the allowable limit; therefore, the concrete of cooling tower’s structure was subjected to aggressive ions. Its intensity depends on concrete quality, concrete cover, and permeability of concrete and on how the structure is maintained during its service life [1].

5. CONCLUSION OF EVALUATION STUDIES AND TECHNICAL RECOMMENDATIONS

Concrete is one of the most widely used construction materials alike other materials with a life time. Concrete deteriorations, due to several causes could considerably affect the technical life time of concrete structures. Therefore, the awareness of deteriorations and their mechanisms, their prevention and/or decreasing the intensity of damages, creating delay in their progress, and considering appropriate requirements afterwards are one of the most important duties of civil engineers who deal with concrete works. On the other hand, the concept of innovative construction materials and also innovative concretes is not only considered as its own materials. But the concept of life time and durability design is also one of the essential parts which must be taken into account. Life time and durability design concepts, along with its deep and wide considerations are surrounded by all material parameters, environmental conditions, construction conditions, human resources and technological conditions [8, 9, 10].

Nevertheless, all concrete structures are always subjected to deteriorations which is affected by many factors. In addition, considering the concepts of life time and durability design for such structures, maintenance planning during physical service life of the structure is one of the most important factors increasing the life time of concrete structures.

In the evaluation of concrete durability of cooling tower unit 28, attention was given to the knowledge of physical-chemical processes of concrete distresses causes in real structure which have been observed and for diagnosing of these causes, several in-site and laboratory tests were carried out. However, interactions
of physical and chemical causes of distresses, which are sometimes complicated, are considered. Regarding the tests results and visual inspection, the conclusion is as follows [1]:

a) Based on considerations and according to concrete Iranian code of practice, the structure of cooling tower unit 28 was subjected to moisture, wet and dry conditions, freezing and thawing, periodic cooling and warming, hence locating in very severe environmental condition.

b) The average compressive strength of concrete specimens shows very high compression strength (average: 44 Mpa on cylindrical specimens) and it is also very dense with low permeability (with 2.25% maximum water absorption).

c) In spite of high compressive strength and low permeability, the structure was always subjected to aggressive harmful ions which was available in water, so that stresses due to wet and dry conditions and periodic cooling-warming caused cracking and ingress of aggressive ions into concrete was intensified.

d) High leakage and penetration of water from inside to the outside of concrete walls of the structure caused an intensive corrosion of reinforcement and sulfatation of concrete. Low cover of concrete in some locations in concrete walls also caused corrosion and concrete delaminations.

e) As some parts of beams and columns in the structures were under 70-90% moisture and other parts were simultaneously subjected to environment and wet-dry condition, corrosion in these elements was developed as cavities. From the point of view of corrosion, increase of concrete water saturation is a useful effect in decreasing the oxygen penetration and on the other hand, it could be harmful because concrete electrical conductivity is considerably enhanced by increasing the degree of saturation. Therefore, it is not surprising that maximum corrosion and cracking in beams and columns occurred in places where there were wet and dry conditions. Getting sulfate of concrete because of aggressive sulfate ion of water and disintegration and exposure of aggregate due to freezing and thawing conditions intensified the concrete surface distresses. Sulfate ion profile in depth of a concrete core is shown in Figure 8.

f) Besides the insensitive factors of deterioration in the structure, one of the most important causes of distress development was the lack of maintenance planning during service life of the structure, lack of periodical inspection and timely prevention of distress development.

g) With chloride ion contents in the depth of concrete cores (Figure 9), corrosion of reinforcement, delamination of concrete cover, sulfatation and frost of concrete surfaces, it could be concluded that the concrete must be removed up to the minimum depth of 100 mm and replaced with a higher strength and low permeable concrete.

h) Also due to the penetration of water from the interior to the exterior, the interior surface of concrete walls must be cleaned and coated with impermeable materials. In addition, the existing joints which have caused leakage of water to the outside must be filled with appropriate resin.
6. MIX DESIGN OF REPAIR MATERIALS
Each repair work has its exclusive conditions and its own requirements in identifying the necessary criteria for repair since in many cases more than one appropriate material for use is available. Concrete repair materials can be formulated to provide a wide variety of properties. Final selection of material or combination of various materials has been implemented by consideration of several factors such as ease of application, cost, skill availability and necessary equipments for their usage. Information about the service life of materials, which were used in previous repair works, plays an essential role in the selection, usage and maintenance of such materials [2, 3].
In selection of repair materials, emphasis is on those which might have higher performance and durability. Therefore, selection of these materials must be on the basis of awareness of their physical and chemical properties, the purpose of their usage and the natural condition of environment in places where they are used.
On the basis of usual inspections, in-site and laboratory tests results in previous sections and knowing the distresses causes such as corrosion of reinforcement, sulfate and frost attacks, with the intention of providing repair mix design and its planning, the following items are carried out [1]:
   b) Mix design alternatives on the basis of various tests on concrete material constituents.
   c) Repair procedures alternatives based on various concrete deteriorations.
   d) Prediction of durability and service life of repaired structure.

7. REPAIR PROCEDURES
Concrete is one of the multiple applied construction materials which is used with reasonable cost, having appropriate strength and durability and flexibility on
shapes and dimensions. In concrete structure of cooling tower unit 28, some problems such as lack of timely maintenance, environmental conditions, aggressive chemical components on passage of time, were the causes of concrete deteriorations and serious distresses. Therefore, repair of damaged places by replacing with appropriate concrete under various procedures which depend on distress depth, its severity and extent, is necessary [2, 3, 7].

In the repair works of this structure, improvement and provision of concrete appearance acceptance in terms of their color and texture between repair zones and other parts are considered. In addition, repair zones must be permanently bonded with main concrete and also have enough low permeability, without shrinkage and crazing cracking and enough resistance against freezing and thawing. Therefore, repair of this structure needs more attention and necessary design and plan as compared to other buildings in the Mobarekeh Steel Complex. With consideration of impairments of the structure and its severity and extent, the following repair procedures alternatives are recommended [1]:

a) Formwork and pouring of concrete in beams and columns.
b) Patch repair of minor deteriorations.
c) Shout Crete for concrete walls.
d) Repair of cracks by injection.
e) In the result of water penetration from inside to outside, the interior surface of concrete walls must be cleaned and coated with impermeable materials.

8. CONCLUSION

It is generally accepted that in the design of concrete structures, durability properties of materials must be considered along with their other characteristics, such as mechanical properties and costs. In the evaluation of concrete durability and the durability of concrete structures, attention must be paid to the point in which most information about physical-chemical processes causing concrete deteriorations is obtained from actual structure history because the simulation of long term status in laboratory is very difficult. Although in reality, the concrete distress is rarely found due to a unique cause. Usually in advanced stages of material degradation, more than one harmful phenomenon is observed. In general, physical and chemical causes of distresses are so complicated and so intensified together that often the separation of cause and effect is not even feasible. Permeability is one of the most important concrete parameters affecting durability. Most aggressive materials which are generally soluble in water penetrate through concrete capillary pores. Also, concrete with low porosity is dense and has better quality in terms of durability and strength. Water absorption is due to low water pressure in concrete. Hydration and concrete drying could decrease water pressure in concrete and increase absorption. In some parts of cooling tower unit 28, periodic water absorption and drying caused deposition of water harmful ions in concrete thereby resulting in increasing of sulfate and chloride ion contents in concrete. After absorption of chloride in water by concrete, it contaminates in depth and chloride gradually penetrates in concrete. Ingress of chloride ion continues in concrete where it reaches reinforcing bars and results in the progress
of corrosion. Therefore, it is important that the concrete surface condition should be improved by replacement with high strength concrete up to 100 mm in depth and also by applying impermeable coating materials so that the service life of cooling tower could be considerably enhanced (minimum 30 years). In addition of the concept of structural life time and durability design consideration, maintenance planning, periodic inspection activities and timely prevention of the progress of deterioration during service life of the structure are necessary precaution actions.

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INVESTIGATION OF A MANAGEMENT FRAMEWORK FOR PERIODIC ASSESSMENT OF CONCRETE STRUCTURES BASED ON INSPECTION AND VISUALIZATION

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ABSTRACT
In order to assess old concrete structures and to evaluate possible distresses, determination of their current conditions is necessary. The aim of this assessment is to gather information on important distress, their causes and the severity and extent. This information is only obtained from a thorough and logical inspection, regular reports during service life of structures, and also design and construction details. Some of the concrete structures in industrial situations are subjected to various distresses due to different activities and lack of timely attention. These impairments cause incompatibility changes on mechanical, physical and chemical concrete properties and usually along with concrete disintegration. Nevertheless, it is realized that, in durability point of view, most concrete structures become deteriorated during their service life, where their severity and extent are affected by various factors. Unfortunately, lack of timely attention and determination of concrete distress causes, especially in places where periodic inspections have not been implemented at regular intervals, often cause concrete structures to become degraded and the estimation of these damages have shown a high cost of repair activities. Therefore, in order to achieve an adequate evaluation and actual cause of impairment and to provide the inspection, maintenance and repair strategies along with acceptable criteria in concrete structures, the present research work was carried out in four stages. It comprises evaluation methods, inspection management, criteria of repair materials and procedures selection, and finally provides a management system for inspection and visualization according to distress mechanisms. For the application of this research, the system results are used by a case study taken from the evaluation of concrete structures in the Mobarekeh Steel Complex.

Keywords: concrete distress, inspection, maintenance strategies

1. INTRODUCTION
Identification of distress cause is the most difficulty and important stage of any repair process. Before deciding on any repair work, the cause of impairment must be diagnosed as clearly as possible. Sometimes the cause is obvious, but as a rule a careful investigation is required. Only afterwards should the method of repair be chosen. A thorough and logical investigation of the current condition of the
structure is the first step of any repair or rehabilitation project. After a comprehensive evaluation, the scope of both the cause and effect of concrete defect, damage and deterioration must be conducted in order to determine the necessary actions to be taken. The results of the evaluation, together with the user's needs or requirements form the necessary external information to select the repair method. Selection of repair methods depend on the nature of impairment, consideration of durability, constructability and compatibility with the existing structure, environment, availability of materials, cost and whether the repair is a temporary or permanent restoration [4, 12].

In order to assess most of the old concrete structures or those with advanced distresses, the determination of their current conditions are often necessary. The aim of this assessment is to gather information on distress importance, their causes and the severity and extent. This information is only obtained from a thorough and logical inspection, regular reports during service life of structures, and also design and construction details [3, 7].

Some of concrete structures in industrial situations are subjected to various distresses due to different activities and lack of timely attention. These impairments cause incompatibility changes on mechanical, physical and chemical concrete properties, usually along with concrete disintegration. Nevertheless, it is understood that, from a durability point of view, most concrete structures get deteriorated during their service life, where their severity and extent is affected by various factors. Unfortunately, lack of timely attention and determination of concrete distress causes, especially in places where the periodic inspections have not been implemented at regular times, often cause concrete structures to become degraded. The estimate of these damages shows high cost of repair activities. Therefore, in order to achieve an adequate evaluation and actual cause of impairment and to provide the inspection, maintenance and repair strategies along with acceptable criteria in concrete structures, the present research work was carried out in four stages. It comprises evaluation methods, inspection management, criteria of repair materials and procedures selection, and finally providing a management system for inspection and visualization according to distress mechanisms. For the application of this research, the system results are used by a case study taken from the evaluation of concrete structures in the Mobarekeh Steel Complex.

2. RESEARCH OBJECTIVES

The research objectives comprise four stages. In the first stage, repair and maintenance strategies, evaluation methods, rehabilitation of concrete structures, characteristics of deterioration and its consequences, and maintenance and durability recommendations for concrete structures were provided. Information regarding maintenance and durability of concrete is very useful for structural and construction engineers and users who may be conducting research on developing their preliminary strategies [1, 7, 11, 12].

The second stage of this research includes inspection planning; deduct values for various distresses in concrete structures, in-situ evaluation of structure according to
different NDT tests and laboratory tests. It is noted that this information is very useful for field inspectors or engineers who are involved in evaluation, repair and maintenance of concrete structures [1, 4, 6, 7, 10].

In the third stage, appropriate materials in durable mix design, repair material selection for concrete structures, concrete removal and preparation for repair methods, and application procedures and techniques for the repair of concrete were provided. This is valuable information for users and engineers who dealt with material and repair procedures selection and also those who intended to work as supervisors on construction repair sites [1, 8, 9].

In the fourth and final stage of the project, various distresses manifested in concrete structures along with their mechanisms, classification of distresses with deduct values was determined and a set of inspection forms and a potential rating system in different concrete structures were provided. At the end of this stage, the system results are used by a case study taken from the evaluation of concrete structures in the Mobarekeh Steel Complex [1, 11, 12].

3. DISTRESS CATEGORIES IN CONCRETE STRUCTURES
Identifying distresses and determining their causes thereby assisting in the design of durable replacement materials or adequate repair methods are important parts of any concrete repair program. It is important to draw conclusions based on the best observations and information available, in order to effectively rehabilitate a structure. This information may be obtained from visual inspection observations by an engineer or may be taken from pictures and databases. In some cases, it may be taken from in-situ tests (e.g. non-destructive testing (NDT)) or laboratory tests (e.g. petrographic examination) [4, 6, 10].

Concrete structures in the Mobarekeh Steel Complex with extension and various activities and enormous production lines are subjected to different deteriorations. These structures consist of cooling towers (e.g. power station, water distribution, unit 28 and 06), hot strip mill walls, acid washing unit, power tunnels, slab unit and concrete pavements. Distresses category in various concrete structures in the Mobarekeh Steel Complex which generally cover all manifestation of deteriorations are classified into physical and chemical causes (see Figure 1) [1].

4. DETERMINATION OF DETERIORATION MECHANISMS IN CONCRETE STRUCTURES
Deterioration of concrete structures follows well identifiable deterioration mechanisms. These mechanisms represent the interaction between the actual environment and the structure, with its geometry and materials composition. It is also evident that all deterioration mechanisms depend on some aggressive substance penetration from the surrounding environment into the outer layer of concrete (covercrete). The development in time of nearly all types of deterioration mechanisms of concrete structures may be modeled by two phases, the initiation phase and the propagation phase [3].

By visual inspection, in-situ and laboratory tests, and evaluation of concrete structures in the Mobarekeh Steel Complex which have been carried out by the
authors, all distresses in these structures presented in Figure 1 are in propagation, progress and development phase. Therefore, it is necessary that immediate attention must be drawn towards repair of these structures to comply with principals and technical properties. Some of deterioration causes are as follows [1]:

- Frost attack
- Sulfate attack
- Corrosion of reinforcement
- Acid attack
- Cracking due to salt expansion in the interior pores
- Surface disintegration
- Thermal stresses
- Scaling and spalling

![Distress Causes in Mobarekeh Steel Complex](image)

**Figure 1.** Distresses category in concrete structures in Mobarekeh Steel Complex

### 5. EVALUATION OF CONCRETE IN A CONCRETE STRUCTURE

A thorough and logical evaluation of the current state of the concrete in a structure is the first step of any repair or rehabilitation work. Therefore, a visual inspection of the exposed concrete is the first step in an in-situ examination of a structure. The purpose of such an examination is to locate and define areas of distress or deterioration. A condition survey will usually include a mapping of the various types of concrete deficiencies that might be found such as cracking, surface
problems (disintegration and spalling) and joint deterioration. One objective of an Evaluation Management System (EMS) is to create assessment procedures that will allow the current condition of the structure, and its components to be expressed numerically to take the best recommended action in the repair and maintenance management. The criteria for the evaluation of a concrete structure are shown in Table 1 [4].

<table>
<thead>
<tr>
<th>Zone</th>
<th>Confidence Level (CL)</th>
<th>Description</th>
<th>Recommended Action</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minor</td>
<td>95-100</td>
<td>Excellent: No noticeable impairments.</td>
<td>Prompt action is not required, but periodic investigation is recommended. In some cases, protection might be needed.</td>
</tr>
<tr>
<td></td>
<td>85-94</td>
<td>Very Good: Barely noticeable impairments. Some ageing or dusting may be visible.</td>
<td></td>
</tr>
<tr>
<td>Moderate</td>
<td>70 - 84</td>
<td>Good: Clearly noticeable impairments. Only minor defect, damage and deterioration are evident.</td>
<td>Detailed investigation and economic analysis of repair alternatives are recommended. In some cases, appropriate repair and protection methods will be needed.</td>
</tr>
<tr>
<td></td>
<td>50 - 69</td>
<td>Fair: Moderate impairments. Some defect, damage and deterioration are evident, but concrete remains serviceable.</td>
<td></td>
</tr>
<tr>
<td>Major</td>
<td>30 - 49</td>
<td>Poor: Severe impairments in at least some major components of the structure have occurred. Concrete remains serviceable.</td>
<td>Detailed investigation and an engineering evaluation should be made to determine the demand for repair, replacement strengthening and stabilization. Safety evaluation is recommended.</td>
</tr>
<tr>
<td></td>
<td>0 - 29</td>
<td>Very Poor: Very severe and extensive impairments in most components of the structure. General failure or a complete failure of structural components.</td>
<td></td>
</tr>
</tbody>
</table>

Once the condition of the structure is understood and documented, the next step in the maintenance management process is to initiate action to correct unsatisfactory conditions and to begin planning for future maintenance and repair needs. For this purpose, a quantitative rating system for the condition of concrete in a structure would make possible the determination of which components within a structure most merit repair. The Evaluation Confidence Level (ECL) extends from 0 to 100, with 0 representing Very Poor condition and 100 representing Excellent condition. The Confidence Level (CL) is divided into Minor, Moderate and Major zones.

The Confidence Level (CL) prescribed here can be applied to Mobarekeh Steel Complex concrete structures in general. The rating system described allows the Confidence Level to be determined by visual inspection using limited equipment such as binocular, covermeter, ruler and carbonation depth. Values in each parts of the survey are properly interpreted as representing the current conditions found at the time the structure was inspected and rated. The rating is related to structural integrity and serviceability of the structure. The Confidence Level system is not intended to
replace the detailed investigation needed to fully document structural deficiencies, to identify their causes and to formulate plans for correcting them. An extended investigation comprising detailed investigation and analysis, and engineering evaluation should be made when the Confidence Level is less than 50 [4].

6. DEVELOPMENT OF DEDUCT VALUES FOR VARIOUS DISTRESSES IN CONCRETE STRUCTURES

The Deduct Value is determined by visual inspection and by recording the information needed in the field inspection. The inspection and condition assessment procedure for determining Deduct Values is based on simple visual inspection techniques. If the condition of the structure being inspected is severely damaged i.e. a Confidence Level of below 50, more detailed investigation and engineering evaluation should be made.

Deduct Values for various distress categories are classified in cracking in concrete (Table 2), disintegration and scaling, spalling and delamination. An inspector should be familiar with the types of distress before performing an inspection to determine the Deduct Value. Deduct Values are based on considering previous works carried out and the author’s opinion and experience. They involve two considerations [4]:

1) The knowledge and experience of expert engineers in the safety of the structure which has been degraded by various types of distress, and
2) Serviceability of the structure.

<table>
<thead>
<tr>
<th>Surface Appearance</th>
<th>Type of Crack</th>
<th>Depth of Crack</th>
<th>Deduct Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pattern</td>
<td></td>
<td></td>
<td>Width of Crack</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Very Fine (VFI)&lt;0.25mm</td>
</tr>
<tr>
<td>21- (PCC) Crazing</td>
<td>SS</td>
<td>5</td>
<td>10</td>
</tr>
<tr>
<td>22- (PDC) D-Cracking</td>
<td>SS</td>
<td>5</td>
<td>10</td>
</tr>
<tr>
<td></td>
<td>DE</td>
<td>10</td>
<td>20</td>
</tr>
<tr>
<td></td>
<td>TH</td>
<td>15</td>
<td>30</td>
</tr>
<tr>
<td>23- (PMC) Map Cracking</td>
<td>SS</td>
<td>5</td>
<td>10</td>
</tr>
<tr>
<td></td>
<td>DE</td>
<td>10</td>
<td>20</td>
</tr>
<tr>
<td></td>
<td>TH</td>
<td>15</td>
<td>30</td>
</tr>
<tr>
<td>Individual Pattern</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>24- (ILC) Longitudinal</td>
<td>SS</td>
<td>5</td>
<td>10</td>
</tr>
<tr>
<td></td>
<td>DE</td>
<td>10</td>
<td>20</td>
</tr>
<tr>
<td></td>
<td>TH</td>
<td>20</td>
<td>30</td>
</tr>
<tr>
<td>25- (ITC) Transverse</td>
<td>SS</td>
<td>2</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>DE</td>
<td>5</td>
<td>10</td>
</tr>
<tr>
<td></td>
<td>TH</td>
<td>10</td>
<td>15</td>
</tr>
<tr>
<td>26- (IDC) Diagonal</td>
<td>SS</td>
<td>10</td>
<td>20</td>
</tr>
<tr>
<td></td>
<td>DE</td>
<td>15</td>
<td>30</td>
</tr>
<tr>
<td></td>
<td>TH</td>
<td>20</td>
<td>40</td>
</tr>
<tr>
<td>27- (IRC) Random</td>
<td>SS</td>
<td>5</td>
<td>10</td>
</tr>
<tr>
<td></td>
<td>DE</td>
<td>10</td>
<td>20</td>
</tr>
<tr>
<td></td>
<td>TH</td>
<td>15</td>
<td>25</td>
</tr>
</tbody>
</table>

Table 2: Deduct values for cracking in concrete structures

Surface and Shallow (SS) (up to 10 mm), Deep (DE) (10 –20 mm), Through (TH) (> 20 mm)
7. CALCULATION OF THE COMPONENT CONFIDENCE LEVEL (CCL)

Once the distress modes in each component of the structure to be rated are determined, the Component Confidence Level (CCL) can be calculated. By inputting the distress types into management system software, hand calculation of Deduct Values and the Confidence Level (CL) can be avoided. Considering the previous work [4, 5], generating a model for concrete distress simulation and the author’s experience and knowledge, the following formula is used for calculating the Component Confidence Level (CCL).

\[
CCL = 100 - [1.0(DV_1) + 0.4(DV_2) + 0.2(DV_3) + 0.15(DV_4) + 0.1(DV_5)]
\]  
(1)

The Component Confidence Level (CCL) is based on the five largest deduct values (DV), with DV\(_1\) the largest value and other values in descending order to the fifth largest, DV\(_5\). Table 3 shows an example of how the Component Confidence Level (CCL) for a column of cooling tower unit 28 in Mobarekeh Steel Complex has been calculated [1].

Table 3: Example of calculation of the CCL for a column of cooling tower unit 28 in Mobarekeh Steel Complex

<table>
<thead>
<tr>
<th>Step 1: Inspect component to determine distresses and quantities.</th>
<th>Step 2: Calculate Deduct Values for each distress.</th>
<th>Step 3: Rank the Deduct Values in descending order to the smallest. Only the five largest are used in the Component Confidence Level (CCL) calculation.</th>
<th>Step 4: Calculate the CCL based on the ranked Deduct Values:</th>
</tr>
</thead>
<tbody>
<tr>
<td>(24)</td>
<td>(51)</td>
<td>(32)</td>
<td>(43)</td>
</tr>
<tr>
<td>ILC-TH-WI</td>
<td>SC-SL</td>
<td>LD-SL</td>
<td>FT-SE</td>
</tr>
<tr>
<td>(70)</td>
<td>(10)</td>
<td>(5)</td>
<td>(40)</td>
</tr>
</tbody>
</table>
| Step 4: Calculate the CCL based on the ranked Deduct Values:  
CCL = 100 – [1.0(DV\(_1\)) + 0.4(DV\(_2\)) + 0.2(DV\(_3\)) + 0.15(DV\(_4\)) + 0.1(DV\(_5\))]  
CCL = 100 – [1.0(70) + 0.4(40) + 0.2(40) + 0.15(30) + 0.1(4)] = 0.50 |

The CCL is 0.5 which is Very Poor according to Table 1 (Very severe and extensive impairments in most columns of the structure).
8. INSPECTION AND VISUALIZATION FORMS FOR CONCRETE STRUCTURES

With the purpose of evaluation of current concrete structures application and according to previous sections, deteriorations are classified in five groups and various distresses are coded as shown in Figure 2. If distresses in concrete structures are in propagation phase, the evaluation of current condition of structure is necessary. Evaluation of current state of structure with the other factors such as current maintenance, rehabilitation, change of serviceability condition, consideration of structure stability and study of concrete application in current environment condition must also be paid attention [2, 3].

The purpose of this evaluation is to gather information about distresses severity and extent. This information is only obtained from a thorough and logical inspection and completed inspection forms for concrete structures. In addition, for planning maintenance of structure during its service life, periodic and regular inspections and preventing from distress progress and development which are important for enhancing service life of concrete structures, different inspection forms in a set of 12 sheets are provided [1].

![Distress coding in a Visual Inspection of Concrete Structures](image)

Figure 2. Distress coding in a visual inspection in a concrete structure.
9. CONCLUSION
One objective of an evaluation management system is to create assessment procedures that will allow the current condition of the structure and its components to be expressed numerically so as to assist in choosing the best course of action in the repair and maintenance management. Engineering judgment and experience were needed to develop a set of criteria in order to implement a quantitative rating of the overall state of concrete using the results of the observation of signs of distress and weighting scales based on severity and extent. It is important that the observed conditions be described in unambiguous terms that can be used by the user to enable him to take engineering and management actions for the repair and maintenance of the structure.

Once the current conditions of structure are documented and determined, the next step in the process of repair and maintenance management is applying an engineering action for correction of inadequate conditions and starting planning for future repair and maintenance requirements. For this purpose, the presented quantity rating system can be applied in a concrete structure in order to determine the components needed repair. Also, with completion of inspection and visualization forms and periodic investigations, the inspector could prevent the extent of deteriorations and enhance the service life of concrete structures.

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A note of appreciation and special thanks to Engineer Shahriari, Deputy of Technology, Dr Ezadi, Director of Research and Development, and Engineer Aghajani, Director of Technical Support, Mobarekeh Steel Complex for their support and help during this research.

REFERENCES
ABSTRACT

Strengthening of structures to avoid future damages is inevitable in different conditions. In recent years, fiber reinforced polymer (FRP) sheets are vastly used for strengthening of different concrete members. In this paper, to evaluate the effect of FRP plates on damaged or non-damaged members, an extensive experimental and numerical program is prepared. For this purpose, 10 beam specimens are built. Some of the beams are made under loading condition and their load-deformation curves are prepared; then, the damaged beams are strengthened with CFRP and GFRP using epoxy resin and have been tested under loading. The second part of the beam specimens are strengthened before loading and then their load-deformation curves are prepared. Some specimens are analyzed using Finite Element Method [1]. The results of numerical model and experimental research work show the improvement of bearing capacity of strengthened damaged beams compared to the non-damaged ones.

Keywords: concrete, damaged-beams, CFRP, GFRP, retrofit

1. INTRODUCTION

In recent years, the use of fiber reinforced polymers (FRP) as an externaly strip have achieved considerable popularity for the strengthening and repair of concrete structures. The FRP composites have been used successfully for rehabilitation and strengthening of deficient reinforced concrete elements. The potential market for such applications is huge since the estimated annual cost of repairing bridges in the United States alone is 9.4 billion dollars [2].

Chen and Teng (2003) presented a simple, accurate and rational method to study the shear capacity of FRP-strengthened beams which fail by FRP debonding. A new shear strength model is then developed, which was validated using experimental data collected from the existing literature [3]. In 2006 a series of 4-point bending experiments of the proposed hybrid FRP–concrete beam model were carried out by Wu et al. In addition, based on the principles of strain compatibility and equilibrium, an iterative analytical model was developed to analyze the flexural behavior of hybrid system [4].

Chen et al. (2006) presented a finite element analysis on the stress distributions in a typical shear test set-up for FRP-to-concrete bond strength. They show that the stress distribution is significantly different from plane stress assumption primarily
because of the difference between the width of the tested FRP plate and that of the concrete block [5].

Aram et al (2007) investigate the Debonding failure modes of flexural FRP-strengthened RC beams. In this paper, different types of debonding failure modes are described. Then, experimental results of four-point bending tests on FRP strengthened RC beams are presented and debonding failure mechanisms of strengthened beams are investigated using analytical and finite element solutions [6].

Oehlers et al (2007) presented intermediate crack debonding resistance of groups of FRP NSM strips in reinforced concrete beams. This paper develops a mathematical model for the intermediate crack debonding resistance of groups of NSM plates for use in the flexural and shear strengthening of reinforced concrete beams [7].

Following the above mentioned researches, in this paper the strength and behavior of Damaged-Concrete Beams retrofitted by FRP layers are studied and their difference with non damaged ones are presented.

2. EXPERIMENTAL INVESTIGATION

2.1. Specimen Design

Ten beam specimens were fabricated and tested. Two of these specimens were tested in their virgin conditions to serve as control, while the remaining eight beam specimens were tested after being strengthened using carbon and glass fibers. All of the RC beams were designed to have the same nominal dimensions: 2000 mm long, 200 mm wide and 300 mm height, with a span of 1800 mm. As shown in Figure 1, the flexural reinforcement consisted of 2T12 deformed bars. The shear reinforcement consisted of 1T8 distanced of 50 mm. Strengthening of specimens was achieved by the external strip of unidirectional carbon and glass fiber sheets using epoxy resin.

![Figure 1. Detail of RC beam bonded with CFRP](image)

2.2. Specimens Preparation

All eight RC beam specimens were fabricated with a normal density concrete mix. After the specimens strengthening was achieved by the external strip of unidirectional carbon and glass fiber sheets. Large-scale retrofitted RC beam tests indicate that failures of FRP plated systems may take place through various possible mechanisms, depending on the concrete grade, rebar provision, FRP
properties, and service environments. Identified failure modes include: (1) concrete crushing before steel yielding; (2) steel yielding followed by concrete crushing; (3) steel yielding followed by FRP rupture; (4) Shear failure; (5) concrete cover de-lamination and (6) de-bonding in the vicinity of the FRP / epoxy / concrete Bond interface. Debonding in a FRP bonded concrete system is a complex phenomenon. At the bottom of the concrete core, CFRP and GFRP strips are axially bonded to carry tensile load and assure the stiffness of the member. Outside the CFRP and GFRP strips, CFRP and GFRP strips are hoops directionally wrapped to provide confinement to the end of concrete core and also to prevent the premature debonding of interior CFRP and GFRP strips (Figures 2, 3). Certain layers of carbon and glass fiber sheets were bonded along the axial direction (at 0 angle) at the bottom surface of the concrete core by epoxy resin and then carbon fiber sheets were wrapped round the beam at 90 angle. Electro-thermal blanket was used in some cases to maintain the curing temperature above 25°C.

![Figure 2. RC beam bonded with CFRP](image1)

![Figure 3. RC beam bonded with GFRP](image2)

### 2.3. Material Properties

The compressive strength of concrete which has been prepared in the laboratory was equal to 33 MPa. The flexural reinforcement consisted of 2T12 deformed bars of yield strength 485 MPa. The shear reinforcement consisted of 1T8 distanced of 50 mm deformed bars of yield strength 430 MPa. The tensile properties of the CFRP and GFRP materials are listed in Table 1, which were determined according to ASTM D3039/D3039M-95a [8]. The corresponding mechanical properties of resin used in experiments are listed in Table 2.

<table>
<thead>
<tr>
<th>CFRP material</th>
<th>Strip</th>
<th>Wrap</th>
</tr>
</thead>
<tbody>
<tr>
<td>Width (mm)</td>
<td>200</td>
<td>300</td>
</tr>
<tr>
<td>Thickness (mm)</td>
<td>0.61</td>
<td>0.61</td>
</tr>
<tr>
<td>Tensile strength (MPa)</td>
<td>3800</td>
<td>3800</td>
</tr>
<tr>
<td>Tensile modulus of Elasticity (GPa)</td>
<td>242</td>
<td>242</td>
</tr>
<tr>
<td>Failure strain (%)</td>
<td>1.5</td>
<td>1.5</td>
</tr>
</tbody>
</table>
### Table 2. Properties of resins

<table>
<thead>
<tr>
<th>Resin type</th>
<th>Epoxy (ML 506)</th>
<th>Polyester (HN160)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density (gr/cm³)</td>
<td>1.11</td>
<td>1.08</td>
</tr>
<tr>
<td>Adhesive Tensile</td>
<td>76.1</td>
<td>55</td>
</tr>
<tr>
<td>Strength (MPa)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Modulus of Elasticity (GPa)</td>
<td>2.789</td>
<td>1.833</td>
</tr>
</tbody>
</table>

#### 2.4. Strengthening Materials and Methods

Beam specimens B1 and B2 were tested in its virgin condition to serve as control (Figure 6). Beam specimens B3 and B4 were strengthened with CFRP strip. Its dimension was: 2000 mm long and 200 mm. Beam specimens B5 and B6 were strengthened using GFRP, the strips were similar to beams B3 and B4. The beam specimens B7 to B10 were loaded before strengthening until the maximum crack width was reached to 0.4mm and then the beam specimens B7 and B8 were strengthened with CFRP strips similar to B3 and B4. The beam specimens B9 and B10 were strengthened with GFRP similar to B5 and B6.

![Figure 6. RC beam in its virgin condition](image)

#### 2.5. Instrumentation and Test Procedure

The beam specimens were simply supported over a span of 1600mm(L) and tested in flexure under two symmetrical point loads, thus giving a L/h ratio of 5.33 and an a/h ratio of 2, where a is the shear span. The load was applied using a servo controlled hydraulic actuator with a maximum capacity of 1000KN according to ASTM C78-00 [9]. The load rate constantly increased the extreme fiber stress between 0.86 and 1.21 MPa/min. The load was applied monotonically up to failure. First linear variable displacement transducer (LVDT) was placed under the mid-span and second LVDT were similarly placed under the beam at the load position. Figure 6 gives the positions of LVDTs.
3. TEST RESULT AND DISCUSSION

The loads vs. mid-span deflections of the beams are presented in Figures 7-11.

As expected, beams strengthening by external bonding of different FRP materials resulted in an increase in stiffness, the highest increase being exhibited in beam specimens B3 and B4 that strengthened by CFRP and epoxy (ML-506) resin. The loads corresponding to the appearance of flexural crack are presented in Table 3. The first crack appeared at the bottom of mid-span section and then propagated to the top of zone.
All the beam specimens failed in bending. The beams B1, B2 failed by crushing of compressive concrete in load of 51KN. B3, B4 which were strengthened by CFRP and resin epoxy failed at a load 33% higher than the beams B1, B2 that its load was 68KN. Beams B5, B6 also show a 15% increase in yielding strength where FRP strip failed that its load was 59KN. Specimens B7, B8 failed at a load lower than B3, B4. Their yielding loads were almost half of yielding load of B3, B4 because of they were damaged. They failed at a load 14% higher than the specimens B1, B2 that its load was 58KN. B9, B10 show a 7% increase in yielding strength its yielding load that FRP strip failed was 54KN. Similar to B7, B8 their yielding load were almost half of the yielding load of B5, B6. The failure of all of strengthening beams was initiated due to failure of the bottom strip. In the table 3, the yield load where FRP strips failed is presented.

<table>
<thead>
<tr>
<th>Beam</th>
<th>Yield Load (KN)</th>
<th>Increase in yielding load after strengthening (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1, B2</td>
<td>51</td>
<td></td>
</tr>
<tr>
<td>B3, B4</td>
<td>68</td>
<td>33</td>
</tr>
<tr>
<td>B5, B6</td>
<td>59</td>
<td>15</td>
</tr>
<tr>
<td>B7, B8</td>
<td>58</td>
<td>14</td>
</tr>
<tr>
<td>B9, B10</td>
<td>54</td>
<td>7</td>
</tr>
</tbody>
</table>

4. FINITE ELEMENT MODELING
Linear elastic FE analyses were carried out to investigate the Load – Displacement. The concrete block was taken to be 1000 mm long, 300 mm thick and 100 mm wide. The Youngs modulus was assumed to be 200, 2.789 and 31.6GPa, respectively for the plate, adhesive and concrete. The corresponding Poissons ratios were 0.3, 0.3 and 0.15, respectively. The geometry, loading, boundary conditions and adopted coordinate system are shown in Figure 12. Only a quarter of the beam was modeled using the symmetrical configuration in the x–y plane. Numerical analyses were conducted using ANSYS.9. Concrete was modeled using solid65 and plate, adhesive, CFRP and GFRP layers were modeled using solid45, and reinforcing bars were modeled using link8 [1]. The element size for all meshes was 25 mm. In most tests, failure occurs within the concrete in top of beams. The comparison between numerical analysis and experimental test results are shown in Figures 13-17.
5. COMPARISON OF NUMERICAL AND TEST RESULTS

Comparison in terms of load–midspan displacement curves, with the numerical results obtained by the finite element analysis are shown in Figures 13-17. It can be initially observed from the Figures that the straight lines obtained by the FEM analysis provide a good approximation for the curves obtained experimentally, with close agreement regarding the beam stiffness. On comparing now the experimental results from the bending tests with the numerical ones, it can be observed that, for the all specimens the FEM analyses curve lies a little above the curves from the experimental, giving slightly lower displacements for the same applied load value. These differences are probably caused by the differences between the elastic properties estimated in the analyses and the true ones, and also due to difficulties in reproducing the true boundary conditions of the experimental tests in the models. It should be noted, however, that the differences are small in comparison with the variations observed in the experimental measurements.

6. CONCLUSIONS

This paper deals with bending strength of the reinforced concrete damaged beams using an externally CFRP and GFRP strip. Test result of ten retrofitted beams in states of before and after damage are presented and discussed. This paper clearly shows that there is difference in strength between using CFRP with epoxy resin or GFRP with epoxy resin in concrete beams; moreover, there is a high difference between retrofitting of damaged or non damaged beams.
REFERENCES


REPAIR AND RETROFIT OF RC COLUMNS WITH PRE-STRESSED BANDS FOR IMPROVED SEISMIC BEHAVIOUR

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²Islamic Azad University of Mashhad, Iran

ABSTRACT
A new technique of seismic retrofit of RC columns has been applied in this paper. Pre-stressed high strength metal strips are externally applied to RC columns. Eight large-scale RC columns with insufficient transverse reinforcements were tested under constant axial and cyclic lateral load reversals. The level of axial load and the pattern of strengthening were parameters of this study. The ability of the technique for improving the ductility of RC columns and increasing the lateral strength of columns as well as repair of damaged columns was studied. It was observed that the technique is capable to enhance the lateral behavior of RC columns significantly. Very ductile behavior was achieved. The height-wise variations of lateral strain on confining strips are studied.

Keywords: RC column, seismic behavior, earthquake, repair, seismic retrofit

1. INTRODUCTION
Major failure modes that have been reported for RC columns in previous earthquakes or laboratory tests including shear failure, lap-splice failure, buckling of longitudinal bars inflexural failure, insufficient capacity of plastic hinge rotation can mainly be interpreted due to the inadequate confinement. Several experimental studies have been performed on application of various techniques of seismic retrofit of reinforced concrete columns such as RC jacketing, steel jacketing, glued steel bands, FRP jacketing, FRP collars, post-tensioned cables, etc. [1-5]
In this paper, an easy and innovative technique of retrofit of concrete is applied for enhancing the lateral behavior of concrete columns. This technique involves post-tensioning high-strength packaging straps around the column and subsequently locking their ends in metal clips. Various configurations have been applied in laboratory tests of reinforced concrete columns including cantilever, double curvature, double ended, flexible base and hammerhead. [7] In this study the cantilever configuration was selected in which a constant axial load is applied and then lateral reversal cyclic displacements are applied by means of hydraulic jacks. This paper presents the experimental results of a study on the application of this retrofit technique on large-scale models of building columns. The main objective of this research was quantification of the improvement of RC columns behavior by applying this technique. This technique was previously applied for retrofitting the compressive behavior of small scale concrete columns with various
shapes and sizes [8]. It could also increase the compressive strength and ductility of spirally reinforced cylindrical columns [9].

2. APPLIED RETROFIT TECHNIQUE
The technique used for retrofitting concrete columns in this study, involves post-tensioning high-strength packaging straps around the column (by using standard strapping machines used in the packaging industry) and subsequently locking their ends in metal clips, as shown in Figure 1.

![Strapping mechanism and devices for retrofit of RC members](image)

Commercially available strapping tensioners and sealers make it easy to pretension the strip and fix the strip ends in the clamps. The available straps have widths of 10 to 50 mm and thicknesses of 0.5 to 1.12 mm. In terms of strength, high strength strips in excess of 10000kg/cm², are available in the market. The strips are tensioned to 30 percent of their yield stress. Hence, an effective lateral stress is applied on the column prior to loading. This has many benefits such as full utilization of the strip capacity and prevention from premature crushing of the confined concrete, as would be the case with not properly tightened strips.

The low cost of strip and speed and ease of application of the strapping technique make this method efficient for use as a repair and strengthening technique for RC structural members. An RC column would normally require six man days’ work to be jacketed whilst a maximum of two days’ work is required for external strapping, which clearly demonstrates the cost saving when using the proposed technique. (Frangou & Pilakoutas 1995)

3. EXPERIMENTAL PROGRAM
Eight 2/3 scale models of reinforced concrete building columns with inadequate transverse reinforcement were made and tested under constant axial load and cyclic lateral displacement reversals. In Figure 1 details of reinforcement of these columns is shown. The specimens were tested under one of the two considered levels of axial compression that were equal with 0.19f'cAg and 0.38 f'cAg.

The yield strength of longitudinal and transverse bars for all specimens was 550MPa and 600MPa, respectively. The average strength of concrete was 26MPa. The retrofit technique that was applied in this study involves strapping the concrete columns with high strength metal strips. The strips are tensioned with a pneumatic tensioner and then the both ends of the strip are locked in a seal by means of a
sealer. Calibration tests were initially conducted and a linear relationship was obtained between air pressure and tensioning force in the strip. This linear relationship was then used in retrofit of RC columns. The yield and ultimate strength of applied metal strips were 850 and 950 MPa, respectively.

Four distinctive failure modes have been reported for RC columns under seismic conditions, including flexural failure, shear failure, shear-flexural failure and lap-splice failure. Typically, in columns of building structures, the aspect ratio and shear span to depth ratio are greater than 8 and 3, respectively. Therefore, the flexural and shear-flexural failure modes have more frequently been reported for building columns. Design of columns was conducted with the aim of achieving a shear-flexural failure mode. So, the ratio of $V_p/V_n$ was so designated that the failure mode of column would be shear-flexural mode, in which shear failure occurs following to the flexural failure.

Test parameters included axial load and retrofitting pattern. Details of column specimens are presented in table 1. Two different amounts of strips were applied through height of columns. As shown in Figure 2, the column height was divided into two different regions of $L_1$ and $L_2$. Generally more confinement was provided in the first region.

For each level of axial load, a control specimen was tested as the basis for quantification of the improvement developed due to strapping the column. One of the retrofitted columns, i.e. $C_2$ was tested under the first level of axial load. This specimen was slightly retrofitted with strips placed at the middle between the internal ties.
Table 1: Test Matrix

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Axial load</th>
<th>Retrofit layout</th>
<th>L1 (mm)</th>
<th>Number of strip layers</th>
<th>S1 (mm)</th>
<th>L2 (mm)</th>
<th>Number of strip layers</th>
<th>S2 (m)</th>
</tr>
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<tbody>
<tr>
<td>C1</td>
<td>0.19 Ag. f’c</td>
<td>Control specimen for first level of axial load</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C2</td>
<td>0.19 Ag. f’c</td>
<td>65</td>
<td>1</td>
<td>32</td>
<td>687</td>
<td>1</td>
<td>150</td>
<td></td>
</tr>
<tr>
<td>C3</td>
<td>0.38 Ag. f’c</td>
<td>Control specimen for second level of axial load</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C4</td>
<td>0.38 Ag. f’c</td>
<td>198</td>
<td>2</td>
<td>33</td>
<td>285</td>
<td>1</td>
<td>36</td>
<td></td>
</tr>
<tr>
<td>C5</td>
<td>0.38 Ag. f’c</td>
<td>198</td>
<td>2</td>
<td>33</td>
<td>285</td>
<td>1</td>
<td>36</td>
<td></td>
</tr>
<tr>
<td>C6</td>
<td>0.38 Ag. f’c</td>
<td>132</td>
<td>2</td>
<td>33</td>
<td>250</td>
<td>1</td>
<td>83</td>
<td></td>
</tr>
<tr>
<td>C7</td>
<td>0.38 Ag. f’c</td>
<td>280</td>
<td>2</td>
<td>71</td>
<td>213</td>
<td>1</td>
<td>71</td>
<td></td>
</tr>
<tr>
<td>C8</td>
<td>0.38 Ag. f’c</td>
<td>132</td>
<td>2</td>
<td>33</td>
<td>332</td>
<td>1</td>
<td>83</td>
<td></td>
</tr>
</tbody>
</table>

Three columns were tested under the second level of axial load. Specimens C4 and C5 were retrofitted with the same layout. The only difference between these specimens was the form of column section. Four wood pieces were attached to each side of column and the form of column section was changed from square to circular. This increases the confinement effectiveness. The lower part of the last column, C6, was fully jacketed with two layer strips from column base to an elevation of 132mm which is the elevation of the first internal tie. From this elevation to elevation of 382mm, which is about 1.5 times the column width, one layer strips were applied at a spacing of 83 mm. Along the vertical edges of the columns a radius of 25mm fillet was formed by including suitable block outs in the formwork to prevent stress concentration that may cause premature failure of strips at the corners as well as to increase the geometrical confinement effectiveness.

The column specimen C7 was initially tested without any retrofit under the higher level of axial load until the lateral strength decreased by 20% of the peak lateral strength. Parts of the cover concrete that have been crushed during the first test were removed and re-filled with a cement based mortar with addition of the epoxy adhesive. The damaged specimen was then retrofitted with steel strips and re-tested. (Figure 3)

Figure 3. Column C7; a) damaged state, b) end of test of repaired column (15% drift ratio)
In order to assess the possibility of increasing the lateral strength of RC columns with application of “Near Surface Mounted” reinforcement together with the strapping technique, column specimen C8 was made and tested. Increasing the strength of a RC column is a necessary issue in the cases of a weak column and strong beam in RC frame buildings. A longitudinal groove was created at each one of the two tensile-compressive faces of column (Figure 4). Then a deformed 12 mm diameter bar was glued inside the groove and the groove was filled with a cement-based mortar. No chemical adhesive was applied.

3.1. Test Setup and Instrumentation
The column stubs were fastened to the strong floor with eight high-strength rods, and each rod was pre-stressed to 200 KN to prevent slip and overturning under large lateral load. A vertical hydraulic jack was used to apply the constant axial load which was controlled by a load cell. Two horizontal hydraulic jacks were utilized to apply lateral cyclic loading. Six vertical LVDTs, four horizontal LVDTs and four inclined LVDTs were used to measure the columns curvature, lateral displacement and shear deformation, respectively. In addition, two LVDTs were used to measure the width of developed crack at the interface of the column and end beam. Ten Electrical resistance strain gages glued to longitudinal and transverse reinforcements and twenty strain gages for measuring the strip strains were used.

The axial load was held constant during the test by vertical hydraulic jack shown in Figure 2. The effect of earthquake on the column specimens was simulated by reversed cyclic loading. Two hydraulic jacks in the test setup were used to displace the top of the columns to achieve a predetermined displacement level. Then the loading direction was reversed to achieve the same displacement level in the opposite direction. The lateral force was applied in the displacement control mode,
consisting of incrementally increasing lateral drift cycles at 0.5, 1.0, 2.0%, etc., until the load resistance dropped by 30%.

4. TEST RESULTS AND OBSERVATIONS

Since fraction of the applied lateral load is cancelled with the horizontal component of the axial load, especially in large drifts, it is necessary to calculate the net applied lateral load. Instead, since the failure modes of retrofitted columns were flexural, the moment at the base of column was drawn versus lateral drift. This drawn moment includes the moment induced by the axial load, i.e. known as the effect of $P-\Delta$.

In Figure 5, the moment versus drift ratio for columns tested under the lower level of axial load are shown. The first control specimen, i.e. C1, behaved quite well. Although the distance between internal tie reinforcements was very large, but due to the large yield strength and suitable configuration of these reinforcement and also the small amount of axial load, C1 showed a ductile behavior. In this specimen, after concrete cracking in tension and yield of longitudinal reinforcements, cover concrete spalled off and subsequently longitudinal reinforcements buckled and considerable loss in lateral strength occurred. Specimen C2 was slightly reinforced with one layer of metal strips. The small amount of confining strips could enhance the behavior of column. The inclined shear cracks were eliminated. Since the first two strips from the column base were applied with no spacing, the cracking and crushing of cover concrete occurred only between the second and the third strips. Longitudinal bar buckling was observed after cover spalling and subsequently longitudinal bars ruptured one by one. It was concluded that the ductility of a relatively ductile column can be enhanced with a small amount of metal strips.

![Figure 5. Hysteretic lateral behavior of columns tested under the lower axial load level](image_url)

In contrast, the second control specimen, C3, that was tested under higher axial load showed a brittle behavior. At 3% drift, suddenly the cover concrete spalled off and longitudinal bar buckling occurred. Shear cracks were also observed on side faces (Figure 6). Specimen C4 showed a very ductile behavior. Due to the effective confinement applied no cracks or damage was created in the column until failure except crack at the interface of column base and foundation. By increasing the lateral drift, this
crack was opened until the longitudinal bars ruptured. In addition to the ductility, the lateral strength of this column was also increased. None of the strips failed until the end of the test. In this specimen the length of plastic hinge was very small, about 3cm!

Figure 6. Hysteretic lateral behavior of columns tested under the higher axial load level

Both strength and ductility of column C5, with a retrofit layout the same as that of C4 but without wood, enhanced considerably. Cracks took place between adjacent strips in addition to the interface crack. So, on the contrary to C4 in which the damage was concentrated only at the interface of column and foundation, the plasticity was distributed over a particular length and therefore longitudinal bar rupture did not occur. Loading was continued to very great values of drift ratio but no rupture was observed in strips or bars.

Similarly, column specimen C6 showed a very ductile behavior. No rupture was observed in confining strips. The main difference between C6 with lower amount of confinement with C5 was its smaller lateral strength. In terms of ductility, both columns behaved very ductile and no strip rupture occurred. The moment-drift relationships of these specimens are shown in Figure 4. It can be observed that the fully jacketed specimens, C4 and C5, have achieved higher strength values. This is because the strips of compressive face lean on each other and resist some compressive forces.

In Figure 7A, envelope of recorded hysteretic moment-drift response of column specimen C7 before and after repair with strapping is shown. The column was initially tested without any retrofit until its lateral load carrying capacity decreased by 20%. The dashed line shows the rest of the curve obtained from control
specimen (C3). The imposed damage level was really significant including complete spalling of the cover concrete in the plastic hinge region, yielding of longitudinal reinforcement in tension and buckling in compression as shown in Figure 3. Such a damage state has been defined as the failure or collapse performance level in the literature. After repairing the column with strapping, the column behaved acceptably. Although it showed less stiffness than the initial undamaged column but it could suffer large lateral displacement and very ductile behavior.

Hysteretic behavior of column C8 with addition of near surface mounted longitudinal bars and external strapping is compared with that of control specimen C3 in Figure 7B. This technique could provide 30% increase in lateral strength as well as considerable enhancement in column ductility.

\[ \text{Figure 7. a) behavior of C7 before and after repair, b) Hysteretic behavior of column C8} \]

5. ANALYSIS OF THE RESULTS

5.1. Envelope of Moment-Drift Ratio

The lateral displacement of a RC column consists of several fractions including flexural deformation, shear deformation and bar slip. The lateral displacement due to flexural deformations is usually obtained by using fiber modeling approach. In this approach, the uniaxial stress-strain behavior of the three parts of column section, i.e. cover concrete, longitudinal bars and core concrete is assigned to the corresponding part. The moment-drift ratio of two columns were obtained by this method and compared to the experimentally obtained relationship. It shall be noted that the shear deformations were not taken into account in these analyses and Mander et al. 1988 confinement model was used for estimation of the stress-strain of concrete confined with strips or internal strips. In analysis of specimen C5 that is confined with external strips, the stress-strain behavior of cover concrete and was calculated based on lateral pressure created by external metal strips. On the other hand, the stress-strain of core concrete was obtained by a two step approach. At the first step, peak strength and corresponding strain of the core concrete confined with internal transverse reinforcements is obtained by application of any confinement model. At the second step, the aforementioned properties are assumed as the properties of plain concrete that is confined with external strips.
As can be seen in this Figure, the analytically obtained curve is relatively in good agreement with experimental results. For both specimens, the analysis has overestimated the stiffness of pre-peak branch of curve. This is mainly because the lateral displacement due to shear deformations has not been considered in the analyses.

5.2. Evolution of the Lateral Strains in Strips
The recorded strains of confining strips were studied for column specimen C4 at the peak of each loading cycle. In Figure 9, strains of five strain gages that were attached on strips of a side face of column, recorded at various drift ratios, are drawn versus elevation of the strip from column base. It can be observed that at an elevation of about 13.5 cm, i.e. the elevation of internal transverse reinforcement, the strip strains has not been increased with increasing the lateral displacement. This is mainly because the internal ties have prevented the dilation of concrete, decreased the shear deformations, reduced the lateral deformation due to longitudinal bar buckling and also decreased the opening of cracks at that elevation. The strains of the lowest strip show the greatest values among the height of column. This implies that the column behavior has been flexural, because for a column with shear deformations the inclined shear cracks at a height of equal with one column width tend to widen and therefore induce tension in lateral confinement over this height. It is also observed in this Figure that there is not any significant difference between the strain values of each strain gage in push and pull cycles.
Variation of strains of each strain gage at the peak points of each loading step are drawn in Figure 10. It can be observed that although in push and pull cycles, strains at side faces are the same, but they result in an asymmetry in curves corresponding to strains at column corners. In addition, Figure 10 shows that only the strains of the first three strips from column base has evolved noticeably and very small strain values have been induced in upper strips.

6. CONCLUSION
A new retrofit technique for seismic enhancement of RC columns was described and applied to eight square sectioned RC columns with inadequate transverse reinforcement. Column specimens were tested under constant axial load and cyclic lateral displacement. It was observed that this technique is capable to enhance ductility of brittle RC columns considerably. By application of ductile confining
strips with effective sealing type, none of the strips fail even under large lateral displacements. The fiber modeling approach can reasonably estimate the lateral behavior of flexural RC members. Due to the flexural behavior of RC columns, only the first three strips from the base of column experience large strains. Mainly the strips that are located between column base to the first internal reinforcement are stressed that by suitable confining this region, considerable ductility enhancement is achieved. Place and length of plastic hinge in retrofitted column is directly dependent to the layout of retrofit. The technique was also successfully applied in repair of severely damaged RC column. In addition, it can be employed together with NSM bar technique to increase the strength as well as ductility of columns.

REFERENCES
A COMPARATIVE STUDY ON COMpressive BEHAVIOR OF
H.S. STEEL VS. FRP CONFtENED CONCRETE

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²Islamic Azad University of Mashhad, Iran

ABSTRACT
This paper presents the results of an experimental and analytical study on application of “high strength steel strapping” technique and comparing it with FRP for retrofit of concrete columns. Various parameters were found to influence the compressive strength and ductility of confined concrete, including confinement mechanical volumetric ratio, number of confinement layers, strength of plain concrete and ductility of confining material. Among these parameters, the latter was found to play the most important role in determining the ultimate strain and post peak behavior of concrete. Axial compressive tests were performed on small-scale circular or square section concrete columns. Three different materials were applied for confining concrete specimens, including CFRP jacket, brittle high-strength steel strips and ductile steel strip. Test Results showed significant increase in strength and ductility of columns due to active confinement by metal strips. CFRP confined concrete also showed enhanced behavior. A database of results of compressive tests on concrete confined with various materials was collected from the literature. An analytical model was proposed based on results of this study and the collected database to determine the strength and ultimate strain of confined concrete. The proposed model takes the confinement ductility into account and shows good agreement with the experimental results.

Keywords: concrete, confinement, FRP, steel strip, ultimate strain

1. INTRODUCTION
It is well known and proven that lateral confinement improves the strength and ductility of concrete. Confinement reinforcement is generally applied to compressive members as lateral reinforcement with the aim of increasing their load carrying capacity and their ductility in case of seismic upgrading. In addition, lateral confinement prevents slippage and buckling of the longitudinal reinforcement (Saadatmanesh et al., 1994). Lateral reinforcement can be provided by using circular hoops, rectangular ties, jacketing by steel, FRP, ferrocement, etc. Since many of the existing RC columns are vulnerable under severe earthquakes due to low ductility, increasing the concrete compressive displacement capacity by confinement becomes a vital issue. Several researches have been conducted in the field of strength and ductility enhancement of concrete by confinement with various materials.
In addition, various models have been proposed for approximating the gain in strength, peak strain and ultimate strain due to confinement. Since the confinement ductility has not been considered as a parameter of study, in almost all of the existing experiments and models, the effect of ductility of confining material on ductility enhancement of concrete has been missed. This paper presents the results of an experimental and analytical study that focuses on this issue. This study was part of a comprehensive investigation on different techniques of concrete retrofit. The study included axial compressive tests on concrete specimens with square or circular sections with or without internal confining bars that were retrofitted with two types of metal strips as well as CFRP jackets.

Confinement models for peak of the compressive behavior a review of the available confining models in the literature shows that almost all of the existing confinement models include an identical form in which strength of confined concrete and the corresponding peak strain is a function of effective lateral pressure $f_{le}$ and strength of plain concrete $f'_c$ as rewritten in Table 1. One of the main differences of these models is the assumed parameters or approaches in computing the effective lateral pressure. Some models use the yield force of confining material for computing the lateral pressure, while a few of these models try to obtain the existing stress in the confining material at the peak axial stress. However, this issue is more important for estimating the strength of steel-confined concrete.

In addition, in some models, the lateral pressure is decreased to account for the ineffectively confined zones of concrete columns which was firstly introduced by Sheikh and Uzumeri 1982 and by Mander et al. 1984 and then applied in EC8. Therefore, according to the most advanced confinement models, the effective lateral pressure is a function of mechanical volumetric ratio of confining material and geometry and dimensions of concrete column and its longitudinal and transverse reinforcements.

Confinement Models for Ultimate Compressive Strain
In contrast to the peak point in stress-strain behavior of concrete, ultimate compressive strain has not been consistently defined with various researchers. In contrast to the tensile tests, in which an apparent rupture could be observed, the definition of the ultimate point in compressive tests of concrete is a controversial issue. For steel-confined concrete, CEB model code 90 (1993) uses the strains on the post-peak branches of stress-strain curves of confined and unconfined concretes that corresponds to a stress level of 85% of strength of unconfined concrete as the ultimate strain of confined and unconfined concretes, respectively. Cusson and Paultré (1995) used the strains at which the stress drops to 50% of corresponding strengths of confined or unconfined concrete as the ultimate strains of confined or unconfined concretes, respectively. Razvi and Saatcioglu 1999 used an approach similar to that of Cusson and Paultré 1995, in which strains at 85% of strength of confined and unconfined concretes are applied as the ultimate strains of confined and unconfined concretes, respectively. It is observed that there is much difference between theses measures.

However, for FRP confined concrete an obvious ultimate point can be observed
which corresponds to the rupture of FRP jacket. Lam and Teng 2003

### Table 1: A Summary of Famous Steel-Based Confinement Models

<table>
<thead>
<tr>
<th>Model</th>
<th>Confined concrete strength</th>
<th>Strain at peak stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>Richart 1928</td>
<td>$f'<em>{cc} = f</em>{co}[1 + 4.1f_{te}]$</td>
<td></td>
</tr>
<tr>
<td>Newman 1971</td>
<td>$f_{co} = f_{ce} \left[ 1 + 3.7 \left(\frac{f_{te}}{f_{co}}\right)^{0.06} \right]$</td>
<td></td>
</tr>
<tr>
<td>Sheikh and Uzumeri 1982</td>
<td>$f_{co} = K f_{ce}$&lt;br&gt;$K = 1.0 - 0.14f_{te}$&lt;br&gt;$f'<em>{cc} = f</em>{co}(1 + 2 \frac{f_{te}}{f_{co}})$</td>
<td>$\varepsilon_{cc} = 80K f'_{cc} \times 10^{-6}$</td>
</tr>
<tr>
<td>Park 1982</td>
<td>$f_{cc} = f_{co}(1 + 2 \frac{f_{te}}{f_{co}})$</td>
<td>$\varepsilon_{cc} = \varepsilon_{co}(1 + 2 \frac{f_{te}}{f_{co}})^{0.03}$</td>
</tr>
<tr>
<td>Fafitis and Shah 1985</td>
<td>$f_{cc} = f_{co} + 6.7f_{te} &lt; 0.68$</td>
<td>$\varepsilon_{cc} = \varepsilon_{co}(1 + 3.3 \left(\frac{f_{te}}{f_{co}}\right)^{0.03})$</td>
</tr>
<tr>
<td>Saatcioglu and Razvi 1992</td>
<td>$f_{cc} = f_{co} + 7.6f_{te}$</td>
<td>$\varepsilon_{cc} = \varepsilon_{co}(1 + 3.3 \left(\frac{f_{te}}{f_{co}}\right)^{0.03})$</td>
</tr>
<tr>
<td>Ahmad &amp; Shah 1982</td>
<td>$f_{cc} = f_{co} \left[ 1 + 4.2556 \left(\frac{f_{te}}{f_{co}}\right) \right]$&lt;br&gt;if $\frac{f_{te}}{f_{co}} &lt; 0.68$</td>
<td>$\varepsilon_{cc} = \varepsilon_{co}(1 + 2.5 \left(\frac{f_{te}}{f_{co}}\right))$</td>
</tr>
<tr>
<td>Mander et al 1988</td>
<td>$f_{cc} = f_{co} \left[ 1 + 4.2556 \left(\frac{f_{te}}{f_{co}}\right) \right]$&lt;br&gt;if $\frac{f_{te}}{f_{co}} &gt; 0.68$</td>
<td>$\varepsilon_{cc} = \varepsilon_{co}(1 + 2.5 \left(\frac{f_{te}}{f_{co}}\right))$</td>
</tr>
<tr>
<td>Karabinis 1994</td>
<td>$f_{cc} = f_{co} + 4.269f_{te}^{0.587}$</td>
<td>$\varepsilon_{cc} = \varepsilon_{co}(1 + 2.5 \left(\frac{f_{te}}{f_{co}}\right))$</td>
</tr>
<tr>
<td>Hoshikuma et al. (1997)</td>
<td>$f_{cc} = f_{co}(1 + 7.6f_{te}^{0.25})$</td>
<td>$\varepsilon_{cc} = \varepsilon_{co}(1 + 2.5 \left(\frac{f_{te}}{f_{co}}\right))$</td>
</tr>
<tr>
<td>Cusson &amp; Paultre (1995)</td>
<td>$f_{cc} = f_{co} + 5(\frac{f_{te}}{f_{co}})^{0.7}$</td>
<td>$\varepsilon_{cc} = \varepsilon_{co}(1 + 2.5 \left(\frac{f_{te}}{f_{co}}\right))$</td>
</tr>
<tr>
<td>EC8 2001</td>
<td>$f_{cc} = f_{co}(1 + 1.25(\frac{f_{te}}{f_{co}}))$&lt;br&gt;if $\alpha \geq 0.1$&lt;br&gt;$f_{cc} = f_{co}(1 + 2.5(\frac{f_{te}}{f_{co}}))$&lt;br&gt;if $\alpha \leq 0.1$</td>
<td>$\varepsilon_{cc} = \varepsilon_{co}(1 + 2.5 \left(\frac{f_{te}}{f_{co}}\right))$&lt;br&gt;$\varepsilon_{cc} = \varepsilon_{co}(1.125 + 1.5\alpha \omega_{k})^2$</td>
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### Table 2: Models for ultimate compressive strain of concrete

<table>
<thead>
<tr>
<th>Model</th>
<th>Ultimate strain of confined concrete</th>
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<tr>
<td>CEB model code 90 (1993)</td>
<td>$\varepsilon_{uSC} = \varepsilon_{uSU} + 0.1 \left(\frac{f_{te}}{f_{co}}\right)$</td>
</tr>
<tr>
<td>Cusson and Paultre (1995)</td>
<td>$\varepsilon_{uSC} = \varepsilon_{uSU} + 0.15 \left(\frac{f_{te}}{f_{co}}\right)$</td>
</tr>
<tr>
<td>Seible et al. (1995)</td>
<td>$\varepsilon_{u} = 0.004 + 1\left(\frac{f_{te}}{f_{co}}\right)$&lt;br&gt;metal $\varepsilon_{u} = 0.004 + 2.68 \left(\frac{f_{te}}{f_{co}}\right)$&lt;br&gt;FRP</td>
</tr>
<tr>
<td>Cusson and Paultre (1995)</td>
<td>$\varepsilon_{uSC} = \varepsilon_{uSU} + 0.15 \left(\frac{f_{te}}{f_{co}}\right)$</td>
</tr>
<tr>
<td>Razvi and Saatcioglu (1999)</td>
<td>$\varepsilon_{uSC} = \varepsilon_{uSU} + 260k_{3}f_{te}^{0.587}\left[1 + 0.5k_{3}(k_{4} - 1)\right]$</td>
</tr>
</tbody>
</table>
2. EXPERIMENTAL PROGRAM
This paper presents parts of the results of a comprehensive study on application of the strapping technique for concrete strengthening. The experiments presented in this paper included axial compressive tests on 30 prismatic concrete columns. The axial and lateral stress-strain behaviors of concrete specimens were obtained by simultaneously measuring the force and axial and lateral displacements of specimens. Several parameters were considered for column specimens and retrofitting including compressive strength of concrete, yield strength, ductility, spacing and size of confining strips. A detailed description of the results of strapped concrete columns with various shapes and sizes has been presented by the authors [13], in which several other parameters affecting the response of strapped concrete is discussed in detail. But the main aim of this paper is studying the effect of mechanical properties of confining material on behavior of confined concrete. Three different materials were used for confining concrete columns, including two different types of strips that are called S and T types, as well as CFRP. Prismatic specimens were fabricated in the structure and concrete Laboratory at the building and housing research center. Three different concrete mixtures were used to study the effect of strength of plain concrete on response of confined concrete as listed in Table 3. The first set (B1) was used to study the strapping technique and compare the application of ductile and brittle metal strips (i.e. S and T type strips). The two latter sets (B2 and B3) were especially made for comparing CFRP with the two types of strips for concrete confinement. The corners of prismatic specimens of the second and the third sets (i.e. B2, B3) were rounded with a radius of 2.5 cm, making it suitable for CFRP wrapping. The material used for the concrete specimens included type I portland cement, local sand and gravel. The maximum size of the gravel was 12 mm. No additive was used in any of the mixes. Applied strips had different widths, thicknesses and mechanical behaviors. Standard tensile tests were performed on three samples of each size of these materials and their average mechanical properties are shown in table 4. The moduli of elasticity of strips and FRP were 200 and 220 GPa, respectively. Ultimate strength, elastic modulus, ultimate strain and thickness of each layer of CFRP was 2800 MPa, 220 GPa, 1.55% and 0.176 mm, respectively. A summary of the mechanical properties of these materials are reported in the following table.

<table>
<thead>
<tr>
<th>Table 3: Concrete Mix Designs (per Cubic meter)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Element</strong></td>
</tr>
<tr>
<td>--------------</td>
</tr>
<tr>
<td>Cement</td>
</tr>
<tr>
<td>Water</td>
</tr>
<tr>
<td>Coarse aggregate</td>
</tr>
<tr>
<td>Fine aggregate</td>
</tr>
<tr>
<td>W/C ratio</td>
</tr>
<tr>
<td>Design compressive strength (Mpa)</td>
</tr>
<tr>
<td>Corners radius</td>
</tr>
<tr>
<td>Number of specimens</td>
</tr>
</tbody>
</table>
Table 4: Mechanical Properties of Applied Strips

<table>
<thead>
<tr>
<th>Material</th>
<th>Width (mm)</th>
<th>Thickness (mm)</th>
<th>Yield stress (kg/cm²)</th>
<th>Ultimate stress (kg/cm²)</th>
<th>Ultimate strain</th>
</tr>
</thead>
<tbody>
<tr>
<td>S strip</td>
<td>16</td>
<td>0.5</td>
<td>1033</td>
<td>1033</td>
<td>0.01</td>
</tr>
<tr>
<td>T strip</td>
<td>32</td>
<td>0.8</td>
<td>8746</td>
<td>9975</td>
<td>0.07</td>
</tr>
<tr>
<td>CFRP</td>
<td>---</td>
<td>0.176</td>
<td>28000</td>
<td>28000</td>
<td>0.0155</td>
</tr>
</tbody>
</table>

Figure 1. Test setup and instrumentation a) strapped b) FRP wrapped

3. AXIAL STRESS-STRAIN BEHAVIOR
Axial and lateral strains of column specimens were obtained by measuring specimen deformations by LVDT. In Figure 2, axial and lateral stress-strain behaviors of two specimens of the first set of specimens (B1) are shown. These specimens were confined with brittle S strips at two distinctive spacing values.

Figure 2. Normalized stress-strain of two confined specimens of B1 set strengthened with S strip

The vertical axis shows the provided increase in strength which is obtained by normalizing the measured stress to strength of plain concrete. It can be seen that the amount of volumetric ratio of confinement affects concrete strength and ductility. However because of the low strength of these specimens (in contrast to
other results) the increase in strength is not as much as ductility enhancement. By considering the evolution of the lateral strains, it is obvious that the more confined specimen experiences less dilation.

Similarly, the stress-strain behavior of specimens of B2 and B3 sets that have been confined with one of the three applied confining materials are drawn in Figure 3. By comparing the behavior of specimens confined with brittle strips, ductile strip and CFRP it can be concluded that:

1. For a particular confining material, both strength and ductility of confined concrete increase with increasing the level of confinement.

2. For a similar confinement pattern, ductility of confined concrete is lower for specimens with higher strength concrete.

3. Although all of the three confining materials have similar elastic stiffness and both metal strips have similar strength values, but the form of stress-strain behavior of the triple sets of specimens differ apparently. As a matter of fact, ductility of confining material has dominated the stress-strain curve of confined concrete. As can be seen in the Figures, for a particular confining material, a similar form of stress-strain curve can be observed for various levels of confinement.

4. For a constant level of confinement with several confining materials, the higher the ductility of confining material, the higher the compressive ductility of confined concrete.

Figure 3. axial stress-strain of specimens confined with different materials for concrete specimens of a) B2 and b) B3 sets
It is also observed from these Figures that by using a ductile confining material, a very ductile compressive behavior for concrete can be achieved. This means great ultimate strain, better post-peak behavior and more toughness and capability to absorb energy. It should be noted that these curves correspond to prismatic specimens that traditionally can not be confined effectively and the obtained results for cylindrical specimens show much more ductility. For cylindrical specimens, it was observed that the ductile strip is capable to provide a very ductile behavior even for high strength concretes.[13]

From the above results and the results of other specimens presented by the authors [13], it can be concluded that for a similar confinement level, i.e. equal confinement pressure, ultimate strain and post-peak behavior of confined concrete is mainly dependent to the deformation capacity of the confining material.

4. ANALYSIS OF THE RESULTS

One of the measures for confinement level that has been widely used in the literature is the effective mechanical volumetric ratio of confining material, i.e. \( K_e \cdot \frac{\rho f_y}{f_c} \). This has been also known as the effective confinement index. In this index, \( K_e \) is a ratio between 0 and 1 that takes the ineffectively confined regions between the longitudinal and transverse reinforcements into account. \( \rho \) is the volumetric ratio of the confining material, \( f_y \) and \( f_c \) are yield strength of confinement and strength of plain concrete.

By considering the equilibrium of confined concrete, it can be shown that the effective confinement index equals with \( K_e \cdot \frac{2f_{ly}}{f_c} \), in which \( f_{ly} \) is the lateral confining pressure corresponding to the yield of confinement. This has also been shown by defining the concept of effective yield-based lateral confining pressure, i.e. \( \frac{2f_{ley}}{f_c} \).

By analyzing the experimentally obtained results of this study and also experiments performed by Moghaddam and Samadi 2008, Frangou and Pilakoutas 1995 and also Mortazavi and Pilakoutas 2004, it was observed that the improvement of strength of confined concrete is strongly dependent to the effective confinement index. As can be observed in Figure 4, there is a fair relationship between the strength gain of confined concrete and the effective confinement index. Although, the available models for strength of confined concrete give relatively different formulas the upper and lower bounds of strength improvement ratio, can reasonably be determined with Richart1928 and Ahmad & Shah 1982 models, respectively.
This strong relationship between effective confinement index and strength increase ratio can also be studied in particular for the three sets of specimens of this study which had different plain strengths. Figure 5 shows the strength gain ratio for specimens of this study that were retrofitted by any of the three confining materials of S strips, T strips and CFRP. It is observed that a close relationship exists between the variation of the strength increase ratio and the effective confinement index. This relationship is approximately the same for the three confining materials and three plain strength values.

However, as observed in Figures 3, the gain in ductility of confined concrete is also dependent to the ductility of confining material and can not be described as only a function of confinement index. Therefore, a modification to the effective confinement index was done to take the deformation capacity of confining material into account. An equation with a form similar to that of Seible et al. 1995 as shown in table 2 was selected to predict the ductility gain of confined concrete and the
modified confinement index was defined as the effective confinement index multiplied by the ultimate strain capacity of confining material. In order to study the ductility of confined concrete, ductility measure of CEB model code 90 was applied. A database of results of compressive tests on concrete confined with various materials was collected from the literature. The first data set of the database was the results of axial compressive tests on cylindrical 10*20 and 15*30 and prismatic 10*20 and 15*30 specimens that were confined with the two types of metal strips. These tests were previously conducted by the authors[13]. The second set of data includes test results of cylindrical 10*20 specimens confined with carbon or aramid FRP sheets by Watanabe et al. 1997. The third data set includes results of tests on cylindrical 15*30 specimens confined with CFRP sheets. The ratio between ultimate strains of confined and unconfined concrete specimens are drawn in Figure 8 versus the modified confinement index. As can be seen, the models of Seible et al. 1995 and EC8 did not suitably predict the ductility gain for neither of the steel-confined nor CFRP-confined concrete specimens. An analytical formula was obtained statistically and is as follows.

\[
\varepsilon_{cc}/\varepsilon_{co} = 1+2500 \left( \frac{f_{le}}{f'_{c}} \right), \quad \varepsilon_{ju} \left( \frac{f_{le}}{f'_{c}} \right) < 0.001
\]

\[
\varepsilon_{cc}/\varepsilon_{co} = 3.5+550 \left( \frac{f_{le}}{f'_{c}} \right), \quad \varepsilon_{ju} \left( \frac{f_{le}}{f'_{c}} \right) > 0.001
\]

The obtained formula of equation 1 is compared to the experimental data of the collected database. Figure 6 shows that, although the ductility gain data are more scattered than strength increase ratio values as presented in Figure 4, but equation 1 can give a better estimation of the increase in ductility due to confinement than the previous models. It is important to note that the experimental data of Figure 6 includes specimens with various shapes, sizes and confining materials and therefore the modified confinement index and equation 1 seem to have given good approximation of the ductility gain.

![Figure 6. The Gain in Ultimate Strain for Concrete Confined with Various Materials](image_url)
In order to verify the accuracy of the proposed equation, the experiments of this study were conducted. As mentioned earlier, prismatic 15*30 specimens with three levels of concrete strength were retrofitted with three confining materials, i.e. S type strips, T type strips and CFRP. The ratio of ultimate strain of confined concrete to that of unconfined one are drawn against the proposed ratio of modified confinement index in Figure 7. As can be seen in this Figure, the modified index for confinement and the proposed equation for ductility enhancement ratio due to confinement could give good approximation of the experimentally obtained values of this study. Adequate correlation can be observed between the experimental and analytical values especially when considering numerous differences than exist between the data points including strengths of plain concrete, confinement levels and confining materials.

![Figure 7. Gain in Ultimate Strain Versus Modified Confinement Index](image)

5. CONCLUSION
Compressive tests were conducted on concrete specimens confined with various confining materials. In addition, some other test results were collected from the literature. Based on these results, some conclusions can be made. For all confining materials, by increasing the level of confinement, both strength and ductility of confined concrete increase. The confinement index (defined as the ratio of effective lateral pressure to strength of plain concrete) shows good correlation with the increase of strength of confined concrete for specimens with any size, shape and strength confined with both steel or FRP. Among various parameters, ductility of confining material plays the main role in determining the post-peak response of confined concrete. Then, for a particular confining material, by increasing the level of confinement the strength and ultimate strain of confined concrete are scaled without any significant change in the form of stress-strain curve of concrete. The confinement index alone can not be used for approximating the ultimate strain of confined concrete. The modified confinement index, that was defined and applied in this paper, showed relatively good correlation with the gain in ductility.
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The authors would like to express their sincere thanks to Dr. Parhizkar and all administrative staff of structures laboratory of building and housing research center.

REFERENCES
IMPROVEMENT IN AXIAL STRESS-STRAIN BEHAVIOR OF COLUMNS USING PRE-STRESSED NON-LAMINATED FRP

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\textsuperscript{3}Swiss Federal Laboratories for Materials Testing and Research (Empa)

ABSTRACT
As a new approach to obtain high efficiency from FRP confinement, pre-stressing of FRP composites have been introduced. Pre-stressed FRP straps change confinement situation from passive to active confinement. At active confinement, without considerable axial stress, confining stresses due to pre-stressed FRP composites are present in perimeter of column. In this paper, an innovative method is presented to confine square reinforced concrete columns using non-laminated CFRP straps. Tests were carried out on three medium scale columns (2.0 m high with a cross section of 0.4 m x 0.4 m). Numerical analyses are performed to predict the effects of the pre-stressing at square column. In this paper, a new geometrical model to account for the effects of FRP pre-stressing and shape modifying of the section is proposed which describes better distribution of confinement stresses. Using FE modeling and proposed model, effectiveness of FRP confinement is revised and effectively confined area increased.

Keywords: columns, finite element, confinement, effectiveness, pre-stressing

1. INTRODUCTION
The use of externally applied fiber-reinforced polymer (FRP) has gained popularity for strengthening and repair of concrete structures. Retrofitting concrete structures with fiber reinforced polymer (FRP) has today grown to be a widely used method throughout most parts of the world. The main reason for this is that it is possible to obtain a good strengthening effect with a relatively small work effort. It is also possible to carry out strengthening work without changing the appearance or dimensions of the structure. Nevertheless, when strengthening a structure with external FRP, it is often not possible to make full use of the FRP. The reason for this depends mainly on the fact that a strain distribution exists over the section due to dead load or other loads that cannot be removed during strengthening. This implies that steel yielding in the reinforcement may already be occurring in the service limit state or that compressive failure in the concrete is occurring. By pre-stressing, a higher utilization of the FRP material is made possible \cite{1}. It is extremely important to ensure that, if external pre-stressing is used, the force is properly transferred to the structure. All over the world, there are structures intended for living and transportation. These structures are of varying quality and
function, but they are all aging and deteriorating over time. Of the structures that will be needed 20 years from now, about 85–90% of these are probably already built. Some of these structures will need to be upgraded or replaced, because they are in such poor condition. This is due not only to deterioration processes, but also to errors that may have been made during the design or construction phase so that the structure needs to be strengthened before it can be fully used.

FRP materials were used successfully for confining circular concrete columns. Their effectiveness for confining rectangular columns, however, is still not fully recognized. Rectangular columns behave quite differently from circular columns. Rectangular specimens engage high confining pressure at their corners, but little pressure on their flat sides. The cross section is therefore only partially confined, which results in a smaller increase in compressive strength. This shape-related negative effect, however, can be reduced by rounding off the corners of a rectangular member. The presence of steel ties near the corners limits the possibility of rounding the corner radius in existing square and rectangular columns.

Lower FRP confinement effectiveness results in softening behavior for square and rectangular sections; the high strength of FRP composites is not fully used and the FRP composite ruptures prematurely. Shape modification of section and active confinement due to pre-stressing of FRP straps can reduce the effect of column corners and flat sides, thereby improving the axial strength capacity of FRP-confined square and rectangular concrete columns [2]. Active confinement method causes relative increase in axial strength of circular columns, but limits axial confined strains. In rectangular and square columns pre-stressing of FRP sheets need to shape modification of section. Shape-modification of rectangular sections prevents premature rupture of FRP due to stress concentration in corners.

In this paper, an innovative method is presented to confine square reinforced concrete columns using non-laminated CFRP straps. The CFRP straps can be either unstressed at the beginning or pre-stressed. Tests were carried out on three medium scale columns (2.0 m high with a cross section of 0.4 m x 0.4 m) [3]. The experimental results demonstrate the effectiveness of shape modification with prefabricated parts (mortar filled hoses) which are arranged on the surface of column at the flat sides. In contrast to other researches, presence of longitudinal and transverse steel reinforcement and medium scale columns leads to real assessment behavior of the columns, which is appropriate for existing deficient concrete columns. Models for column confinement are generally based on laboratory tests carried out on small size specimens, generally between 75 and 200 mm (3 and 8 in.) side dimension. For such small size columns, few layers of FRP wrap represent a high volumetric ratio and generally provide a rigid confinement. As a result, a substantial increase in the confined concrete strength can be obtained. In the case of full-scale columns, results are generally extrapolated from such tests and the concept of FRP volumetric ratio is used. The FRP volumetric ratio in real applications is normally less than that of the tested specimens. With the FRP jacket, increasing the column size decreases the FRP volumetric ratio and, therefore, the confined concrete strength also decreases.
2. TESTS SETUP
In this paper an innovative method is presented to confine square reinforced concrete columns using non-laminated CFRP straps. The CFRP straps can be either unstressed at the beginning or pre-stressed. Tests were carried out on three medium scale columns. The first column was the unconfined reference column, the second column was confined with pre-stressed non-laminated CFRP straps and the third column was confined with non pre-stressed non-laminated CFRP straps. Three medium scale reinforced concrete columns were tested under increasing axial loading. The columns had a height of 2 m and a square cross section of 400 x 400 mm (Figure 1). Table 1 summarizes the Non-laminated CFRP straps properties.

<table>
<thead>
<tr>
<th>Thickness [mm]</th>
<th>width [mm]</th>
<th>ultimate strain [-]</th>
<th>E-modulus [GPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.125</td>
<td>30</td>
<td>0.0183</td>
<td>131</td>
</tr>
</tbody>
</table>

3. SPECIMENS' CONSTRUCTION AND PRESTRESSING METHOD
Column (1) was tested without confinement and was used as a reference column. Column (2) was confined with pre-stressed non-laminated CFRP straps and column (3) was confined with non-laminated CFRP straps but without pre-stressing. At each side of the column, two hoses with an outer diameter of 42 mm were fixed over the whole length of the column. This ensured reasonably uniform application of confinement forces on each face of column. The column was confined by 6 layers of CFRP straps with a total thickness of 0.75 mm. After this, a mortar mixture of cement, water and additives, normally used for filling the gaps between concrete and post-tensioned cables, was pumped inside the hoses. For column (3) with non pre-stressed CFRP straps the hoses were filled with mortar before the wrapping of the CFRP layer. In order to achieve the same geometric shape, the hoses were filled between two wooden plates and only then attached to the column. The CFRP straps were then wrapped around the perimeter of the column in the same manner as for column (2). The confined columns (2) and (3) were instrumented for displacement and strain measurements according to Figure 1.

4. OBSERVED BEHAVIOR OF SPECIMENS
FRP-confined concrete compression members exhibit either a hardening or a softening behavior. This separation has been used in the past by other researchers [4]. Effectively confined concrete compression members exhibit hardening behavior. Figure 2 shows the load-deformation behavior of the column (2) with pre-stressed non-laminated CFRP straps which exhibits large deformations with a distinct plateau. Figure 3 shows behavior of Column (3) with non pre-stressed CFRP straps exhibits even larger deformations than column (2). Column (2) and (3) in Figure 2 and 3 compared with behavior of column (1) that predicted by analytical Mander's model.
5. INITIAL POST-TENSIONED TRANSVERSE FIBER STRAIN IN FRP SHELLS

One way of overcoming low utilized strength of FRP composites in rectangular columns is to reduce the lateral strain of the concrete at failure through pre-tensioning of the confinement materials. This method is particularly useful for low modulus materials (like glass) or when relatively low amounts of confinement are applied, such as in full-scale structures [12]. In addition, existing structures may already have very large strains due to existing loading. Adding pre-tensioned FRP wraps can help reduce the stress from the internal links and provides active confinement to the structure in its service condition [13]. Lateral pre-tensioning of composites is not easy to achieve unless the fibres are stretched before the resin hardens. Resin injections under pressure were adopted by some researchers [14, 15] when trying to address the problem of inadequate starter bar lap length in columns. However, the active pressure created by this method (resin injection) is generally quite small in comparison with the passive pressure generated by concrete dilation [16]. Confinement of concrete, using shape modification of square and rectangular compression members using post-tensioned FRP composite shells with expansive cement concrete was investigated by Yan et al.(2006) and
Mortazavi et al (2003). From the experiments carried out (Yan 2005), an empirical relationship was developed as; [17]

\[
\varepsilon_{,ini} = c_1 - 0.00041 \left( \frac{B_j}{D_j} \right) 3 \geq \left( \frac{B_j}{D_j} \right) \geq 1
\]

(6)

The initial post-tensioned transverse fiber strain \( \varepsilon_{,ini} \) depends on the aspect ratio \((B_j/D_j)\) of the prefabricated FRP shell, which is the ratio of the cross section major axis \(B_j\) to the minor axis \(D_j\). For shells with two layers of CFRP composite \(c_1 = 0.0020\), and for shells with six layers of GFRP composite \(c_1 = 0.0025\); the value of constant \(c_1\) is material dependent and decreases with increasing elastic modulus of the FRP composite.

6. FINITE ELEMENT MODELING

Concrete damage plasticity model is used for confined concrete [18]. Suitable elasticity models are used for fiber-reinforced polymers. A nonlinear static analysis was performed where the load was applied incrementally as axial direction displacements uniformly impose at top surface of column. Concrete is modeled using a solid 8-node element (C3D8R) with linear reduced Gauss integration points and enhanced hourglass control. FRP jacket could be modeled as quadrilateral lamina element with either membrane or shell properties. Membrane elements are surface elements that transmit in-plane forces only (no moments), and have no bending stiffness, but shell section has bending stiffness (Figure 4). Therefore for modeling FRP sheets saturated by a resin which has bending stiffness, using shell section is suitable. For unbounded FRP sheets that wrapped around column without resin, no bending stiffness was considered, thus membrane elements are recommended. Therefore in this study, FRP jacket is modeled as quadrilateral lamina element with membrane properties (M3D4), linear reduced Gauss integration points (one point per element), and enhanced hourglass control. Displacement compatibility is considered between concrete and composite material in the lateral direction.

A simplified model has been made with totally 2 tubes that were placed on straight face of square columns (Figure 5). A major output of this model is describing stress distribution in concrete core due to lateral confinement produced by prestressed FRP non-laminated straps. Tubes are shells with low stiffness where
pressure loads are inserted in the internal surface of tubes. Several interactions have been assigned between tubes concrete and tubes–FRP and concrete–FRP. Initial strains due to pre-stressing of FRP straps are 20% of amount strains in EMPA tests. A large stress is inserted onto the concrete surface due to expansion of tubes. As Figure 5 shows at pre-stressing phase, there are no significant confinement stresses of corners, but large lateral stress from below tubes developed and extended to the inside of concrete. As similar to Fig 5, if two other tubes are placed on the other flat sides, similar behavior and lateral stresses is brought about from tubes on the vertical sides.

A simplified numerical model to illustrate the advantages of shape modified section has been built. A triangular part with 20 mm height and equal width to column section width is placed at the flat sides. It is similar to column (3) which has two tubes on the flat sides. As shown in Figure 6, confinement stresses at the section of shape modified column are well distributed than conventional square column (right). At square and rectangular, because of low flexural stiffness of FRP jacket, there is a low amount of confinement stresses. Therefore, some researchers concluded that a parabola area at each of the flat sides, with depth of \( w/4 \), is unconfined area, \( w=a-2r \), and introduces a coefficient that is the ratio effectively confined to the gross area from which actual confinement stresses which assumed uniformity at the whole of the section is obtained. Then confinement ratio \( \frac{f'_{cc}}{f_c} \) is gained. As shown in Figure 6 (left) unconfined length is divided into two parts at each flat side. Therefore, unconfined area is reduced. In Figure 6, the area in white color has confinement stresses that are less than the assumed confinement stresses which are needed to consider an area as confined area.
7. FRP AND STEEL CONFINED AREA

Figure 7 shows the deformed shape of stirrups in conventional column. Lack of constraint at flat sides of section causes large deformation of stirrups towards the outside. If cross ties are implemented at cross section of column, deformations have been limited. But older concrete columns demonstrated deficiency in reinforcement detailing. At tested columns, there are no cross ties. Therefore, deformation of stirrups becomes somewhat as shown in figure 7-a. Using new innovative method tested, deformation of transverse reinforcement has been changed and limited. Indeed, tubes on the flat side act as cross ties. Figure 7-b shows modified deformation of transverse reinforcement in column (2) and (3). According to this modified behavior of transverse reinforcement new effectively confined area is proposed for steel confined area of mentioned columns. Figure (8-a) and (8-b) present effectively confined area by steel transverse reinforcement.

In tube injection phase, considerable stresses are produced between tubes and concrete. In this phase, confining stresses in corners is lower than tube interaction faces. At axial loading phase, due to FRP confining effects on the square and rectangular sections, confining stresses at corners increase. At axial loading phase, increasing of lateral confining stresses at corners is higher than flat sides and tube interaction faces.

Therefore, using this method can modify efficiency of FRP confinement in square and rectangular sections, (Figure 9).
8. PROPOSED GEOMETRICAL AND MECHANICAL MODELS

In this study, a geometrical relationship of the investigated method to produce pre-stressing at FRP and shape modifying of square section column is proposed. According to the new proposed model, confining stresses are developed between pre-stressing devices or shape modifying parts and concrete. Confinement stresses relationships are basically derived from dimensions of the column and diameter of tubes and corner radius, as well as FRP material properties. By virtue of symmetrical condition, one quarter of the section has been considered (Figure 10-a). Following the simple approach of Karam et al. [19] for a generic rectangular cross section column (Figure 10-b), it is proposed that the action of the confining wrap on a square column with pre-stressing tubes on the center of flat sides of column (Figure 10-c) with a side $2a$, and a corner radius $R$, and a tube diameter $D_t$ should be observed. The concrete is assumed to be subjected to uniform confining stresses at its middle sections with $f_a$ acting along the sides. The relationship between $f_a$ and the FRP tensile stress $f_j$ is obtained from statics equilibrium of forces in the in plain $x$ and $y$ directions. At first and before tube expansion, a common form of FRP confinement, FRP has been positioned approximately in contact with the flat sides of column. After mortar injection and due to expansion of tube, non-laminated and un-bonded FRP straps separates from flat sides and takes an angle ($\theta$) with the sides.

![Figure 10](image)

Figure 10. a) Square cross section with expanded tubes on the sides; b) Free-body diagram of FRP wrap; c) Free-body diagram of FRP wrap with pre-stressing effects

In Figure 10-c, status of FRP before and after tube expansion and tensile stress and concentric force after tube expansion has been illustrated. Confining stresses at middle sections and at the corners motivated through the tube expansion. Wrap is assumed to act as a cable around the corner. Assuming no friction, the relationship between $f_j$ and the confining stresses at the corner region is also found from statics. (Eq.7)

$$f_j \times t \cos \theta = \int (f_x \times \cos \alpha) (R \times d\alpha) = Rf$$

$$f_j = \frac{f \times t \cos \theta}{R}$$
$f_i$ and $f_r$ are tensile stress of FRP and confinement stress at corner zone, and $\theta$ is angle between straps and concrete flat side in degree, and $t$, is the thickness of FRP jacket. Using Statics, equilibrium of forces in the horizontal direction results in confining stresses $f_a$ at middle sections. On the contrary with common FRP-confined sections, an additional term appears in equation, $(t_f \times \sin \theta)$. It demonstrates effects of stresses which the tube has produced. Tubes in middle of the side introduce something like line load toward the core of concrete.

$$f_a \times a = t \times f_i \times \cos \theta + t \times f_r \times \sin \theta$$

$$f_a = \frac{f_i \times t (\cos \theta + \sin \theta)}{a} \quad (8)$$

Now tensile stresses of FRP should be expressed as a function of parameters which changes during tube expansion and pre-stressing process. A point at middle of the corner arc is taken as base point from which FRP elongation is measured. At the other side of the selected point there is a similar situation, thus the selected point could be immovable. Using this approach, tensile stress of FRP during pre-stressing has been expressed as a function of tube diameter ($D_t$), side length, corner radius, and $\theta$. Eqs. 9 to 12 describe mentioned approach, and finally confining stresses at corner and middle section have been derived.

$$\Delta l_j = \frac{D \times \sin \theta}{2} \quad l_{\mu} = a + \left( R\theta \frac{\pi}{180} \right)$$

$$\varepsilon_j = \frac{\Delta l_j}{l_{\mu}} = \frac{D \times \sin \theta}{2(a + R\theta \frac{\pi}{180})} \quad (9)$$

Figure 11. geometrical calculation details
Where \( \varepsilon_j \) = strap strain, \( \Delta l_j \) = elongation of straps, \( l_{j0} \) = initial length of strap that was measured from tangent point, \( D_t \) = diameter of tube;

\[
\tan \left( \frac{D_t}{a - R} \right) = \theta \quad (10)
\]

\[
f_c = \frac{E, \varepsilon_t \times \cos \theta}{R} = \frac{E, D_t \times \sin \theta \times t \times \cos \theta}{2R(a + R \theta \pi/180)} \quad (11)
\]

\[
f_c = \frac{tE, D_t \times \sin \theta (\cos \theta + \sin \theta)}{2aR(a + R \theta \pi/180)} \quad (12)
\]

Using this proposed equation, the uniformly distributed confinement stresses is calculated which considers all usable forces and stresses that are there to produce confinement stresses.

As diameter of the tube increases, the angle between FRP and side of column increases. Then FRP at the corner and rounded region separates from concrete core and as illustrated in Figure 12 this results in lessening of direct contact between FRP and concrete. Due to mentioned reason, concentration of stresses at the corner increases. Changing angle between FRP and concrete from 90 degrees to a higher amount is advantageous to transform confinement condition from square to circular column, but lesser contact between FRP and concrete at the corner will reduce effectiveness of this method. At highest possible angle which is 45 degree, contact between FRP and concrete at the corner transforms to a point. The Tangent point which from FRP starts to separate from concrete is shown in Figure 11.

9. CONCLUSIONS

A study is performed on the new FRP confining approach for pre-stressing of FRP straps which proposed and tested by EMPA laboratories. A numerical model using advanced concrete model is performed to study the effects of pre-stressing of FRP non-laminated straps on the confinement stress distribution across the section and the effects of shape modifying. Results have presented that expansion of the tubes imposes lateral stress on the flat sides of rectangular section and this leads to increase in effectiveness of FRP confinement and delays the buckling of longitudinal reinforcement. FRP confinement stress from pre-stressed straps are present before axial loading and results in increasing of initial peak axial stress in comparison with usual using of FRP and a higher strength is achieved in less lateral expansion. A geometrical model for evaluating the confinement stress across the section due to expansion of tubes to produce pre-stressing at FRP is proposed. Using this model and based on developed changes in initial position of the FRP, confinement stress due to pre-stressed FRP straps is calculated.
REFERENCES
APPLICATION OF FIBER REINFORCED PLASTICS FOR CONCRETE T-BEAM BRIDGE STRENGTHENING

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ABSTRACT
A strengthening method using fiber reinforced plastics (FRP) has been widely applied for deteriorated reinforced concrete bridges. Advantages of this method may include that the strengthened structures do not increase dead weight and that no corrosion is concerned. From various experimental studies, behavior characteristics of the FRP strengthened structural members have been generally well evaluated. The majority of those studies, however, have been performed on laboratory sized structural members rather than actual full scale bridges. The study herein used a full scale actual deteriorated bridge to evaluate the strengthening effects with three different FRP materials, carbon fiber sheet, glass fiber reinforced plastics and aramid fiber sheet. In the field load tests, concrete weight blocks were used as a loading system instead of commonly used a live-truck load or hydraulic jacking force. The strengthening was designed as specified in ACI 440.1R which is based on the ultimate strength design concept. From the measured behaviors, it was confirmed that the strengthening using FRP materials successfully improved the flexural capacity of the aged and deteriorated concrete bridge. The strengthened girders behaved linearly up to the design moment even though the applied stress distribution mechanisms were different depending on the FRP materials. Therefore, it is concluded that the strengthening design method specified in ACI 440.1R can be successfully used for actual full scale bridges strengthened with external FRP bonding.

Keywords: strengthening, CFS, FRP, T-beam bridge, USD

1. INTRODUCTION
For deteriorated concrete bridges, an external strengthening method with fiber reinforced plastics (FRP) has been widely applied with some advantages. Among others, the method does not increase dead weight and no corrosion consideration is needed. Therefore, it could be a good alternative method for the ordinary steel plate strengthening method. In the early stage of the application of FRPs, most research studies were to evaluate the material properties of FRPs. Since late 90s, the strengthening in various forms of FRPs has been rigorously studied and its effectiveness was successfully proven. In the last decade, engineers and researchers have been intended to develop design and construction specifications for the
practical applications of the FRP strengthening in the field [1, 2, 3]. Through a variety of experimental studies, structural behaviors of the FRP strengthened bridges have been generally well evaluated, and the problems of stress concentration at the end section of the bonded FRP have been improved [4, 5, 6, 7]. Most of these studies, however, have been performed on scaled down in-laboratory specimens rather than actual full scale bridges. There must be limitations in representing deteriorating conditions in the field, such as the difference in environmental conditions, dependency on neighboring structures, and girder-slab composite behavior [8]. In addition, depending on field construction conditions and workmanship, the accurate installation of the FRP strengthening materials may not be expected as desired. Therefore, in order for the strengthening method using external FRP bonding to be practically applied to deteriorated bridges, the performance of the strengthening should better be evaluated using a full-scaled field bridge. Results or data from the evaluation using the actual bridge might also be used as basic information for the specifications or provisions for strengthening design and installation.

The study herein used a full-scaled actual deteriorated bridge to evaluate the strengthening effects from three different FRP materials, carbon fiber sheet (CFS), glass fiber reinforced plastics (GFRP), and aramid fiber sheet (AFS). Another interesting feature of this study is the use of concrete weight blocks as a loading system instead of a truck load or a hydraulic jacking force, which are commonly used in the performance evaluation of bridges. However, some shortcomings of the later methods are: The field loading test using the truck load may provide information only up to a limited elastic range. The hydraulic jacking method costs relatively too high, and sometimes is not feasible to install reaction frames [9].

In addition to these reasons, the target bridge was no longer in-service so that the loading method using the concrete weight blocks was thought very appropriate and challenging for this study.

2. EXPERIMENTAL STUDY
2.1. Bridge Description
The target concrete T-beam bridge originally consisted of two continuous three-spans, but south two spans were replaced with steel I girder after severe damage from accident impact load. When tested, the bridge consisted of one continuous three-span T girder, one simple span T girder and two of simple spans of steel I girder. It should be noted that the concrete bridge age is 80 years.

Based on visual inspection report, the selected three spans were repaired using an epoxy injection and mortar prior to the strengthening. This repair process was to provide a theoretically identical level of deterioration condition. Compressive strengths of the girders were measured using the Schmidt Hammer and appeared to be from 16.9 to 18.5MPa. based on non-destructive tests and concrete power peeling off, it was revealed that the span 1 and 3 had 25mm diameter rebars and the span 2 20mm diameter rebars as shown in figure 1. Tensile strength of the steel
rebars was assumed to be 300MPa.

2.2. Strengthening Design
In this experimental study, three different FRP strengthening materials were considered and they were CFS, GFRP and AFS. The strengthening was designed according to ACI 440.1R [10] which is based on the ultimate strength design (USD) method that provide 10% higher flexural capacity than the repaired condition. An equation is given below to compute the nominal flexural capacity of the FRP strengthened beams.

\[
M_n = A_s f_y \left( d - \frac{\beta_1 c}{2} \right) + \psi_{frp} A_{frp} f_{frp} \left( d - \frac{\beta_1 c}{2} \right)
\]

where \(M_n\) is a nominal moment, \(A_s\) and \(A_{frp}\) are cross sectional areas of rebar and FRP strengthening material, \(f_y\) and \(f_{frp}\) are yielding and ultimate strength of rebar and FRP strengthening material, \(d\) is an effective depth of beam, \(c\) is a distance from extreme compression fiber to the neutral axis, \(\beta_1\) is taken as 0.85 for the given concrete strength and, \(\psi_{frp} (=0.85)\) is a additional reduction factor to account for more brittleness of the FRP reinforcement. The equivalent steel ratios of the FRP strengthened girders were less than the balance ratio that was intended not to cause an unexpected brittle failure. The span 1 was strengthened with CFS, the span 2 with GFRP and the span 3 with AFS as shown in Figure 2. The Properties of the strengthening materials provided from the manufacturers are given in Table 1. Table 1 also details the strengthening details and the design flexural capacity.

2.3. Field Load Testing
For the three spans strengthened with each different FRP, loading test were conducted using the concrete weight blocks which had two different weights, 12.8KN and 25.5KN. The concrete weight blocks were loaded until the weight...
reached 95% of the design flexural capacity for the safety concerns considering the age of the target bridge. Before the weight blocks were loaded by a crane, an accurate weight of each block was measured. When each block was loaded, vertical mid-span deflections and strains of the rebar and the strengthening materials were measured by LVDTs and strain gages.

![Figure 2. Epoxy repair and strengthening views](image)

### Table 1: Properties of strengthening materials and strengthening design capacity

<table>
<thead>
<tr>
<th>Test Span</th>
<th>Material</th>
<th>Ultimate Strength (MPa)</th>
<th>Ultimate Strain</th>
<th>Thickness (mm)</th>
<th>Width (cm)</th>
<th>No. of Layer (ply)</th>
<th>$M_n - M_d$ (KN.m) before</th>
<th>$M_n - M_d$ (KN.m) after</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>AFS</td>
<td>2,942</td>
<td>0.026</td>
<td>0.193</td>
<td>35</td>
<td>1</td>
<td>2,706</td>
<td>3,119</td>
</tr>
<tr>
<td>2</td>
<td>GFRP</td>
<td>441</td>
<td>0.020</td>
<td>1.300</td>
<td>35</td>
<td>2</td>
<td>1,363</td>
<td>1,783</td>
</tr>
<tr>
<td>3</td>
<td>CFS</td>
<td>3,481</td>
<td>0.015</td>
<td>0.110</td>
<td>35</td>
<td>1</td>
<td>2,706</td>
<td>3,285</td>
</tr>
</tbody>
</table>

### 3. DISCUSSION OF TEST RESULTS

#### 3.1. Span 2: GFRP Strengthened

The maximum applied load was 535KN and 677KN for the non-strengthened and the GFRP strengthened girders, respectively. As the load increased, a certain amount of the flexural cracks occurred and then the diagonal shear cracks formed. In the girder 3, some of the diagonal shear cracks occurred before the flexural cracks. This might be because of the degradation of stirrups due to corrosion and an incomplete epoxy repair for the inside of the girder. After the strengthening, the crack patterns were similar but the numbers and the widths of the cracks smaller by visual inspection. No delamination of the strengthening material, GFRP, was observed at the bonded interface due to the attached U-shaped strengthening at the both ends.

For the non-strengthened girder, initial cracks developed at about 100KN of loading with a slight decrease of stiffness. It was also observed that there was a secondary stiffness reduction at about 480KN. This might be, even though the yielding strain of the rebar was assumed as 0.0015, because the rebar in the bridges already experienced a significant amount of plastic deformation accumulated during more than 60 years of the service period and also because of the steel grade.
for the old bridge. For the GFRP strengthened girder, the deflection and rebar strain increased momentarily to some degree at 640KN.

3.2. Span 1 and 3: CFS and AFS Strengthened
The spans 1 and 3 were also repaired before the loading test with an epoxy and mortar, and then strengthened with CFS and AFS for the span 1 and 3, respectively. During the loading blocks were loaded, both the CFS and AFS strengthened girders developed some bending cracks followed by the shear cracks at the bottom with no rupture or delamination of the strengthening material. The AFS strengthened girder developed initial flexural cracks at about 245KN at girder 1 and 2, and then diagonal shear cracks. For the CFS strengthened girders more cracks developed but the widths were relatively smaller. The load-deflection behaviors were linear but as the loading approached to the maximum there was slight increase in deflection for both cases. Once the loading was removed the girders exhibited residual strain greater than 200µ, which was similar to the span 2. In the CFS strengthened girder, above 500KN, the applied load seems to be suspended more by CFS rather the rebar. In the AFS strengthened girder, the applied load is carried more by the rebar. In the experiments, small scale localized ruptures occurred on the surface of AFS. None of the CFS and AFS strengthened girders examined any significant reduction of load-carrying capacity during the testing and the measured load deflection behaviours (figure 3) seems as if the girders are within the elastic ranges, even though the stress distribution mechanisms are different for both strengthening materials. These observations imply that the strengthening design method used in this study appropriately takes account of the material properties of CFS and AFS. Therefore, it can be concluded that the strengthening design method specified in ACI 440.1R [10], which is based on the USD concept, can be successfully used for actual full scale bridges which are strengthened with external FRP bonding.

4. CONCLUSIONS
This study conducted field load tests on an existing full scale reinforced concrete T-typed girder bridge to evaluate the performance of structural strengthening. In the field load tests, the loading system used concrete weight blocks rather than using a live-truck load or hydraulic jacking force. The method was found to be safe and efficient especially when failure behavior over elastic limit is interested. The strengthening used three different types of FRP materials, AFS, GFRP, and CFS, which were bonded on the bottom of the girders to enhance the flexural load carrying
capacity. The strengthening was designed as specified in ACI 440.1R which is based on USD concept. It was confirmed that AFS, GFRP and CFS strengthenings successfully improved the flexural capacity of the aged and deteriorated concrete bridge. The strengthened girders behaved linearly up to the design moment even though the applied stress distribution mechanisms were different depending on the FRP materials. Therefore, it is concluded that the strengthening design method specified in ACI 440.1R [10] can be successfully used for actual full scale bridges which are strengthened with external FRP bonding.

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DURAPGULF, A PROBABILISTIC APPROACH FOR DURABILITY DESIGN OF RC STRUCTURES IN THE PERSIAN GULF USING

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ABSTRACT
A probabilistic approach in durability design of reinforced concrete structures has been studied using DuraPGulf model. DuraPGulf is a service life design model, the first version of which provides a realistic prediction of corrosion initiation for RC structures in Persian Gulf region. Output parameters are interpolated using a complete database of conducted experiments in this region. Although relevant data is still lacking, this approach has been successfully applied to a concrete structure in Persian Gulf environment. In order to facilitate the probability-based durability analysis, simple analysis has been developed, where the probabilistic approach is based on a Monte Carlo simulation. A comparative study of deterministic and probabilistic approach has been carried out using the data available from durability assessment of a jetty in Persian Gulf. In particular, probability based design seems to provide more realistic results than deterministic durability design analysis.

1. INTRODUCTION
Reinforced concrete has proved to be a reliable structural material with good durability performance when used properly. It has been one of the most used building materials in the last decades. However, there are many structures which show early, premature deterioration, and sometimes failure, namely those exposed to aggressive environments (Vaysburd & Emmons 2004). The Persian Gulf environment has a long record of stigma for its harsh climate, desert features and saline waters, that increases the chloride penetration and consequently chloride induced reinforcement corrosion rates (Haque et al. 2007). In many cases in the Persian Gulf, even structures which have been designed and constructed in compliance with Iranian code for durability of concrete structures in Persian Gulf region (BHRC-PNS428 2005), express corrosion problems in early age of their service life. In order to solve this problem, besides changing the code requirements in respect of durability, one should represent methods of durability design so that to find the most economical solution according to the desired service life for reinforced concrete structures (Ghalibafian et al. 2003). Since all parameters both for concrete durability and environmental exposure typically show a high scatter, a probability-based approach provides a very powerful basis for
durability analysis (Gehlen & Schiessl 1999, Bentz 2003). This approach is primarily applied in order to obtain more controlled durability and long-term performance of new concrete structures.

During the past decades, many physical and mathematical models have been introduced to calculate the chloride diffusion parameters into concrete and to estimate the time to corrosion initiation. DuraPGulf (Chini et al. 2004, Ghods et al. 2007), like many other programs, was deterministic in its operation, meaning that it will produce only one predicted time to initiation of corrosion for one set of input parameters. This single output contrasts with the well-known fact that concrete structures are quite variable in properties both throughout the structure and in terms of quality of construction and materials from one project to another. It would be useful if programs like DuraPGulf were able to predict a range of expected times to initiate corrosion rather than a single value to allow owners in a better risk management (Bentz 2003).

This paper describes the DuraPGulf model structure. The demonstration of this practical application is also explained through an example in this paper. Results obtained from the model have been verified with the data available from durability assessment of a jetty in Persian Gulf region.

2. EXPERIMENTAL PROGRAM
Concerning the fact that there was few data available for concrete durability studies specially regarding the chloride diffusion in Persian Gulf region, a complete set of field experiments were conducted in order to investigate the effect of different parameters on chloride diffusion such as water to cement ratio, silica fume content, curing condition, exposure condition, environment temperature and surface coating. A detailed review regarding this experiment can be found elsewhere (Chini et al. 2004, Ghods et al. 2007). In this project, 120 prism specimens measuring 150×150×600 mm were exposed to marine environment of Bandar-Abbas city. Sampling of the specimens for chloride diffusion has been carried out at the ages of 3,9 and 36 months. Figure 1 shows the concrete prism specimens for the long-term durability studies.

By curve fitting of chloride profiles of each specimen to Fick’s second law (Crank
1975), data for diffusion coefficient and surface chloride content were calculated. Data have been used in order to develop the DuraPGulf database.

3. NUMERICAL MODELING
DuraPGulf uses advanced mathematical concepts for analyzing input data in order to predict Diffusion Coefficient ($D_c$) and Surface Chloride Content ($C_s$) values for new cases. Moving Least Squares method (MLS) is used for data regression. Accordingly, for each set of new input data in the n-dimensional space of primary data, a regression is conducted so that the nearby primary data have the highest effect on the final output (i.e. diffusion coefficient and surface chloride content). According to the Figure 2, in the Moving Least Squares approach the weighting function $F$ is defined in shape and size, and is translated over the domain so that it takes the maximum value over the point $k$ identified by the coordinate $X_k$ where the unknown function $\hat{u}$ is to be evaluated. The weight factor in this regard, which influences the effect width, can be calibrated after trial and error for new set of experiments (Lancaster & Salkauskas 1981).

![Figure 2. Schematic illustration of Moving Least Squares (MLS) method.](image)

Finite Difference Method (FDM) is used to solve differential equation of the Fick’s second law of diffusion over time:

$$\frac{\delta C}{\delta t} = D_c \cdot \frac{\delta^2 C}{\delta X^2}$$

(1)

Considering the following boundary conditions:

$$C(X > 0, t = 0) = 0$$
$$C(X = 0, t > 0) = C_s$$
$$C(X = \infty, t > 0) = 0$$

The solution for this differential equation is:
Where \( D_c \) is the chloride diffusion coefficient, \( C_s \) is the equilibrium chloride concentration on concrete surface and \( C \) is the chloride content at the depth of \( X \) from the surface at the time. The time, at which chloride content on the reinforcement surface reaches the chloride threshold value, is considered as the corrosion initiation time. In the DuraPGulf software, the percentage of chloride threshold is depended on mixture proportion and thickness of concrete cover. The time dependence of the diffusion coefficient is normally expressed as (Thomas & Bentz 2001):

\[
D(t) = D_0 \frac{t^\alpha}{t_0}
\]  

Where \( D_0 \) is the diffusion coefficient at a given time \( t_0 \) and the exponent \( \alpha \) represents the time dependence of the diffusion coefficient or the increased ability of the concrete to resist chloride penetration over time.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Average</th>
<th>Coefficient of Variation</th>
<th>Standard Deviation</th>
<th>Distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chloride diffusivity (m2/s. 10-12)</td>
<td>0.56</td>
<td>0.25</td>
<td>0.140</td>
<td>Normal</td>
</tr>
<tr>
<td>Surface chloride content (%weight of concrete)</td>
<td>0.80</td>
<td>0.30</td>
<td>0.24</td>
<td>Normal</td>
</tr>
<tr>
<td>Critical chloride content (%weight of concrete)</td>
<td>0.13</td>
<td>0.20</td>
<td>0.027</td>
<td>Normal</td>
</tr>
<tr>
<td>Concrete cover (mm)</td>
<td>75</td>
<td>0.10</td>
<td>7.5</td>
<td>Normal</td>
</tr>
<tr>
<td>Aging factor</td>
<td>0.31</td>
<td>0.25</td>
<td>0.078</td>
<td>Normal</td>
</tr>
</tbody>
</table>

Employing Ahrenius equation, the effect of temperature is considered on the diffusion coefficient for different months during the year (Page et al. 1981):

\[
\frac{D_T}{D_0} = \exp \left( \frac{U}{R} \times \frac{1}{T_0} - \frac{1}{T} \right)
\]  

Where \( T \) and \( T_0 \) are temperature in Kelvin degree, \( R \) is the gas general constant value and \( U \) is the activation energy of the diffusion process. A special value of 2948 for \( U/R \) was suggested in this study for DuraPGulf based on the experiments at the Construction Materials Institute (Chini et al. 2004). Similar to the temperature, the effect of humidity on the diffusion coefficient is considered through the model proposed (Bazant & Najar 1972):

\[
\frac{D_{HI}}{D_0} = 1 + 256(1 - \frac{h_4}{100})^{-1}
\]
Where $h$ is the humidity of environment ($\%$).

4. PROBABILISTIC APPROACH
The probabilistic approach is based on the Monte Carlo Method, which can be briefly described as a statistical simulation method, where sequences of random numbers are applied to perform the simulation (Gehlen & Schiessl 1999, Bentz 2003). In the present application of the simulation, the physical process is simulated directly by use of the modified Fick’s Second Law of Diffusion for describing the transport process. The only requirement is that all the input parameters to the equation be described by a probability density function. Once the probability density functions of the various durability parameters of the system are known, the probability of failure is based on the evaluation of the limit state function for a large number of trials. Figure 3 shows a normal curve divided into eight sections, each with a probability of 0.125 (Bentz 2003). At about the centroid of each region is a discrete point that represents the characteristic value for that region. The locations of these points are positive and negative 1.65, 0.89, 0.47 and 0.155 standard deviations from mean. By varying each set of the input parameters through all eight of these calculation points, it becomes possible to know exactly the number of iterations necessary to fully cover the input domain and produce a reasonable estimate of the solution. The accuracy of the Monte Carlo Method depends mainly on the number of trials undertaken and the method is easy to implement, a simulation based on this method appears to be both simple and intuitive.

![Figure 3. Division of normal curve into discrete regions (Bentz 2003).](image)

5. PROGRAMMING
DuraP Gulf software has been developed based on data regression and analyses according to the mathematical methods mentioned earlier. The program uses a FDM kernel and graphic user interface (GUI) provided by VISUAL BASIC programming.

6. EXAMPLE
A typical reinforced concrete pier of a jetty in Bandar-Emam port is subjected to the tidal zone exposure of the Gulf. The thickness of the concrete cover is assumed
to be 75 mm. Temperature diagram of Bandar-Emam is also shown in Figure 4.
Concrete has been cured for 3 days after demolding in July. The concrete mixes were
developed at the water to cementitious ratios of 0.38, cementitious materials content of
approximately 450 kg/m³ and 6.5% silica fume. These parameters were used as input to the
program. Running DuraPGulf, the calculated profile of chloride concentration versus depths
for a given time of 27 months and the result of the sampling are shown in Figure 5. The
corrosion initiation time is calculated with the deterministic approach to be 128 years, which
is very high.
In order to analyze the probability levels of the above combinations of concrete
quality and concrete cover, initial probability analysis with input parameters as shown in
table 1 were carried out. In this table, the surface chloride content of 0.8% by weight of
concrete, which reflects the exposure conditions in tidal zone, was adopted. Based on a
complete database of experiments, a threshold chloride content of 0.13% (Frederiksen, J.M.,
2000) by weight of concrete and aging factor of 0.31 was assigned. Using Monte Carlo
simulation, the probability of steel corrosion and development of further chloride
penetration is illustrated in Figure 6. In most codes for reliability of structures, an upper
level of 10% for probability of failure is normally accepted (NS 3490, 2004). As can be
seen from Figure 6, a 10% level for the risk of steel corrosion will result in a service period
of 32 years and for a service period of 128 years, the level of risk would be 65%.

Figure 4. Annual temperature diagram of Bandar-Emam

Figure 5. Comparison of the results of experiment with the results predicted by DuraPGulf model
7. CONCLUSIONS
Regarding results and experiments following conclusions can be drawn:
1. Durability design of RC structures has received a great concern in the recent decades;
2. In Iran, the need for a model for service life design of concrete structures in Persian Gulf region is highly necessary;
3. The predicted chloride profile by DuraPGulf model is generally in agreement with the experimental results.
4. Probabilistic durability model provides more realistic solutions than deterministic durability design.

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CD05
Durability and Evaluation
MODELING THE CORROSION OF REINFORCED CONCRETE STRUCTURES BASED ON THE FUZZY SYSTEMS

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ABSTRACT

Reinforced concrete structures are susceptible to be deteriorated in harsh environments. One of the major causes of deterioration of RC structures in these environments is the corrosion of their reinforcements. Generally the corrosion of reinforcements involves some uncertainties. In brief these uncertainties can be categorized into classes: 1) random nature of corrosion process and 2) linguistic terms of construction process. The first one is successfully handled by probabilistic methods, but to deal with the linguistic ambiguities, the fuzzy systems are general form of confrontation. One main problem is to obtain the fuzzy membership functions of corrosion process; in particular, the corrosion initiation and propagating time periods. In this paper we developed an algorithm to extract the fuzzy membership functions from the available stochastic information. In this regard, an integrated system proposed to convert the probabilistic information into the corresponding fuzzy sets. In this process, a genetic algorithm optimization utilized to gain the standard Triangular and Trapezoidal fuzzy sets, by means of minimizing the distance of fuzzy sets from the corresponding normalized probability density functions.

Keywords: reinforced concrete, corrosion, service life, probability, fuzzy variables, genetic algorithm

1. INTRODUCTION

Reinforced concrete structures are susceptible to be deteriorated in harsh environments. There may be many detrimental agents at the environment of structures, that could cause some serious damages on both reinforcement and concrete. ACT's guide to durable concrete [1] recognized five general classes of concrete deterioration as: a) Freezing and thawing, b) Aggressive chemical exposure, c) Abrasion, d) Corrosion of steel and other materials embedded in concrete, and e) Chemical reactions with aggregates. Among these deteriorative mechanisms, the corrosion of reinforcement is very important for the structures located in the harsh environment of the southern parts of Iran, especially in the region of the Persian Gulf and Oman Sea. In these regions, high humidity along with high temperature, and availability of detrimental agents
like chloride ions, are preparing a suitable environment for premature deterioration of RC infrastructures which has been built there. In this view, one of the important issues that should be considered is the corrosion of the built RC structures at these regions. Up to now well-established models have been proposed for estimating of the rate of chloride ingress and corrosion rate. Unfortunately the most of these models are based on deterministic variables that can not handle the associated uncertainties. The stochastic models are also established to deal with some uncertainties regarding the randomness nature of the corrosion. While the uncertainties regarding the linguistic terms are usually dismissed and can not be handled by the stochastic methods. The fuzzy systems are famous for their capabilities to consider the verbal rules and regulations. Therefore in this paper we try to extend the appropriate stochastic modes to the fuzzy models, to be used in the framework of fuzzy systems that could be simultaneously handled by fuzzy-logic operations and knowledge processing techniques.

2. CONCRETE AND DURABILITY
Over the millennia, concrete prepared by the Romans using lime, pozzolana and aggregates has survived the elements, giving proof of its durability [2]. Prestigious concrete works have been handed down to us: buildings such as the Pantheon in Rome, whose current structure was completed in 125 A.D. and also structures in marine environments have survived for over two thousand years. This provides a clear demonstration that concrete can be as durable as natural stone, provided that specific causes of degradation, such as acids or sulphates, freeze-thaw cycles, or reactive aggregates, are not present [2]. Today, thanks to progress made over the past few decades in the chemistry of cement and in the technology of concrete, even these causes of deterioration can be fought effectively. With an appropriate choice of materials and careful, adequately controlled preparation and placement of the mixture, it is possible to obtain concrete structures which will last in time, under a wide variety of operative conditions. The case of reinforced concrete is somewhat different. These structures are not eternal, or nearly eternal, as was generally supposed up until the 1970s [2]. Instead, their service life is limited precisely because of the corrosion of reinforcement. Actually, concrete provides the ideal environment for protecting embedded steel because of its alkalinity. If the design of a structure, choice of materials, composition of the mixture, and placement, compaction and curing are carried out in compliance with current standards, then concrete is, under most environmental conditions, capable of providing protection beyond the 50 years typical of the required service life of many ordinary structures, at least in temperate regions. In fact, cases of corrosion that have been identified in numerous structures within periods much shorter than those just mentioned, can almost always be traced to a failure to comply to current standards or to trivial errors in manufacturing of the concrete. However, under environmental conditions of high aggressiveness (generally relate to the presence of chlorides), even concrete which has been properly prepared and placed may lose its protective properties and allow corrosion of reinforcement long before 50 years have elapsed, sometimes resulting in very serious consequences [2].
3. CORROSION OF REINFORCED CONCRETE STRUCTURES

As emphasized in previous section, the problem of corrosion in reinforced concrete structures is a very crucial matter and must be given special considerations. The durability of reinforced concrete structures is impacted by the chloride penetration and susceptibility of the reinforcement to chloride-induced corrosion, when exposed to chloride-laden environment or deicing salts. Once the chloride content at the reinforcement reaches a threshold value and enough oxygen and moisture are presented, the reinforcement corrosion will be initiated. Corrosion products then accumulate in the concrete–steel interface transition zone (ITZ), generate expansive pressure on the surrounding concrete, and cause crack initiation and propagation [3]. In this paper we are aimed to study the corrosion initiation and corrosion-induced cracking periods in the fuzzy sets vision. To develop such models it is needed to find a way to establish fuzzy model and their sets configurations. To this aim, we developed our fuzzy models based on the statistical information that reflects the real distributions of the basic variables which is gained from the available information in the literature.

4. TRANSFORMING THE PROBABILISTIC INFORMATION INTO FUZZY SETS

In many engineering problems which involve a number of variables, sufficient information may be available to model some of the variables using probability distribution functions (pdf), while the other variables are treated as fuzzy. But, in some cases, such as steel corrosion analysis, it may be more realistic to carry out the decision analysis in the framework of fuzzy set theory. In fact, it is easy to deal with fuzzy sets than the probability distributions. So, obtaining the corresponding fuzzy sets from its known probability distribution could be very crucial issue in the framework of fuzzy experts systems and some further decision making processes.

4.1. Fuzzy Probability

A usual method, used for converting a probability distribution into a fuzzy set, is by dividing the pdf of the distribution by the peak value of the pdf [4]. While this method is simple, as pointed out by Dubois and Prade [5] there is a need to explicitly check the resulting fuzzy set for possibility/probability consistency principle. Using this method, a probability distribution with given pdf, \( p(x) \), can be converted into a fuzzy set by

\[
f_p(x) = \frac{p(x)}{\sup_{x\in\mathbb{R}}(p(x))}
\]

where \( f_p(x) \) is the membership function of the fuzzy set. The resulting fuzzy set is of the same form as that of the probability distribution. But, for typical engineering applications involving several uncertain variables, fuzzy sets with triangular or trapezoidal form are preferred due to the computational simplicity.
4.2. Extracting the Standard Fuzzy Memberships

Klir and Yuan [6] proposed the method of least-square curve fitting for constructing a membership function from samples of membership grades for some elements in the universal set $X$. Given the sample data $(x_i, a_i)$, $i = 1, \ldots, n$, where $a_i$ is the grade of membership of $x_i$ in fuzzy set $A$, and a suitable class of functions $f(x; \alpha, \beta, \ldots)$ where $\alpha, \beta, \ldots$ are parameters whose values distinguish functions in the class from one another, the method of least-square curve fitting selects that function $f(x; \alpha_0, \beta_0, \ldots)$ from the class for which the following norm reaches its minimum (discrete form).

$$E = \sum_{i=1}^{n} [f(x_i; \alpha, \beta, \ldots) - f_p(x_i)]^2$$  \hspace{1cm} (2)

This method can be used to convert the probabilistic fuzzy set into the equivalent triangular or trapezoidal fuzzy set. The equivalent fuzzy set, $f(x; \alpha_0, \beta_0, \ldots)$, is the one which minimizes the function $F$, given by (continuous form)

$$F = \int_{X} [f(x; \alpha, \beta, \ldots) - f_p(x)]^2 \, dx$$  \hspace{1cm} (3)

where $f_p(x)$ is the membership function of the probabilistic fuzzy set given by Eq. (1). For a triangular fuzzy set, $f(x; \alpha, \beta, \ldots)$ is given by (Figure 1(a))

$$f(x; \alpha, \beta, \gamma) = \begin{cases} 0 & x < \alpha \\ \frac{\alpha - x}{\alpha - \beta} & \alpha \leq x \leq \beta \\ \frac{\gamma - x}{\gamma - \beta} & \beta < x \leq \gamma \\ 0 & x > \gamma \end{cases}$$  \hspace{1cm} (4)

For a trapezoidal fuzzy set, $f(x; \alpha, \beta, \ldots)$ is given by (Figure 1(b))

$$f(x; \alpha, \beta, \gamma, \delta) = \begin{cases} 0 & x < \alpha \\ \frac{\alpha - x}{\alpha - \beta} & \alpha \leq x \leq \beta \\ \frac{\delta - x}{\delta - \gamma} & \beta < x < \gamma \\ 1 & \gamma \leq x \leq \delta \\ 0 & x > \delta \end{cases}$$  \hspace{1cm} (5)
In this paper, genetic algorithm would be used to optimization process to find the standard fuzzy membership functions of corrosion initiation and corrosion propagation periods.

5. FUZZY MODEL FOR CORROSION INITIATION PERIOD
The steel remains passive in the concrete due to the alkalinity of the environment. Once the chloride ions attack the reinforcement, the passive oxide layer on the steel surface would be broken. This stage, known as the corrosion initiation time. The ingress of chloride ions into the concrete media is directly depends on the permeability of the concrete that is affected by the quality of the concrete practice. If we suppose that there is no initial chloride ion in the concrete \( C_i = 0 \), then the Fick's second law of diffusion could be applicable in the form of Eq. 6.

\[
\hat{T}_i = \frac{\bar{C}^2}{4\bar{D}_c} \left[ \text{erf}^{-1}\left( \frac{\bar{C}_s - \bar{C}_{cr}}{\bar{C}_s} \right) \right]
\]  

where \( \bar{C} \) is the random concrete cover depth, \( \bar{D}_c \) is the random diffusion coefficient of concrete, \( \bar{C}_s \) is the surface chloride concentration and \( \bar{C}_{cr} \) is the random critical chloride concentration. This is a probability or stochastic form for the corrosion initiation time. Note that in this equation, \( \text{erf} \) is error function and equals to \( \left( \frac{2}{\sqrt{\pi}} \right) \int_0^x e^{-t^2} dt \). Now if we exert section 4 method to convert the random representation of parameters into the fuzzy ones, we can reach the fuzzy model for corrosion initiation time as follow:

\[
\hat{T}_i = \frac{\hat{C}^2}{4\hat{D}_c} \left[ \text{erf}^{-1}\left( \frac{\hat{C}_s - \hat{C}_{cr}}{\hat{C}_s} \right) \right]
\]  

where \( \hat{C} \) is the fuzzy concrete cover depth, \( \hat{D}_c \) is the fuzzy diffusion coefficient of concrete, \( \hat{C}_s \) is the fuzzy surface chloride concentration and \( \hat{C}_{cr} \) is the fuzzy
critical chloride concentration.

### 5.1. Probability Distribution for Corrosion Initiation Time

The probability distribution for the corrosion initiation could be gained based on the simple Monte Carlo Simulation (MCS) method. The basic variables for MCS of corrosion initiation are summarized in Table 1.

Table 2, summarizes ANOV results for lognormal distribution fitted on the MCS of Eq. 6. According to the results of this table, the corrosion initiation lognormal pdf is:

$$T_i \approx \frac{1}{1.478t} e^{-\frac{(\ln t - 3.89)^2}{0.6956}}$$

(8)

The supreme value for $T_i$ is 0.0163. By dividing the Eq. (8) to the supreme value, the fuzzy model for corrosion initiation can be gained as follows:

$$\hat{T}_i = \frac{1}{0.0241} e^{-\frac{(\ln t - 3.89)^2}{0.6956}}$$

(9)

Figure 2 shows the visualization of the fuzzy distribution of corrosion initiation.

---

**Table 1: Basic variables for $T_i$**

<table>
<thead>
<tr>
<th>Variable</th>
<th>pdf type</th>
<th>Mean</th>
<th>Standard division</th>
<th>unit</th>
<th>reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\bar{C}$</td>
<td>normal</td>
<td>40</td>
<td>8</td>
<td>[mm]</td>
<td>[7]</td>
</tr>
<tr>
<td>$\bar{D}_c$</td>
<td>normal</td>
<td>30</td>
<td>5</td>
<td>[mm²/year]</td>
<td>[7]</td>
</tr>
<tr>
<td>$\bar{c}_c$</td>
<td>normal</td>
<td>0.3</td>
<td>0.05</td>
<td>[%]</td>
<td>[7]</td>
</tr>
<tr>
<td>$\bar{c}_t$</td>
<td>normal</td>
<td>0.650</td>
<td>0.03</td>
<td>[%]</td>
<td>[7]</td>
</tr>
</tbody>
</table>
### Table 2: Lognormal pdf fitting ANOV results for $T_i$ ($\mu=58.7226$, $\sigma=1434.28$)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Estimated</th>
<th>Std. Err.</th>
<th>Estimated Covariance of Parameter Estimates</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\mu$</td>
<td>3.89893</td>
<td>0.00589746</td>
<td>mu 3.47789e-005, sigma 2.9251e-019</td>
</tr>
<tr>
<td>$\sigma$</td>
<td>0.589736</td>
<td>0.00417038</td>
<td>mu sigma 2.9251e-019, 1.7392e-005</td>
</tr>
</tbody>
</table>

### Table 3: GA to adjust $\hat{T}_i$ MF parameters

<table>
<thead>
<tr>
<th>Type</th>
<th>Fitness function</th>
<th>Variable</th>
</tr>
</thead>
<tbody>
<tr>
<td>Membership function</td>
<td>Triangular</td>
<td>$\alpha \beta \gamma$</td>
</tr>
<tr>
<td></td>
<td>Trapezoidal</td>
<td>$\alpha \beta \gamma \delta$</td>
</tr>
<tr>
<td>Population type</td>
<td>double</td>
<td></td>
</tr>
<tr>
<td>Population size</td>
<td>30</td>
<td></td>
</tr>
<tr>
<td>Fitness scaling</td>
<td>rank</td>
<td></td>
</tr>
<tr>
<td>Selection</td>
<td>stochastic uniform</td>
<td></td>
</tr>
<tr>
<td>Mutation</td>
<td>adaptive feasible</td>
<td></td>
</tr>
<tr>
<td>Crossover</td>
<td>scattered</td>
<td></td>
</tr>
</tbody>
</table>

**Optimized parameters**

<table>
<thead>
<tr>
<th>MF type</th>
<th>Variables</th>
<th>Adjusted result</th>
</tr>
</thead>
<tbody>
<tr>
<td>Triangular</td>
<td>$\alpha \beta \gamma$</td>
<td>[6.7813 29.0 119.0]</td>
</tr>
<tr>
<td>Trapezoidal</td>
<td>$\alpha \beta \gamma \delta$</td>
<td>[7.5625 28.0 31.0000 118.50]</td>
</tr>
</tbody>
</table>

### 5.2. Genetic Algorithm to Get Standard Fuzzy MF Of $\hat{T}_i$

To extract the standard fuzzy membership functions for the corrosion initiation time, Eq. 2 or 3 should be minimized. In this paper we use genetic algorithm for its robustness in optimization task with detail summarized in Table 3. Moreover, Table 3 summarized the GA-optimized variables for each of triangular and trapezoidal standard fuzzy membership functions. Moreover, optimized fuzzy triangular and trapezoidal MF functions are demonstrated in Figure 3a and Figure 3b respectively.
6. FUZZY MODEL FOR CORROSION-INDUCED CRACKING TIME

The corrosion of reinforcing bar in the concrete has very wide ranges of deteriorative effects. Figure 4 shows some effects of corrosion on residual strength [8]. In this paper we consider the cracking of cover concrete due to the radial forces of expansive corrosion products.

Table 4: $\alpha$ for various corrosion products [10]

<table>
<thead>
<tr>
<th>corrosion product</th>
<th>FeO</th>
<th>Fe$_3$O$_4$</th>
<th>Fe$_2$O$_3$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\alpha$</td>
<td>0.777</td>
<td>0.724</td>
<td>0.699</td>
</tr>
</tbody>
</table>

Table 5: Basic variables for $T_{cr}$

<table>
<thead>
<tr>
<th>Variable</th>
<th>Distribution type</th>
<th>Mean</th>
<th>Standard deviation</th>
<th>Unit</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>$C$</td>
<td>normal</td>
<td>40</td>
<td>8</td>
<td>[mm]</td>
<td>[7]</td>
</tr>
<tr>
<td>$D_c$</td>
<td>&lt;&lt;&lt;&lt;</td>
<td>30</td>
<td>5</td>
<td>[%]</td>
<td>[7]</td>
</tr>
<tr>
<td>$C_{cr}$</td>
<td>&lt;&lt;&lt;&lt;</td>
<td>0.3</td>
<td>0.05</td>
<td>[%]</td>
<td>[7]</td>
</tr>
<tr>
<td>$C_r$</td>
<td>&lt;&lt;&lt;&lt;</td>
<td>0.650</td>
<td>0.03</td>
<td>[mm$^2$/year]</td>
<td>[7]</td>
</tr>
<tr>
<td>$\bar{D}$</td>
<td>&lt;&lt;&lt;&lt;</td>
<td>12</td>
<td>0.15</td>
<td>mm</td>
<td>[11]</td>
</tr>
<tr>
<td>$I_{cr}$</td>
<td>uniform</td>
<td>1.5</td>
<td>2.5</td>
<td>$\mu$A/cm$^2$</td>
<td>[7]</td>
</tr>
<tr>
<td>$d_0$</td>
<td>deterministic</td>
<td>12.5</td>
<td>---</td>
<td>$\mu$m</td>
<td>[7]</td>
</tr>
<tr>
<td>$\nu_c$</td>
<td>&lt;&lt;&lt;&lt;</td>
<td>0.18</td>
<td>---</td>
<td></td>
<td>[11]</td>
</tr>
<tr>
<td>$E_{cr}$</td>
<td>normal</td>
<td>18.82</td>
<td>0.12</td>
<td>GPa</td>
<td>[12]</td>
</tr>
<tr>
<td>$f'_i$</td>
<td>deterministic</td>
<td>3.3</td>
<td>---</td>
<td>MPa</td>
<td>[12]</td>
</tr>
<tr>
<td>$f'_c$</td>
<td>&lt;&lt;&lt;&lt;</td>
<td>31.5</td>
<td>---</td>
<td>MPa</td>
<td>[12]</td>
</tr>
<tr>
<td>$\alpha$</td>
<td>&lt;&lt;&lt;&lt;</td>
<td>0.57</td>
<td>---</td>
<td></td>
<td>[12]</td>
</tr>
<tr>
<td>$\rho_{rust}$</td>
<td>&lt;&lt;&lt;&lt;</td>
<td>3,600</td>
<td>---</td>
<td>kg/m$^3$</td>
<td>[12]</td>
</tr>
<tr>
<td>$\rho_{st}$</td>
<td>&lt;&lt;&lt;&lt;</td>
<td>7,850</td>
<td>---</td>
<td>kg/m$^3$</td>
<td>[12]</td>
</tr>
</tbody>
</table>
Table 6: Normal pdf fitting ANOV results for $T_{cr}$: $(mu=0.552096, \sigma=0.0143802$

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Estimated</th>
<th>Std. Err.:</th>
</tr>
</thead>
<tbody>
<tr>
<td>mu</td>
<td>0.552051</td>
<td>0.00119047</td>
</tr>
<tr>
<td>sigma</td>
<td>0.119047</td>
<td>0.000841853</td>
</tr>
</tbody>
</table>

Estimated covariance of parameter estimates:

<table>
<thead>
<tr>
<th>mu</th>
<th>sigma</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.41722e-006</td>
<td>1.41722e-006</td>
</tr>
<tr>
<td>7.08717e-007</td>
<td>-3.40944e-020</td>
</tr>
</tbody>
</table>

Table 7: GA-optimized MFs for $\tilde{T}_{ij}$

<table>
<thead>
<tr>
<th>MF type</th>
<th>Optimized parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Triangular</td>
<td>$[\alpha, \beta, \gamma]$</td>
</tr>
<tr>
<td>Trapezoidal</td>
<td>$[\alpha, \beta, \gamma, \delta]$</td>
</tr>
</tbody>
</table>

Adjusted result:

<table>
<thead>
<tr>
<th>Triangular</th>
<th>Trapezoidal</th>
</tr>
</thead>
<tbody>
<tr>
<td>[0.2729 0.5487 0.8339]</td>
<td>[0.2696 0.5346 0.5754 0.8229]</td>
</tr>
</tbody>
</table>

Extended probabilistic formulation for the time to corrosion-induced cracking time period was proposed as follows [9]:

$$\bar{T}_{cr} = \frac{\bar{W}^2_{crit}}{2\bar{k}_p}$$

(10)

Here we convert this formulation to its fuzzy model using the method described in section 4 as follows:

$$\hat{T}_{cr} = \frac{\hat{W}^2_{crit}}{2\hat{k}_p}$$

(11)

where $\hat{T}_{cr}$ is the fuzzy time to crack initiation, $\hat{W}_{crit}$ is the fuzzy critical amount of corrosion products, $\hat{k}_p$ is the fuzzy rate for rust production as the corrosion product. These quantities can be obtained using the following formulas:

$$\hat{W}_{crit} = \rho_{rust} \left[ \pi \left( \frac{\hat{D}}{E_{ef}} \left( \frac{a^2 + b^2}{b^2 - a^2} + \nu_c \right) + d \right) + \hat{W}_u \right] \hat{D}$$

(12)

where $D$ is the fuzzy diameter of rebar and $\hat{W}_u = \alpha \hat{W}_{crit}$ and,

$$\hat{k}_p = 0.105(\frac{1}{\alpha})\pi\hat{D} \cdot \tilde{i}_{cor}$$

(13)

where $\tilde{i}_{cor}$ is the fuzzy annual rate of corrosion in $\mu$A/cm$^2$ and $\alpha$ depends on the type of corrosion product which can be adopted from the Table 4.

6.1. Probability Distribution for Corrosion-Induced Cracking Time

The probability distribution for the corrosion cracking could be gained based on the
simple Monte Carlo Simulation (MCS) method. The basic variables for MCS of corrosion cracking are summarized in Table 5. Table 6, summarizes ANOV results for normal distribution fitted on the MCS of Eq. 11. According to the results of this table, the corrosion initiation lognormal pdf is:

\[
\bar{t}_{\text{cr, Norm}} \approx \frac{1}{0.552} e^{\frac{-((t-0.552051)^2}{0.0283}}
\]  

(14)

The supreme value for \(\bar{t}_{\text{cr}}\) is 3.35. By dividing the Eq. (14) to the supreme value, the fuzzy model for corrosion initiation can be gained as follows:

\[
\hat{t} = \frac{1}{0.0241t} e^{\frac{-(\ln(t-3.89)^2}}{0.6956}}
\]  

(15)

6.2. Genetic Algorithm to Get Standard Fuzzy MF Of \(\hat{t}_{\text{cr}}\)

Figure 5 shows the visualization of the fuzzy distribution of crack-initiation time. To extract the standard fuzzy membership functions for the corrosion initiation time, Eq. 2 or 3 should be minimized. As said before, in this paper we use genetic algorithm for its robustness in optimization task with detail similar to Table 3 with minor modifications. Table 7 summarized the GA-optimized variables for each of triangular and trapezoidal standard fuzzy membership functions. Moreover, optimized fuzzy triangular and trapezoidal MF functions are demonstrated in Figure 6a and Figure 6b respectively.
Figure 6. a) Optimized triangular MF for $T_{cr}$, b) Trapezoidal MF for $T_{cr}$

Figure 7. Integrated algorithm to model the corrosion initiation and propagation periods

Basic variables of corrosion of RC elements
- Cover thickness
- Diffusion coefficient
- Surface chloride concentration
- Critical chloride concentration
- Concrete modulus of elasticity
- Reinforcing bar diameter
- Corrosion rate

Experimentation and/or visual inspection
Characterizing appropriate probability distribution for all variables

Corrosion initiation fundamental model

Apply Monte Carlo Simulation
Estimation of the probability distribution of corrosion initiation and corrosion cracking model: $p(x)$

Convert probability distribution $p(x)$ into fuzzy-probability set:
$$f(x) = \frac{p(x)}{\sup_{\mu(x)}(\mu(x))}$$

Use genetic algorithm to minimize the
$$E = \sum_{i=1}^{n} [f(x; \alpha, \beta, \ldots) - f_{i}(x)]$$

Obtain optimized fuzzy membership variables for each model
Use developed fuzzy models in a fuzzy-support system to predict reliable service-life

Figure 7. Integrated algorithm to model the corrosion initiation and propagation periods
7. FUZZY-BASED ALGORITHM TO MODEL THE CORROSION OF RC ELEMENTS

At previous sections, we discussed about how to develop a fuzzy model from available stochastic models. Here we integrate these steps in an algorithm to be more useful. This algorithm is shown in Figure 7.

8. SUMMARY AND CONCLUSION

In this paper, we introduced a method to communicate with the available probabilistic information and conversion of them into the fuzzy sets. In this sense, the probability distribution functions for corrosion initiation and corrosion-cracking time period were converted into the fuzzy sets. The fuzzy sets are famous for their capabilities in processing the linguistic information rather than the random nature. So the constructed fuzzy sets could be used in a decision support system to eliminate the linguistic ambiguities of the corrosion in the RC structures. Thus the method introduced in this paper, is a major modules for developing a so-called Structural Health Monitoring System (SHMS) for reinforced concrete infrastructures.

REFERENCES

SCC CONTAINING POZZOLANIC MATERIALS AS FILLER REPLACEMENT AT ELEVATED TEMPERATURES

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1Professor in Civil Engineering, Amirkabir University of Technology, Tehran, Iran
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ABSTRACT
Few investigations have been reported on the properties of Self-Compacting Concrete (SCC) when it is exposed to elevated temperatures; because it is commonly understood that concrete can resist very well against elevated temperatures. Even so, it's necessary to evaluate all structures after they exposed to elevated temperatures.
Mechanical properties of SCC containing two types of Pozzolans (Silica Fume and Pumice) as filler replacement at elevated temperatures were experimentally investigated in this paper. At the age of 28 days, the specimens were placed in an electrical furnace and heating was applied up to the maximum temperatures of 200, 450, 600 and 800°C for 2 hr. Then, the specimens were allowed to be cooled in the furnace and subsequently tested for compressive strength, rebound hammer, ultrasonic pulse velocity and weight loss. The results show that concretes without Silica Fume and Pumice as a replacement for filler show slightly better performance in terms of lower strength loss.

Keywords: SCC, pozzolanic materials, elevated temperatures, mechanical properties, filler

1. INTRODUCTION
Self-Compacting Concrete (SCC) was first developed in 1988 to achieve durable concrete structures. Since then it has been used for a wide range of structures and infrastructures, such as bridges and tunnels. SCC is usually considered as a special type of High-Performance Concrete (HPC) produced with higher amounts of filler materials and lower water/binder ratios as compared with other concretes. Thus, porosity of SCC is usually reduced and the material is characterized by a high diffusion resistance [1]. Concrete mixture of high diffusion resistance such as SCC and HPC, are usually considered as more vulnerable to fire attack. Due to the lower porosity and lower connectivity of pores in SCC and HPC, the accumulating moisture and water vapor can hardly escape from the structure. So, very high pore pressure may be built up as functions of temperature, heating rate, and size of the specimens [2].
The cracking starts around the Ca(OH)₂ crystals and then progresses to areas near the unhydrated cement grains, as supported by Scanning Electron Microscopy (SEM) observations [3]. Cracking increases significantly as the temperature is
raised beyond 300°C [3,4]. When the maximum exposure temperature is below 300°C, concrete damage is dominated by only localized boundary cracking between the aggregates and the cement paste [5]. Cracks of the heated concrete could be further extended and developed during postcooling [6]. Therefore, a reduction of Ca(OH)₂ content in the cement paste containing supplementary cementing materials such as Silica Fume (SF), Pumice, etc., due to the pozzolanic reaction could help to reduce cracking due to postcooling. However, it should be noted that, above the dissociation temperature of Ca(OH)₂ at about 500°C, most concretes are likely to lose their structural properties [5].

Pozzolanic concretes are used extensively throughout the world; the oil, gas, nuclear, and power industries are among the major users. The applications of such concretes are increasing day by day due to their superior structural performance, environmental friendliness, and energy conserving implications [7]. As the use of Pozzolanic concretes becomes common, the risk of exposing them to elevated temperatures increases. So, it's necessary to evaluate all these structures after they are exposed to elevated temperatures.

This paper presents the results of an experimental investigation studying the mechanical properties of SCC containing two types of Pozzolans; Silica Fume (SF) and Pumice (P) that were used as filler replacement, subjected to elevated temperatures.

2. EXPERIMENTAL PROGRAM
2.1. Materials and Mix Designs
A total of four different mixtures were made; control SCC, Traditional Concrete (TC), one SCC with 7.5% Silica Fume (SF) replacing filler by weight and the other with 15% Pumice (P) replacing filler by weight.

Table 1 lists mix design proportions of SCCs and the TC. Properties of fresh and hardened concretes are depicted in Table 2. Local natural aggregate with maximum size of 10 mm; city potable water and Type I Portland cement were used. Limestone was used as filler. Superplasticizer was used according to the results obtained for the slumps. SCCs were prepared and tested in fresh conditions according to the EFNARC specifications [8].

2.2. Preparation of Specimens and tests
The specimens prepared were 100 (mm) cubes. Concrete test specimens were kept protected after casting to avoid water evaporation. After 24 hr the 100 (mm) cubes were cured for 28 days in lime-saturated water at 23 ± 2 °C to prevent possible leaching of Ca(OH)₂ from these specimens. Then the specimens used for measuring the 28 day compressive strength, rebound hammer number, pulse velocity and weight loss. At the age of 28 days, specimens were placed in an electrical furnace with heat applied at the rate of 2.5 (°C /min) until the desired temperature was reached (Figure 1). Before fire testing, two cubes were dried to reach to a constant mass.

A maximum temperature of 200, 450, 600 and 800 °C was maintained for 2 hr under the same conditions and without any imposed load. Specimens were then
allowed to cool in the furnace and tested for compressive strength, rebound hammer, pulse velocity and weight loss. Control tests were also performed on specimens cured at room temperature (23 ± 2°C). Residual compressive strength was determined as the mean value of two cubes tested per temperature, whereas rebound hammer was determined as the mean value of two measurements (two opposite sides of the cubes used for compressive strength measurements). Pulse velocity measurements were determined as the mean value of four measurements (two other opposite sides of the cubes used for compressive strength measurements) at any temperature. The weight loss of specimens was determined as the mean value of two cubes' weight loss, with which their weight being measured before and after the fire testing.

**Table 1: Mix design proportions of self-compacting concretes and the traditional concrete**

<table>
<thead>
<tr>
<th>Mixture</th>
<th>Constituents (Kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>TC</td>
<td>SCC</td>
</tr>
<tr>
<td>P2</td>
<td>SF2</td>
</tr>
<tr>
<td>450</td>
<td>450</td>
</tr>
<tr>
<td>34</td>
<td>67.5</td>
</tr>
<tr>
<td>82.5</td>
<td>116</td>
</tr>
<tr>
<td>890</td>
<td>900</td>
</tr>
<tr>
<td>593</td>
<td>600</td>
</tr>
<tr>
<td>180</td>
<td>0.4</td>
</tr>
<tr>
<td>0.4</td>
<td>1.1</td>
</tr>
</tbody>
</table>

- Type I Portland cement
- Silica Fume (SF)
- Pumice (P)
- Filler
- Coarse Aggregate
- Fine Aggregate
- Water
- w/c
- Superplasticizer (lt/100 kg of binder)

**Table 2. Proportions of fresh and hardened concretes**

<table>
<thead>
<tr>
<th>Mixture</th>
<th>Mixture properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>P2</td>
<td>SF2</td>
</tr>
<tr>
<td>700</td>
<td>680</td>
</tr>
<tr>
<td>1.1</td>
<td>0.97</td>
</tr>
<tr>
<td>5.7</td>
<td>5.16</td>
</tr>
<tr>
<td>69</td>
<td>70</td>
</tr>
</tbody>
</table>

- Slump (mm)
- Slump flow (mm)
- L Box (H₂/H₁)
- V-funnel (s)
- $f_{28}$ (Mpa)

Figure 1. Specimens in the electrical furnace
3. RESULTS AND DISCUSSION

3.1. Compressive Strength

The residual compressive strength at the age of 28 days for all mixtures is shown in Figure 2. When concretes are exposed to high temperatures, there are changes in the mechanical properties and the durability of them. However, the mechanisms causing these changes in properties is quite complex as a result of the concurrence of chemical and physical changes in hardened cement paste (HCP), aggregate, and at the interfaces.

The results show that all mixtures had an increase in their residual compressive strength up to 200 °C and then a sudden decrease occurred in SCC mixtures containing silica fume and pumice as replacement for filler. No spalling occurred at any temperature for all mixtures. In general, SCC with 15% pumice and another one with 7.5% silica fume replacement with filler (P2 & SF2) have higher strength loss with increasing temperatures than other mixtures without pozzolanic materials. The increase in compressive strength can be partially due to the strengthened HCP during the evaporation of free water [9,10]. Further hydration of cementitious materials is another important cause of the hardening of HCP [9].

![Figure 2. Residual compressive strength of all mixtures](image)

3.2. Rebound Hammer

The residual rebound hammer number at the age of 28 days for all mixtures is shown in Figure 3. The rebound values are influenced mainly by the condition of the surface of concrete to a depth not exceeding 3 cm approximately [11]. Since a temperature rise up to 200°C causes drying and hardening of the surface layer, rebound measurements present a small increase. At temperatures above 450°C, intensive internal cracking and chemical decomposition of the surface layer become more significant and rebound numbers show a significant reduction.
3.3. Ultrasonic Pulse Velocity

The residual pulse velocity at the age of 28 days for all mixtures is shown in Figure 4. It is clearly seen that pulse velocity reduces almost linearly with increasing temperature. It is obvious that the transmission of pulse waves through a concrete mass is highly influenced by the microcracking of concrete. Thus, the decrease in pulse velocity with increasing temperature is a sensitive measure of the progress of microcracking in the material.

Because microcracks might have developed along the boundary due to the swelling of physically bound water layers and the thermal incompatibility between aggregates and cement pastes [12]. Microcracking also increased significantly beyond 300°C, which is responsible for further durability loss in specimens heated to 450, 600, and 800°C [4,13].
3.4. Weight Loss

Figure 5 shows the weight loss at the age of 28 days for all mixtures at various temperatures. It can be observed that the TC samples show higher levels of weight loss than the others. Between 23 ±2°C and 200°C, a quick weight loss occurred in all samples, especially the TC and control SCC samples. This corresponds to the loss of the evaporable water and part of the physically bound water [2]. From 200 to 600°C, the weight loss includes the loss of chemically bound water from the decomposition of the CSH [2]. The weight loss of TC is higher than the others for temperatures up to 800°C. However, when the temperature is higher than 600°C, a dramatic loss of weight was observed in all samples. This is due to the decomposition of limestone filler, releasing carbon dioxide [14]:

\[
\text{CaCO}_3 \rightarrow \text{CaO} + \text{CO}_2
\]

(1)

![Figure 5. Weight loss of all mixtures](image)

3.5. Residual Compressive Strength and Residual Ultrasonic Pulse Velocity

Figure 6 shows the relation between residual compressive strength and residual ultrasonic pulse velocity for all concrete mixtures. At temperatures above 450°C, both compressive strength and ultrasonic pulse velocity decrease almost linearly with increasing temperature because of intensive internal cracking progress in the samples.

![Figure 6. Relation between residual compressive strength and residual ultrasonic pulse velocity for all mixtures](image)
It can be seen that in spite of the residual ultrasonic pulse velocities of all concrete mixtures were nearly equal, the residual compressive strength loss of SCC with 7.5% Silica Fume replacement with filler (SF2) was higher than others.

3.6. Residual Compressive Strength and Weight Loss
Figure 7 shows the relation between residual compressive strength and weight loss for all concrete mixtures. It is observed that the SCC mixtures with 15% pumice replacement with filler (P2) and the one with 7.5% silica fume replacement with filler (SF2) had higher residual compressive strength than the other specimens. As shown in this Figure, a linear relationship can be obtained for compressive strength and weight loss at temperatures between 450-800 °C.

![Figure 7. Relation between residual compressive strength and weight loss for all mixtures](image)

3.7. Residual Ultrasonic Pulse Velocity and Weight Loss
The relation between residual ultrasonic pulse velocity and weight loss at the age of 28 days for all mixtures is shown in Figure 8. It can be seen that higher temperature has resulted in higher weight losses due to the chemical decomposition of materials. This has caused microcracks in the cement pastes and micro-structure change and hence lower pulse velocity results. There is also no linear relationship between Ultrasonic Pulse Velocity (UPV) and weight loss at all temperatures.

![Figure 8. Relation between residual ultrasonic pulse velocity and weight loss for all mixtures](image)
4. CONCLUSIONS
The following conclusions were drawn from the study:
(1) In the range of 25–200°C, an increase in strength was observed in all concrete mixtures which can be resulted due to the evaporation of free water and further hydration of cementitious materials. From 200 to 450°C, a decrease in strength was observed in SCCs containing Silica Fume and Pumice as a replacement for filler. A loss in strength within the range of 70–75% was observed in the 400–600°C temperature range. At 800°C residual strength of SCCs varies between 25 and 30%. The sever loss in strength at an elevated temperature is probably due to the intensive internal cracking and chemical decomposition of concrete components.
(2) In general, concretes without Silica Fume and Pumice as a replacement for filler show slightly better performance in terms of lower strength loss.
(3) Results obtained for residual strength of heated samples by standard crushing test, rebound hammer and pulse velocity are different. This variation is attributed to the surface hardness measurement by hammer test and the influence of the microcracks on UPV test results.
(4) It is important that building designers, building officials, and the fire service organization be aware of the loss in mechanical properties of concretes which could reduce the load carrying capacity and durability of affected structural components.

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INVESTIGATION OF DURABILITY OF REINFORCEMENT CONCRETE IN SEVERE CORROSIVE MARINE ENVIRONMENT

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ABSTRACT
The intensity of damage in concrete structures has a vital relationship to the position of concrete versus the sea water level. With regard to this concept, the environment of located structures near the coast (i.e. coastal structures) can be divided into four zones: marine atmospheric, splashing, tidal and submerged [1, 2]. A hydraulic model consists of a channel with 10 meters in length, 0.7 meters width, 1 meter depth and a wave maker machine capable of making regular waves with various heights and periods, has been designed and constructed in hydraulic laboratory of Sahand University of Technology to simulate the marine environment and conditions in this research study. The Oroomiyeh lake water was used to reproduce severe corrosive marine environment. Two water/ cement ratios (0.45 and 0.55) and two cement contents (350 and 400 kg/m³) were selected for reinforcement and plain concrete specimens. Furthermore, silica fume was used as supplementary cementing material.

After standard curing, these specimens were kept in different conditions: submerged, tidal, splash, atmospheric and outdoors.

A series of tests such as corrosion potential measurement, electrical resistivity and reinforcement corrosion intensity, chloride ion concentration and compressive strength were carried out at different ages of concrete. In this paper, the function of specimens' durability in different zones have been investigated and compared with one to the others.

Keywords: reinforcement corrosion, chloride ion concentration, corrosion potential, splashing zone, atmospheric zone

1. INTRODUCTION
Due to the importance of the oil industry, marine transportation and mining in seas, construction of various structures (e.g. jetty, platform, etc) has been increased remarkably in recent years. Although concrete is a durable material, there are some reasons that can damage it in its lifetime. In designing the concrete structures, it is necessary to consider various factors to which the concrete should be exposed [3]. Research in real marine environment is very difficult to do as it needs many tools, equipments, etc. However, the marine conditions can be simulated partly in the
Investigations have been done in durability of concrete in marine environment across the world. For instance, marine durability of some concrete specimens in tidal condition in Japan coasts was reported in late 20th century [4]. Also, in I.R.Iran durability of concrete specimens in different conditions in Persian Gulf has been investigated by Building and Housing Research Center (BHRC) in 2006[5]. The specimens in latter research had been kept in real coastal environment, tidal and submerged conditions. In most researches, durability of concrete has been reported only for submerged, tidal and outdoors conditions. However, there were no reports in atmospheric and splashing conditions. In this research, durability of concrete in splashing and atmospheric conditions has been investigated in addition to submerged, tidal and outdoors conditions.

2. EXPERIMENTAL PROGRAM
2.1. Materials
Cement type 2 and silica fume have been used. The chemical analysis of cement and silica fume is shown in table 1.

<table>
<thead>
<tr>
<th>Chemical composition (%)</th>
<th>SiO₂</th>
<th>Al₂O₃</th>
<th>Fe₂O₃</th>
<th>MgO</th>
<th>CaO</th>
<th>SO₃</th>
<th>C₃S</th>
<th>C₅S</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement type 2</td>
<td>21.97</td>
<td>4.62</td>
<td>3.55</td>
<td>2.33</td>
<td>64.56</td>
<td>1.65</td>
<td>50.68</td>
<td>24.76</td>
</tr>
<tr>
<td>Silica fume</td>
<td>95.1</td>
<td>1.32</td>
<td>0.87</td>
<td>0.97</td>
<td>0.97</td>
<td>0.49</td>
<td>0.1</td>
<td>-</td>
</tr>
</tbody>
</table>

Gravel, having the size of 19mm at maximum and sand with a stiffness module of 2.94 were used. Gravel and sand unit weights are 2650, 2560 kg/m³, respectively. Tap water was used for mixing.

2.2. Mix Proportion, Specimens’ Details, Exposure Conditions and Tests
In this research, two water/cement ratios (0.45 and 0.55) and two cement contents (350 and 400 kg/m³) were selected. Silica fume was used as supplementary cementing material. Expected slump in all mixes gained by adding plasticizer up to 1% of total mass of cementitious material. Concrete mix proportions are shown in Table 2.

<table>
<thead>
<tr>
<th>Abbreviation Symbol</th>
<th>Concrete mix</th>
<th>(\frac{W}{C})</th>
<th>Weight (kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>M1</td>
<td>Cement type 2</td>
<td>0.55</td>
<td>Cement type 2: 350</td>
</tr>
<tr>
<td>M2</td>
<td>Cement type 2 + 10% Silica fume</td>
<td>0.45</td>
<td>Cement type 2: 360</td>
</tr>
</tbody>
</table>

All specimens were kept in mold for 24 hours, then in tap water in laboratory environment for 72 hours and finally they were moved to different conditions:
• Outdoors: laboratory environment
• Reference environment: pond tap water in the laboratory conditions
• Submerge zone: A pond containing Oroomiyeh Lake water (the water was renewed every month) in the average temperature of 30°C.
• Tidal zone: Composed of two ponds containing Oroomiyeh Lake water. The water was pumped from the first pond to the second and vice versa. The specimens were on average subjected to 10h of wetting and 14h of drying per 24h. The average pond water temperature was approximately 30°C.
• Splashing zone: A channel with 10 meters length, 0.7 meters width, 1 meter depth and a wave maker machine capable of making regular waves. This channel contained Oroomiyeh Lake water. The specimens were kept at the still water level to be exposed to the waves and splashes.
• Atmospheric zone: In this simulation, a pump and several nozzles were used to spray water into the system. The specimens were kept at a short distance from nozzles to get more accurate results.

In Table 3, a comparison between the constituents of Oroomiyeh Lake and Persian Gulf water is shown.

<table>
<thead>
<tr>
<th>Table 3: Chemical analysis of Oroomiyeh lake water</th>
<th>K⁺</th>
<th>Ca²⁺</th>
<th>Mg²⁺</th>
<th>So₄²⁻</th>
<th>Na⁺</th>
<th>Cl⁻</th>
</tr>
</thead>
<tbody>
<tr>
<td>Oroomiyeh lake</td>
<td>1404</td>
<td>1360</td>
<td>12418</td>
<td>22752</td>
<td>86066</td>
<td>143704</td>
</tr>
<tr>
<td>Persian Gulf</td>
<td>450</td>
<td>430</td>
<td>1460</td>
<td>2720</td>
<td>12400</td>
<td>21450</td>
</tr>
<tr>
<td>Ratio of Oroomiyeh lake water to Persian Gulf's water</td>
<td>3.12</td>
<td>3.16</td>
<td>8.5</td>
<td>8.3</td>
<td>6.9</td>
<td>6.7</td>
</tr>
</tbody>
</table>

The details of specimens and keeping conditions for each test are shown in Table 4.

| Table 4: Details and characteristics of specimens and the keeping conditions |
|---------------------------------|-----------------|-----------------|
| Test                            | Specimens size (cm) | Characteristics | keeping conditions                     |
| compressive strength            | 10×10×10         | -               | submerged, tidal, splash, atmospheric and Reference environment |
| chloride ion concentration      | 10×10×10         | -               | submerged, tidal, splash, atmospheric |
| corrosion potential and intensity | 10×10×20     | 2.5 cm concrete cover | submerged, tidal, splash, atmospheric |
| electrical resistivity          | 10×10×10         | -               | submerged, tidal, splash, atmospheric and Reference environment |

3. TESTS AND RESULTS
Compressive strength of specimens in different conditions was measured at ages of 28 and 90 days. Furthermore, corrosion potential, reinforcement corrosion intensity
and electrical resistivity were measured at different ages and conditions. The half cell apparatus with Ag/AgCl reference electrode was used to measure the rate of corrosion potential [6]. Reinforcement corrosion intensity was also measured by Potentiostat [7, 8]. Electrical resistivity was measured by using Weston (standard) cell [9]. Chloride ion concentration was measured at the age of 3 months used concrete by powder sample from the depth of 2-3 cm and chloride ion (% wt. of concrete) determined.

![Graph](image1)

**Figure 1.** Half cell potential for various mixes in different conditions at the age of 90 days

![Graph](image2)

(a)
Figure 2. Compressive strength in different conditions at the age of 90 days for a) M1 mix b) M2 mix

Table 5: Corrosion potential range and corrosion probability based on ASTM-C876 for Ag/AgCl Half Cell [6]

<table>
<thead>
<tr>
<th>Corrosion potential range</th>
<th>Corrosion probability</th>
</tr>
</thead>
<tbody>
<tr>
<td>≤-84 mv</td>
<td>There is 90% probability that corrosion does not exist.</td>
</tr>
<tr>
<td>-234 mv ≤ ≤ -84 mv</td>
<td>Corrosion does not exist positively but is absolutely possible</td>
</tr>
<tr>
<td>≤-234 mv</td>
<td>There is 90% probability for corrosion</td>
</tr>
</tbody>
</table>

Table 6: Chloride ion (% wt. of concrete) in different conditions at the age of 90 days.

<table>
<thead>
<tr>
<th>Abbreviation Symbol</th>
<th>Concrete mix</th>
<th>Chloride ion (% wt. of concrete)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Submerged zone</td>
</tr>
<tr>
<td>M1</td>
<td>Cement type 2</td>
<td>0.55</td>
</tr>
<tr>
<td></td>
<td>Cement type 2 + 10% Silica fume</td>
<td>0.45</td>
</tr>
</tbody>
</table>

Table 7: Corrosion current density for various mixes in different conditions at the age of 180 days.

<table>
<thead>
<tr>
<th>Abbreviation Symbol</th>
<th>Concrete mix</th>
<th>Corrosion current density (µA/cm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Submerged zone</td>
</tr>
<tr>
<td>M1</td>
<td>Cement type 2</td>
<td>0.55</td>
</tr>
<tr>
<td></td>
<td>Cement type 2 + 10% Silica fume</td>
<td>0.45</td>
</tr>
</tbody>
</table>
Table 8: Corrosion intensity range and its interpretation [8]

<table>
<thead>
<tr>
<th>Corrosion Current Density (µA/cm²)</th>
<th>Extent of Corrosion</th>
</tr>
</thead>
<tbody>
<tr>
<td>I_{corr} &lt; 0.1</td>
<td>Passive condition</td>
</tr>
<tr>
<td>0.1 &lt; I_{corr} &lt; 0.5</td>
<td>Low to moderate corrosion</td>
</tr>
<tr>
<td>0.5 &lt; I_{corr} &lt; 1</td>
<td>Moderate to high corrosion</td>
</tr>
<tr>
<td>I_{corr} &gt; 1</td>
<td>High corrosion</td>
</tr>
</tbody>
</table>

Figure 3: Chloride ion (% wt. of concrete) and Electrical resistivity in different conditions at the age of 90 days for a) M1 mix b) M2 mix
4. EXPERIMENTAL RESULTS AND DISCUSSION

4.1. Chloride Ion Penetration
For the splashing zone, due to capillary absorption and surface condensation phenomenon, concrete is more prone to be damaged [10].
In the second mix, because of the reduction of water/cement ratio and effects of silica fume on making concrete pores smaller, the rate of chloride ion penetration decreased.

4.2. Corrosion Potential and Intensity
In terms of corrosion, the splashing zone has the worst conditions, by increasing moisture, free chloride ion concrete and enough oxygen, corrosion intensity also increased.
In submerged condition, because of thin air, although corrosion potential of reinforcement was high, the rate of corrosion intensity was low.

4.3. Compressive Strength
Presence of Sulphate (SO$_4^{2-}$) ion in corrosive environment caused some decreases in compressive strength for all concrete mixes in comparison with reference specimens. Specimens in splashing zone have the highest strength deterioration factor (SDF) because of leach out, efflorescence under cyclic consequence wetting, drying and salt crystallization (which creates internal pressure and causes cracking of concrete).
In the second mix, decreasing of water/cement ratio and using silica fume caused lower SDF [11].

5. CONCLUSION
1. The main reasons of destruction of marine concrete structures are the chloride ion penetration and reinforcement corrosion which result in cracking of concrete.
2. Splashing condition has the highest chloride ion penetration in concrete which decreases electrical resistivity and increases reinforcement corrosion intensity.
3. Specimens in splashing condition have the highest strength deterioration factor.
4. In marine environment, concrete in splashing zone, which is exposed to waves and splashes, is more vulnerable in comparison with other zones and needs special attention in the curing of buildings.
5. In submerged condition, because of thin air, although corrosion potential of reinforcement is high, the rate of corrosion intensity is low.
6. In all specimens kept in various conditions, electrical resistivity was decreased as time passed. This attenuation may result in an increase in corrosion intensity.
7. Comparison between the experimental results shows that reinforcement corrosion intensity reduces in splashing, tidal, atmospheric and submerged condition, respectively.
REFERENCE
CONTRADICTORY EFFECTS OF SILICA FUME CONCRETES IN SULFURIC ACID ENVIRONMENTS

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ABSTRACT
In certain industrial activities sulfuric acid is used during the production process, which may cause degradation of concrete structures. Another important phenomenon where sulfuric acid is responsible for concrete corrosion is biogenic sulfuric acid corrosion, which occurs often in sewer systems. Therefore, researchers used sulfuric acid solutions to simulate such aggressive environments. Also, they used suitable cement replacement materials such as silica fume to improve acid resistance of concretes. But some researchers reported lowest susceptibility of concrete containing silica fume to corrosion at such environments while the other groups reported unsuitable performance of silica fume.

In this paper, control specimens and specimens containing 8% silica fume as cement replacement materials were immersed in sulfuric acid solutions with pH of 1.0 and 2.0. The dense packing of siliceous aggregates and cementitious materials were used to achieve low porosity concretes. They were periodically examined for appearance and measured for mass change up to 315 days. Total porosity of samples at the age of 90 days was measured. Results show that the porosity plays an important and twofold role in the mass loss of mortar specimens immersed in sulfuric acid solutions. In low pHs such as 1.0 corrosion rate of low porosity concretes is more than that of high porosity concretes. But, in the high pHs such as 2.0 corrosion rate of low porosity concretes is less than that of the high porosity concretes. In the low and high pHs of sulfuric acid solutions internal and external or surface degradations were observed respectively.

Contradictory results were obtained for the concretes containing silica fume because of its effect on the porosity and the production of silica gel at ITZ by reacting of portlandite. Investigations by optical microscopy on thin section samples indicate that the use of silica fume makes mixtures more homogeneous and hence lowers the corrosion rate in sulfuric acid environments.

Keywords: silica fume, sulfuric acid attack, dense concretes, porosity

1. INTRODUCTION
Concrete is the most widely used construction material for sewer structures. However, the environment in some sewer structures can become very acidic due
mainly to formation of sulfuric acid converted from hydrogen sulfide by bacterial action. Significant deterioration of concrete in such harsh environments has been reported all over the world [1,2,3]. Also several reports were published elucidating the mechanisms of concrete deteriorations in sewer environments [4,5,6]. Although, it has been reported that some new materials such as high performance coating, glass fiber reinforced lining, special mortars and high proportions of polymer modified binder can be more acid resistant, but they are too costly for most practical applications [7,8,9]. Therefore, the research on evaluation of acid resistance of normal concretes is still attractive.

It is well known that the porosity of the cement paste is the most important parameter determining mechanical properties and consequently, the durability of the material in the hardened stage. Therefore, information on porosity is a paramount importance for engineering concrete applications. The densest packing of the complete mixture made of aggregates and binder can lead to an extremely high density, low porosity system, and at the same time to the minimum binder content requirement.

In this paper, the dense packing of siliceous aggregates was obtained by replicating ASTM C29 standard test method for the aggregate fractions. The Fuller ideal grading curve for particle size distribution (PSD) of cementitious material including ultra fine filler was used. Finally, the effect of different pHs of sulfuric acid solutions, silica fume and ultra fine filler on resistance of mixtures to sulfuric acid attack were investigated. Sulfuric acid solution with pH of 1.0 and 2.0 were considered to represent the aggressive sewer environments. The pH of 1.0 is widely used in many laboratory tests to investigate the acid resistance of concretes for sewer structures [6,7,10]. But the pH of 2.0 is considered by some researchers to simulate the aggressive environments [11, 12]. Three mixtures including control mixture, mixture containing ultra fine filler and mixture containing 8% silica fume and ultra fine filler were investigated. Mortar and concrete samples were immersed in sulfuric acid solution for over 315 days. Water binder ratio and cementitious material content were considered as 0.42 and 325kg/m³ respectively. Specimens were regularly investigated by visual inspection of surface deterioration and measuring mass change. Porosity of mortar samples was investigated to find more knowledge about the mechanism of sulfuric acid attack. Also cement matrix and protection layer in samples that is formed during the sulfuric acid attack were investigated by optical microscopy on thin section samples.

2. EXPERIMENTAL PROGRAM
2.1. Materials
The materials used in this investigation were locally sourced and they satisfied the requirements of respective National Standards. Table 1 presents the results for typical chemical compositions of the Type II Portland cement, silica fume, and Quartz powder. The coarse and fine aggregates used in this investigation were 5-20mm and 0-5mm siliceous crushed river gravel and silica river sand respectively. A superplasticizer was used in the concrete mixtures to achieve a slump between 50-100mm.
### Table 1. Chemical composition of cement, silica fume and Quartz powder

<table>
<thead>
<tr>
<th>Oxide</th>
<th>CaO</th>
<th>SiO₂</th>
<th>Al₂O₃</th>
<th>Fe₂O₃</th>
<th>SO₃</th>
<th>MgO</th>
<th>(Na₂O)eq</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>62.94</td>
<td>21.1</td>
<td>5.05</td>
<td>3.08</td>
<td>1.5</td>
<td>3.4</td>
<td>0.87</td>
</tr>
<tr>
<td>Silica fume</td>
<td>1.02</td>
<td>95.1</td>
<td>0.6</td>
<td>1.1</td>
<td>1.2</td>
<td>0.6</td>
<td>-</td>
</tr>
<tr>
<td>Quartz powder</td>
<td>1.05</td>
<td>96.4</td>
<td>5</td>
<td>1.08</td>
<td>-</td>
<td>0.5</td>
<td>-</td>
</tr>
</tbody>
</table>

### 2.2. Material Proportions

#### 2.2.1. Aggregates

Aggregates take up 60-90% of the total volume of concrete. Proper selection of aggregate type and particle size distribution affects the main properties of concrete such as workability, mechanical strength, permeability, durability and the total cost of hardened concrete. High density along with low specific surface of aggregates is used to achieve minimum cement consumption and low porosity concrete production by replicating ASTM C29 standard test method. Process of aggregate proportion selection was published in the previous works [13-16]. Grading curve of selected aggregates and BS standard limits are shown in Figure 1.

![Grading curve of aggregates and BS standard limits](image)

**Figure 1.** Grading curve of aggregates and BS standard limits.

### 2.3. Quartz Powder Content

It is well known that porosity has an important role on many properties of concrete such as strength, permeability and durability. To obtain a dense structure in the hardened mortars, the density of dry binder should be maximized [17]. The packing density of the commercial cement powder is relatively low because of its narrow PSD obtained through a closed-circuit grinding process. It could be demonstrated mathematically that the cement powder lacks the section of super fine particles compared with the dense packing powder [18, 19]. In order to achieve the dense packing powder in the cementitious material, a very fine Quartz powder as a filler with particle size between 0-16 microns was added to the cementitious material. Proportion of super fine Quartz powder (\(k\)) was calculated from optimization of
followed objective function:

\[
F = \sum_{i=1}^{n}
\left[
U_i(D_i) - \left((1-k)U_2(D_i) + kU_3(D_i)\right)\right]^2
\]  

(1)

Where \( n \) is the number of sieves, \( U_i(D_i) \): is the PSD of ideal grading curve according to Fuller ideal curve by \( n = 0.38 \), \( U_2(D_i) \): is the PSD of cementitious material and \( U_3(D_i) \): is the PSD of ultra fine filler or Quartz powder. Optimization process has been published elsewhere [13, 14]. Results indicated that the use of 22% and 13.7% ultra fine filler in the second and third mixtures could lead to the production of low porosity concretes. Exception was the control concrete mixture which had no filler addition. Quartz powder was used as aggregate replacement in the second and third mixtures.

2.4. Preparation of Test Specimens

Three concrete mixtures were designed with constant cementitious material contents and water binder ratios to investigate their mechanical properties and resistance in sulfuric acid solution with pH of 1.0 and 2.0. Water binder ratio and cementitious material content were considered 0.42 and 325kg/m³ respectively. Mixture proportions are shown in Table 2. Mortar plates (10×10×2cm) and concrete cubes (10×10×10cm) were cast from each mixture according to ASTM C192 test method. In addition, mortar prisms (4×4×16cm) and concrete cubes (10×10×10 cm and 15×15×15cm) were cast for flexural, compression and water permeability tests respectively. All mortars were obtained by sieving concretes using No. 4 sieve. After 24 hours, specimens were demoulded and cured up to 28 days in the control room. Some specimens remained in the control room up to 90 days for flexural, compression and water permeability tests. The samples were then immersed in acid solution for a period of 315 days. Specimens were periodically washed during the test period and then their mass changes were measured. All measurements were carried out at the saturated surface dry condition (SSD).

<table>
<thead>
<tr>
<th>Mixture</th>
<th>W/C</th>
<th>Cement (kg)</th>
<th>Silica fume (kg)</th>
<th>Trass (kg)</th>
<th>Pumice (kg)</th>
<th>Filler (kg)</th>
<th>Sand (kg)</th>
<th>Gravel (kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C₁</td>
<td>0.42</td>
<td>325.0</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>889.4</td>
<td>1019.9</td>
</tr>
<tr>
<td>C₂</td>
<td>0.42</td>
<td>325.0</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>71.5</td>
<td>852.7</td>
<td>977.70</td>
</tr>
<tr>
<td>C₃</td>
<td>0.42</td>
<td>299.0</td>
<td>26</td>
<td>---</td>
<td>---</td>
<td>44.5</td>
<td>863.4</td>
<td>990.00</td>
</tr>
</tbody>
</table>

2.5. Sulfuric Acid Solution

The ASTM C267 test method was modified to investigate mortars and concretes in sulfuric acid solution. A sulfuric acid solution with pH of 1.0 and 2.0 were chosen to simulate the aggressive environment of sewer structures. The pH of sulfuric acid solution was kept constant in the range of 0.97 to 1.07 for pH of 1.0 and 1.87 to
2.23 for pH of 2.0 by adjusting the pH weekly using 98% sulfuric acid. All specimens were continuously immersed in monthly refreshed sulfuric acid solution during the test period. Test setup is shown in Figure 2.

Figure 2. Test setup for simulation of sewer structures

3. TEST RESULTS AND DISCUSSIONS

3.1. Air Content, Water Absorption and Water Penetration Depth

Air content of fresh concrete, half an hour and 24 hour water absorption at the age of 28 days and water penetration depth of hardened concretes at the ages of 28 and 90 days were measured according to ASTM C231, BS1881-122, ASTM C642 and DIN 1048 test methods, respectively. Test results are summarized in Table 3. Selection of dense packing aggregate proportions decreased the air content, water absorption and water penetration depth of the control mixture in comparison with the plain concretes. Utilization of ultra fine Quartz powder that improves PSD of cement decreases the air content, water absorption and water penetration depth of the second mixture \( C_2 \) when compared with the control one. In addition using silica fume that improve the interfacial transition zone (ITZ) decrease the above parameters in comparison with the second mixture \( C_2 \). The dense packing of aggregates and dry binder seems to disconnect the capillary pores. Disconnection of voids in the \( C_2 \) and \( C_3 \) mixtures seems to be greater than the control mixture due to the usage of ultra fine filler and silica fume.

Table 3: Air content of fresh concrete, water absorption and water penetration depth (WPD)

<table>
<thead>
<tr>
<th>Mixture</th>
<th>W/C</th>
<th>Cementitious material (kg)</th>
<th>Air content (%)</th>
<th>Water Absorption Half hour (%)</th>
<th>Water Absorption 1 day (%)</th>
<th>WPD (28 days) mm</th>
<th>WPD (90 days) mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>( C_1 )</td>
<td>0.42</td>
<td>325</td>
<td>2.8</td>
<td>1.72</td>
<td>5.55</td>
<td>8</td>
<td>7</td>
</tr>
<tr>
<td>( C_2 )</td>
<td>0.42</td>
<td>325</td>
<td>2.0</td>
<td>1.49</td>
<td>4.07</td>
<td>3</td>
<td>2-3</td>
</tr>
<tr>
<td>( C_3 )</td>
<td>0.42</td>
<td>325</td>
<td>1.5</td>
<td>1.62</td>
<td>4.01</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>
3.2. Mechanical Properties of Concretes and Mortars

Compressive and flexural strengths test results for concrete mixtures under standard curing conditions are shown in Figure 3 and 4. Higher compressive strengths with low cementitious material content (325 kg/m³) are due to the dense packing of aggregates. Flexural strength of the third mixture or \( C_3 \) at the age of 28 days is higher than other mixtures because of the high pozzolanic activity of silica fume. Also, higher compressive and flexural strengths of control mixture were probably obtained due to low porosity production (see Table 4). Total porosity of mixtures containing ultra fine filler is more than the control mixture probably due to satisfactory compaction of control mixture. Therefore, compressive and flexural strengths of these mixtures were less than the predicted values.

![Figure 3. Compressive strength of concrete mixtures](image)

![Figure 4. Flexural strength of mortar prisms](image)

3.3. Acid Attack Test Results

3.3.1. Visual Inspection of Mortar and Concrete Specimens

During the immersion period in sulfuric acid solutions, the samples were periodically retrieved from the acid solutions for measurements and visual inspection of the surface appearance. At the early age of immersion, surface of
specimens changed to white and yellow color for specimens subjected to sulfuric acid solution with pH of 1.0 and 2.0 respectively as shown in Figure 5. Then, softening of the cement matrix due to excessive expansion and dissolution was also observed during visual inspections of the concrete and mortar samples. The expansions of mortar and concretes were increased by exposure of aggregates. Typical surface appearances of concrete cubes after 3 and 6 months immersion in sulfuric acid solution with pH of 1.0 are shown in Figure 6. After 3 months of immersion, it is clearly seen that the control concrete (C₁) containing Portland cement has suffered the most severe damage with exposure of coarse aggregates and significant loss of cement mortar at all external surfaces. For C₂ and C₃ concrete mixtures, the acid affects the edges, corners, and part of the surfaces of the specimens. After 6 months immersion, surfaces of the mixtures were corroded by acid with exposure of aggregates. After 315 days immersion, all specimens were corroded at different depths. It should be noted that, there is a tan layer between corroded and uncorroded concrete which is more resistant to acid (see Figure 10). Apparently this protective layer is formed near the surface of specimens subjected to sulfuric acid solutions with pH of 2.0. Also, typical surface appearances of mortar plates after 2 and 6 months immersion in sulfuric acid solution with pH of 2.0 are shown in Figure 7. After 2 months immersion, all mortar plates have suffered damage with exposure of aggregates and significant loss of cement mortar. After 6 months immersion, C₁ mixture performed better when compared with other mixtures.

Figure 5. Mortar plates of C₂ mixture after 1 week immersion in sulfuric acid solutions with pH of 1.0 and 2.0

Figure 6. Concrete cubes after 3 and 6 months immersion in sulfuric acid solution with pH of 1.0
3.4. Mass Change Measurement

The mass change of a sample as a percentage of the initial mass is a widely used indicator for assessment of the deterioration of concrete subjected to acid attack. In this investigation the initial mass of samples was determined under saturated surface dry (SSD) conditions at the age of 28 days. Then, the specimens were immersed in sulfuric acid solutions. The measurements of mass change of samples at SSD condition were taken weekly within the first and second months and monthly afterwards until 11 months except for the third, fifth and seventh week and eighth and tenth month. During measurement, the samples were rinsed with tap water, brushed gently with a plastic brush to remove loose particles and then measured for their SSD masses. Test results of mortar and concretes in sulfuric acid solution with pH of 1.0 are shown in Figure 8. Also weight loss of mortar plates in sulfuric acid solution with pH of 2.0 is shown in Figure 9. As shown in Figure 8, the control concrete or C1 samples showed mass gain over the first 14 days immersion in the acid solution. But, the C2 and C3 concrete mixtures had net mass gains over 45 days after immersion. In the mortar specimens, mass loss of C1 begins after 14 days immersion. However, mass loss of C2 and C3 begins after 30 days immersion. Deterioration of mortar specimens is higher than the concrete samples because of their low thicknesses and higher cementitious material contents. Also, mass loss of the control mixture or C1 in the concrete and mortar specimens is higher than the other mixtures. After 315 days immersion, the C1 to C3 concrete specimens showed a reduction in mass loss of about 31.9%, 30.1% and 30.6%, respectively. However 69.1%, 64.0% and 57.2% mass reduction in the mortar specimens after 7 months immersion were observed respectively. Some of the mortar specimens after 7 months immersion were failed. Therefore, based on visual inspections and weight loss test results, it can be seen that the usage of silica fume and ultra fine filler may enhance the service life of concretes against high concentration of sulfuric acid solutions.
Figure 8. Weight loss of samples after 315 days immersion in pH of 1.0 a) concrete cubes b) mortar plates

Figure 9. Weight loss of mortar samples after 11 months immersion in pH of 2.0
But, performance of mortars in sulfuric acid solutions with pH of 2.0 is quite different. The control mixture that had the worst performance in the sulfuric acid solution with pH of 1.0 showed minimum weight loss at about 8.2%. The C2 and C3 specimens showed a reduction of about 12.9% and 12.6% respectively. Based on visual aspects and weight loss test results, mechanism of sulfuric acid attack with different pHs is different. Therefore, micro structural analysis should be carried out on the samples for further clarifications.

Figure 10. Tan layer formation between corroded and uncorroded concrete in mixture

3.5. Micro Structural Analysis
3.5.1. Porosity Study
Mortar samples were taken from freshly broken prisms after 90 days curing in control room. Samples dried at 105°C until mass stabilization and then their total porosity were measured by using following equation:

\[ P_i = \left(1 - \frac{\rho}{\rho_0}\right) \times 100\% \]  

(2)

Where, \( \rho \) and \( \rho_0 \) are dry bulk and solid phase densities and \( P_i \) is the volume percent of total porosity. Test results are summarized in Tab. 4. Also, total porosity of C1 and C2 samples that is measured by mercury porosimeter is shown in Tab. 4. Comparison of total porosity that is measured by mercury porosimeter with total porosity obtained according to equation (2) indicates that the compaction of control mixture was satisfactory. Also, actually porosity plays a twofold role in sulfuric acid attack to concrete. When a sulfuric acid solution with low pHs such as 1.0 attacks the concrete with low porosity or dense concrete, the dense structure prevents the absorption of acid by the concrete. Consequently the acid reacts with the cement paste at the narrow depth of concrete causing significant degradation due to expansion of gypsum or ettringite. For high porosity concretes, owing to a more porous structure, concrete absorbs the acid which reacts with hydration products to form gypsum and ettringite, which would fill the pores of the concrete, and create a protective layer. This layer forms within the depth of specimens between corroded and uncorroded concrete causes reduction in chemical reactions.
(see Figure 10). Thus the depth of degradation which is a function of exposure time, pH of sulfuric acid solution and mixture parameters should be investigated separately.

### Table 4: Total porosity of mixtures after 90 days curing

<table>
<thead>
<tr>
<th>Samples ID</th>
<th>$\rho$ (g/cm$^3$)</th>
<th>$\rho_0$ (g/cm$^3$)</th>
<th>$P_t$</th>
<th>Vol.%</th>
<th>$P_{\text{m}}$ %, Mercury porosimeter</th>
</tr>
</thead>
<tbody>
<tr>
<td>C$_1$</td>
<td>2.23</td>
<td>2.557</td>
<td>12.8</td>
<td>10.4</td>
<td></td>
</tr>
<tr>
<td>C$_2$</td>
<td>2.17</td>
<td>2.586</td>
<td>16.1</td>
<td>10.1</td>
<td></td>
</tr>
<tr>
<td>C$_3$</td>
<td>2.17</td>
<td>2.632</td>
<td>17.6</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Also, this layer formed near the surface of specimens in low porosity concretes and deteriorated at a shorter time. Therefore, based on test results, using an air-entraining agent could improve the resistance of concretes against sulfuric acid solutions with low pHs as was investigated in the literature [20]. Also, improvement of concrete resistance against sulfuric acid by increasing of water to cement ratio have been reported by several researchers without investigation of corrosion depth [13,21,22]. However, in case of higher pH of sulfuric acid solutions such as 2.0, sulfuric acid solutions couldn't penetrate in the depth of concretes owing to low concentration of sulfate ions. Consequently the acid reacts with the cement paste at the narrow depth or surface of concretes in both low and high porosity concretes causing degradation. Surface of high porosity concretes is more than that of low porosity concretes. Therefore corrosion rate of high porosity concretes is higher than that of low porosity concretes. Therefore, based on twofold effect of porosity against sulfuric acid solutions, contradictory effect of silica fume could be elaborated.

#### 3.6. Investigate of Samples by Optical Microscopy

Mixture characteristics, Thickness of tan layer that is formed during the sulfuric acid attack between corroded and uncorroded concrete and products of degradation were investigated on thin sections using transmitted light microscopy. Mortar samples were taken from freshly broken prisms after 315 days immersion in sulfuric acid solution with pH of 1.0. Effect of ultra fine filler and silica fume on portlandite consumption which resulted in homogeneous mixtures is shown in Figure 11. Comparison of cement matrix of C$_2$ with control mixture indicates that the mixtures containing ultra fine filler is more homogeneous than that of the control mixture. Also using silica fume reduces calcium hydroxide content. In Figure 12, typical micrographs of transmitted light microscopy on thin section samples of C1 and C$_3$ mixtures in the corroded area are shown. Also, average thickness of tan layers measured by optical microscopy is summarized in Table 5. It is clearly seen that the thickness of the tan layer depends on the cementitious materials composition. In other words, a corrosion rate depends on stiffness of cement matrix. Usage of silica fume increases the thickness of tan layer owing to silica gel formation in the ITZ. Gypsum is observed in all samples in front of the
tan layer as a product of degradation (see Figure 12). Based on Figure 8, Table 4 and 5, weight loss of high porosity concretes begins at the later ages, but corrosion depth of high porosity concretes is probably more than that of the low porosity concretes.

<table>
<thead>
<tr>
<th>Mixture code</th>
<th>Thickness of tan layer (μm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C_1</td>
<td>200</td>
</tr>
<tr>
<td>C_3</td>
<td>290</td>
</tr>
</tbody>
</table>

4. CONCLUSIONS

Based on the test results, the following observations and conclusions are drawn:

1. Using an ideal aggregate grading decreases the porosity of mortars and concretes and increases their durability.
2. Most properties of concretes containing ultra fine filler such as compressive and flexural strengths, density, air content of fresh concrete and porosity were further improved.
3. Porosity plays a twofold role in different concentration of sulfuric acid solutions. External and internal corrosion was observed for high and low concentration, respectively.
4. For high concentration of sulfuric acid solutions that is caused internal corrosion, higher porosity concretes showed less mass loss. But, their
corrosion depths should be investigated separately.
5. Utilization of ultra fine Quartz powder reduces porosity and hence increasing the resistance of concretes against sulfuric acid attack.
6. Utilization of silica fume prepared with an ideal grading curve of the cementitious materials improved the durability of concretes in sulfuric acid environments.
7. Corrosion rate varies with variation in micro structure of cement matrix.
8. Gypsum is observed in all specimens subjected to sulfuric acid solutions with pH of 1.0 as a product of degradation.
9. Application of ideal grading curve for cementitious materials enhances the homogeneity of the mixtures.

REFERENCES
THE ANALYSIS OF CHLORIDE DIFFUSION COEFFICIENT IN CONCRETE BASED ON NEURAL NETWORK MODELS

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ABSTRACT

Chloride diffusion is one of the major causes of deterioration of concrete structures. A large amount of research has been conducted to study the chloride diffusion of concrete, both experimentally and theoretically. Because chloride diffusion experiments are time consuming, it is desirable to develop a model to predict the chloride profiles in concrete. This paper studies the feasibility of using a neural network as an adaptive synthesizer as well as a predictor to meet such a requirement.

So some neural network models to predict chloride diffusion coefficient were made. The models were trained by results of chloride profile experiments. Input parameters were water to binder ratios, the amount of silica-fume and environmental condition of samples. The output parameter was chloride diffusion coefficient.

Neural network models are multi layer Perceptron models and they differ in the number of hidden layers and neurons. To control the accuracy of the model, an ANNs model was made and the result of the model was compared with test specimens. The result demonstrates that both neural network models have the ability of predicting the chloride diffusion coefficient with good accuracy.

Keywords: neural network model, chloride diffusion coefficient

1. INTRODUCTION

Steel reinforced concrete is one of the most durable and cost effective construction materials. The durability of reinforced concrete depends on the surrounding environment and exposure conditions, including the factors such as carbonation, corrosion, alkali-slices reaction and freezing/thawing [1,2]. Corrosion of reinforced steel resulting from the ingress of chloride ion is one of the most important issues concerning the durability of concrete structures. The prevention of reinforcement corrosion is primarily in the design stage with the use of high quality concrete and adequate cover. It is well known that steel is protected from corrosion by a microscopically thin oxide layer (Passive film: $\gamma$-Fe$_2$O$_3$-H$_2$O) that is formed in the highly alkaline condition of concrete pore solution. This protective film suppresses the iron dissolution to negligibly low values and furthermore, this oxide is insoluble and highly stable [3]. Corrosion occurs by loss of the alkalinity of concrete in the form of carbonates, thereby providing a direct route for chlorides to
approach the reinforcing steel and prevent re-passivation reaction that leads to pitting corrosion [4]. Carbonates, chlorides and sulphates media can be found in concrete when using contaminant aggregate, or adding CaCl₂ during the mixing step or they are found under the effect of sea-water or ground water on concrete and they can also result from an attack on concrete by the surrounding environment in coastal regions. Carbonation destroys the protective oxide layer presented on the surface of embedded steel in concrete leading to corrosion. As the corrosion of embedded steel continues, the products formed exert enormous stress on the surrounding concrete leading to cracking and later spalling of the concrete. These stresses have been reported to be as high as 450 Mpa [5]. Methods of corrosion control include cathodic protection, surface treatments of the rebar and the use of admixtures in concrete [6]. Use of blended cements incorporating supplementary cementing materials such as silica-fume, blast furnace slag, fly ash or natural pozzolan, is a solution that leads to mixtures with greater resistance against chloride [7].

There are a number of computational analysis techniques that deal with concrete [8-12]. One of the most known techniques is artificial neural network (ANNs) [13, 16]. Topcu and Sndemire [17] that used ANNs and Fuzzy logic for prediction of mechanical properties of recycled aggregate concretes containing silica fume. They obtained successful simulation result from both ANNs and fuzzy logic. Altun et al. [18] used ANNs for predicting the compressive strength of steel fiber added lightweight concrete and they compared ANN result with multi layer regression technique results. They concluded that ANNs predicts the compressive strength of steel fiber added lightweight concrete more accurately than multi layer regression. Sakla and Ashour [19] predicted tensile capacity of single adhesive anchors using ANNs. They concluded that ANN is a useful technique for predicting of tensile capacity of adhesive anchors. Since ANNs has taken into account nonlinear transfer functions, they can automatically consider the nonlinear relations between the data. Hence better prediction results than other statistical tools can be obtained in general. Topcu et al. [3] used ANNs to model corrosion currents of reinforced concrete. They used two types of cement and 3 different ratios of fly ash for their modeling. Their Ann model produced close prediction current values to currents measured in experiment. They concluded that ANN is an appropriate tool for modeling the corrosion currents. Parichatprecha, and Nimityongskul.[20] used ANNs to durability analysis of high performance concretes. Their results indicated that the ANN models can be used to efficiently predict the chloride ions permeability across a wide range of ingredients of HPC. Based on the simulated total charge passed model, built using trained neural networks, they also concluded that the optimum cement content for the design of HPC in terms of total charge passed ranges from 450 to 500 kg/m³.

The aim of this study is to construct an ANNs model to investigate the influence of mix proportion parameters on the resistance of chloride ion penetrability on concretes containing silica-fume. For this purpose, data for developing the neural network model are collected from the experiments. The design of the experimental program is based on the relevant parameters, namely W/B, cement content, silica
fume content and some experimental data.

2. ARTIFICIAL NEURAL NETWORKS
Artificial neural networks are computing systems that simulate the biological neural systems of the human brain. They are based on a simplified modeling of the brain’s biological functions exhibiting the ability to learn, think, remember, reason, and solve problems. Conceptually, a neural networks model consists of a set of computational units and a set of one-way data connection joining units or weights as shown in Figure 1.

![Figure 1. Single processing element of ANNs](image)

Units that receive no input from others are called input nodes, while those with no outgoing links are called output nodes. All other intermediate units are called hidden nodes. The multi-layered model has several layers, and each layer consists of numerous neurons which are connected with each other. In this model, information is sent from input layer to output in one direction, and learning is preceded so as to minimize the difference between the output of the model and the target output. ANNs can solve challenging problems of interest to computer scientists and engineers such as pattern classification, categorization, function approximation, prediction and forecasting, optimization, content-addressable memory, and control robotics. Rumellhart et al. [21] developed a method called error back-propagation, or more simply back-propagation, for learning associations between input and output patterns using more than the two layers of Rosenblat’s original perceptron. Back-propagation is a supervised learning technique that compares the responses of the output units to the desired response, and readjusts the weights in the network so that the next time when the same input is presented to the network, the network’s response will be closer to the desired response. Errors that arise during the learning process can be expressed in terms of mean square error (MSE) and are calculated using Eq. (1).

$$MSE = \left( \frac{1}{p} \right) \sum_j (t_j - \sigma_j)^2$$  \hspace{1cm} (1)
In addition, the absolute fraction of variance ($R^2$) and mean absolute percentage error (MAPE) are calculated using Eqs. (2) and (3), respectively.

$$R^2 = 1 - \left( \frac{\sum_j (t_j - \sigma_j)^2}{\sum_j (\sigma_j)^2} \right)$$  \hspace{1cm} (2)$$

$$MAPE = \frac{1}{p} \sum_j \left( \frac{\sigma_j - t_j}{\sigma_j} \right) \times 100$$  \hspace{1cm} (3)$$

where $t_j$ is the target value of $j$th pattern, $\sigma_j$ is the output value of $j$th pattern, and $p$ is the number of patterns.

3. EXPERIMENTAL STUDIES
3.1. Materials Used
3.1.1. Cement and silica-fume
In experimental studies, the CEM I 425 R Portland cement which is produced by Tehran cement factory were used.

3.1.2. Aggregates
Crushed sand and crushed stone aggregates were used. The maximum particle size of aggregates is 20 mm. As a result of the experiment, the specific gravities of sand and crushed stone are obtained as 2.62 and 2.71 kg/dm$^3$, respectively.

3.2. Mix Proportions
Cement type I.425 was used in concrete mixtures. Concretes are produced using 0, 7 and 10% replacement level of SF by weight of cement. These specimens were cured at 28, 90 and 270 days. The amounts of materials used in 1 m$^3$ concrete are given in Table 1.

<table>
<thead>
<tr>
<th>Specimen code</th>
<th>W/B</th>
<th>csf/(c+csf)</th>
<th>sand</th>
<th>gravel</th>
</tr>
</thead>
<tbody>
<tr>
<td>M-35-0</td>
<td>0.35</td>
<td>0</td>
<td>800</td>
<td>1050</td>
</tr>
<tr>
<td>M-35-7</td>
<td>0.35</td>
<td>7</td>
<td>800</td>
<td>1050</td>
</tr>
<tr>
<td>M-35-10</td>
<td>0.35</td>
<td>10</td>
<td>800</td>
<td>1050</td>
</tr>
<tr>
<td>M-40-0</td>
<td>0.4</td>
<td>0</td>
<td>800</td>
<td>1050</td>
</tr>
<tr>
<td>M-40-7</td>
<td>0.4</td>
<td>7</td>
<td>800</td>
<td>1050</td>
</tr>
<tr>
<td>M-40-10</td>
<td>0.4</td>
<td>10</td>
<td>800</td>
<td>1050</td>
</tr>
<tr>
<td>M-50-0</td>
<td>0.5</td>
<td>0</td>
<td>800</td>
<td>1050</td>
</tr>
<tr>
<td>M-50-7</td>
<td>0.5</td>
<td>7</td>
<td>800</td>
<td>1050</td>
</tr>
<tr>
<td>M-50-10</td>
<td>0.5</td>
<td>10</td>
<td>800</td>
<td>1050</td>
</tr>
</tbody>
</table>

$csf$: content of silica-fume in concrete
4. EXPERIMENTAL PROGRAM AND DATA COLLECTION
The first step in developing the network is to obtain good and reliable training and testing examples. To obtain the data for developing the neural network models, different experiments were done on specimens. The aim of these experiments was to find a relationship between mix design and chloride diffusion coefficient in concrete. For this reason, the specimens were exposed to chloride in 3 different conditions for more than 270 days. The environmental conditions were submerge, tidal and atmospheric zone. Persian Gulf modeling room of Building and Housing Research Center (BHRC) was used to model the mentioned environment. In addition to this experiment, RCPT, concrete compressive strength and water permeability of concrete under pressure were done to find a relationship between concrete durability contents and chloride penetration coefficient. Results of experiments can be finding in ref. [22].

4.1. Variables Selected for Neural Networks
Considering the environmental conditions at the construction sites and in order to find the important variables that might strongly affect the chloride diffusion coefficient, 7 different ANNs were selected with different input variables and hidden layers. 1 variable was chosen as the desired output. Table 2 gives the list of the ANNs inputs and outputs. In this study, the neural networks were developed and performed under MATLAB programming. The learning algorithm used in the study was gradient descent with adaptive learning rate back-propagation, a network training function that updates weight and bias values according to gradient descent with adaptive learning rate [21]. The error incurred during the learning process was expressed in terms of mean-squared-error (MSE).

<table>
<thead>
<tr>
<th>Code</th>
<th>W/B</th>
<th>SF (%)</th>
<th>RCPT index</th>
<th>Time of exposing</th>
<th>RCPT index</th>
<th>Diffusion coefficient</th>
<th>Number of Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>M1</td>
<td>*</td>
<td>*</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>24</td>
</tr>
<tr>
<td>M2</td>
<td>*</td>
<td></td>
<td></td>
<td></td>
<td>*</td>
<td></td>
<td>16</td>
</tr>
<tr>
<td>M3</td>
<td>*</td>
<td></td>
<td></td>
<td></td>
<td>*</td>
<td></td>
<td>16</td>
</tr>
<tr>
<td>M4</td>
<td>*</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>24</td>
</tr>
<tr>
<td>M5</td>
<td>*</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>24</td>
</tr>
<tr>
<td>M6</td>
<td>*</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>24</td>
</tr>
<tr>
<td>M7</td>
<td>*</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>24</td>
</tr>
</tbody>
</table>

All model structures were based on the following cases:
1. The minimum and maximum neurons in the hidden layer were changing between 1.5 and 3 times the input number of parameters. For example, in the model with 2 input parameters, the number of hidden layer neurons was 3 to 6.
2. The number of iterations and MSE between output parameter of model and test data was the criteria used for selecting the best model.
5. RESULTS AND DISCUSSION

For 7 models, the summary of models has been collected in tables 3-9. According to the criteria mentioned for choosing the best model in each ANNs, the selected model has been shown in different colors in the rows.

Table 3: The summary of results of M1 ANNs model

<table>
<thead>
<tr>
<th>Code</th>
<th>Number of iterations</th>
<th>Number of neurons in hidden layer</th>
<th>MSE ($10^{-4}$)</th>
<th>MAPE</th>
</tr>
</thead>
<tbody>
<tr>
<td>M1-3-1</td>
<td>6</td>
<td>3</td>
<td>6.52</td>
<td>7.93</td>
</tr>
<tr>
<td>M1-4-4</td>
<td>5</td>
<td>4</td>
<td>6.52</td>
<td>7.93</td>
</tr>
<tr>
<td>M1-5-6</td>
<td>5</td>
<td>5</td>
<td>6.52</td>
<td>7.93</td>
</tr>
<tr>
<td>M1-6-4</td>
<td>4</td>
<td>6</td>
<td>6.52</td>
<td>7.93</td>
</tr>
</tbody>
</table>

Table 4: The summary of results of M2 ANNs model

<table>
<thead>
<tr>
<th>Code</th>
<th>Number of iterations</th>
<th>Number of neurons in hidden layer</th>
<th>MSE ($10^{-4}$)</th>
<th>MAPE</th>
</tr>
</thead>
<tbody>
<tr>
<td>M2-5-1</td>
<td>13</td>
<td>5</td>
<td>1</td>
<td>12.68</td>
</tr>
<tr>
<td>M2-6-5</td>
<td>9</td>
<td>6</td>
<td>1</td>
<td>22.93</td>
</tr>
<tr>
<td>M2-7-7</td>
<td>7</td>
<td>7</td>
<td>1</td>
<td>7.63</td>
</tr>
<tr>
<td>M2-8-2</td>
<td>7</td>
<td>8</td>
<td>1</td>
<td>11.97</td>
</tr>
<tr>
<td>M2-9-4</td>
<td>6</td>
<td>9</td>
<td>1</td>
<td>62.57</td>
</tr>
</tbody>
</table>

Table 5: The summary of results of M3 ANNs model

<table>
<thead>
<tr>
<th>Code</th>
<th>Number of iterations</th>
<th>Number of neurons in hidden layer</th>
<th>MSE ($10^{-4}$)</th>
<th>MAPE</th>
</tr>
</thead>
<tbody>
<tr>
<td>M3-5-3</td>
<td>9</td>
<td>5</td>
<td>1</td>
<td>21.50</td>
</tr>
<tr>
<td>M3-6-1</td>
<td>9</td>
<td>6</td>
<td>1</td>
<td>20.61</td>
</tr>
<tr>
<td>M3-7-2</td>
<td>8</td>
<td>7</td>
<td>1</td>
<td>9.30</td>
</tr>
<tr>
<td>M3-8-3</td>
<td>6</td>
<td>8</td>
<td>1</td>
<td>1.76</td>
</tr>
<tr>
<td>M3-9-2</td>
<td>5</td>
<td>9</td>
<td>1</td>
<td>16.14</td>
</tr>
</tbody>
</table>

Table 6: The summary of results of M4 ANNs model

<table>
<thead>
<tr>
<th>Code</th>
<th>Number of iterations</th>
<th>Number of neurons in hidden layer</th>
<th>MSE ($10^{-4}$)</th>
<th>MAPE</th>
</tr>
</thead>
<tbody>
<tr>
<td>M4-3-1</td>
<td>1000</td>
<td>3</td>
<td>5.01</td>
<td>120.4</td>
</tr>
<tr>
<td>M4-4-2</td>
<td>1000</td>
<td>4</td>
<td>1.19</td>
<td>84.97</td>
</tr>
<tr>
<td>M4-5-2</td>
<td>1000</td>
<td>5</td>
<td>0.02</td>
<td>92.90</td>
</tr>
<tr>
<td>M4-6-4</td>
<td>1000</td>
<td>6</td>
<td>0.0008</td>
<td>9894.78</td>
</tr>
</tbody>
</table>

Table 7: The summary of results of M5 ANNs model

<table>
<thead>
<tr>
<th>Code</th>
<th>Number of iterations</th>
<th>Number of neurons in hidden layer</th>
<th>MSE ($10^{-4}$)</th>
<th>MAPE</th>
</tr>
</thead>
<tbody>
<tr>
<td>M5-3-3</td>
<td>1000</td>
<td>3</td>
<td>3.23</td>
<td>66.05</td>
</tr>
<tr>
<td>M5-4-3</td>
<td>1000</td>
<td>4</td>
<td>0.772</td>
<td>23.12</td>
</tr>
<tr>
<td>M5-5-2</td>
<td>1000</td>
<td>5</td>
<td>0.0002</td>
<td>150.07</td>
</tr>
<tr>
<td>M5-6-2</td>
<td>1000</td>
<td>6</td>
<td>0.0919</td>
<td>205.68</td>
</tr>
</tbody>
</table>
Table 8: The summary of results of M6 ANNs model

<table>
<thead>
<tr>
<th>Code</th>
<th>Number of iterations</th>
<th>Number of neurons in hidden layer</th>
<th>MSE (*10^-4)</th>
<th>MAPE</th>
</tr>
</thead>
<tbody>
<tr>
<td>M6-3-3</td>
<td>1000</td>
<td>3</td>
<td>1.27</td>
<td>7.30</td>
</tr>
<tr>
<td>M6-4-3</td>
<td>1000</td>
<td>4</td>
<td>0.0975</td>
<td>8.64</td>
</tr>
<tr>
<td>M6-5-1</td>
<td>1000</td>
<td>5</td>
<td>0.0448</td>
<td>36.79</td>
</tr>
<tr>
<td>M6-6-2</td>
<td>1000</td>
<td>6</td>
<td>8.04*10^-9</td>
<td>29.46</td>
</tr>
</tbody>
</table>

Table 9: The summary of results of M7 ANNs model

<table>
<thead>
<tr>
<th>Code</th>
<th>Number of iterations</th>
<th>Number of neurons in hidden layer</th>
<th>MSE (*10^-8)</th>
<th>MAPE</th>
</tr>
</thead>
<tbody>
<tr>
<td>M7-3-3</td>
<td>1000</td>
<td>3</td>
<td>5.7</td>
<td>9.46</td>
</tr>
<tr>
<td>M7-4-1</td>
<td>1000</td>
<td>4</td>
<td>0.0683</td>
<td>14.27</td>
</tr>
<tr>
<td>M7-5-3</td>
<td>1000</td>
<td>5</td>
<td>0.0683</td>
<td>23.10</td>
</tr>
<tr>
<td>M7-6-1</td>
<td>1000</td>
<td>6</td>
<td>3.33*10^-8</td>
<td>84.35</td>
</tr>
</tbody>
</table>

As it can be seen from the results, the selection of mix design parameter (W/B and S.F percentage) makes better output than RCPT. It is because of the uncertainties of RCPT. Furthermore, both the number of neurons in hidden layer and number of hidden layers in relation with each other has a positive effect in ANNs output. It's because of the nonlinear nature of chloride diffusion in concrete.

6. CONCLUSION

After the tests, it is observed that the diffusion of chloride in concrete changes by SF ratio used instead of cement and water to binder ratio. As a result of the analysis, ANN structures that produce close prediction current values to measured ones are presented and the robustness of ANN structure is tested. 7 ANN model was tested and in each model, the input and output parameters was changed to find the best input variable for prediction of chloride diffusion coefficient in concrete. The results show that W/B ration and percentage of silica-fume in concrete are better inputs than RCPT results. Furthermore, the results show that both the number of neurons in hidden layer and number of hidden layers in relation with each other has positive effect in ANNs output. To sum up, it is concluded that ANN is an appropriate tool for modeling the diffusion coefficient of chloride in concrete.

REFERENCES

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PREDICTION OF CHLORIDE COEFFICIENT IN CONCRETE WITH DIMENSIONAL ANALYSIS IN PERSIAN GULF

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ABSTRACT
One of important causes for failure of concrete structures particular in Persian Gulf region is diffusion of chloride in concrete. One of the parameters which can increase the diffusion chloride ion is level of diffusivity of the concrete. Prediction of concrete diffusion factor is an important issue as a key parameter in life cycle of concrete structures. Experimental method in this field have problems such as time consummation, high cost, lack of experimental instruments, existence of inconvenience ions during experiment, committing unwanted errors and distorting from the true model. For this reason in this research singular value decomposition (SVD) for modeling the diffusivity of chloride ion in concrete has been used. The aim of this modeling is to find mathematical relation between the concrete diffusivity coefficient with effective parameters on it which means the proportion of water per cement and percent of silica fume.

Keywords: prediction, dimensionless analysis, persian gulf, chloride diffusion, SVD

1. INTRODUCTION
Concrete is such a construction material that is widely used in the world. The advantages of concrete are low cost, availability of constituents, workability, durability and convenient compressive strength that make it popular near engineers and builders. However, these advantages seriously depend on the correct mix and placing and curing [1,2].
Per year millions dollars are spent as the result of destruction of concrete structures near seashore and industrial refineries which contain chloride ions. The concrete used in this structures-specially sea structures- should resist against factors such as chloride attack and steel corrosion. In middle east specially in Persian gulf and southern banks of the Iran, the problems in concrete processing with high reliability cause high cost for the country[3].
Economically the best solution for preventing the precocious destruction is increasing the concrete resistance against chloride permeation during the concrete life. By operating of laboratory instruments we can produce concrete which can
fulfill our needs. But there is no clear way to determine the duration of concrete structures and predicting the diffusion of chloride. The main problem is the vast variation of mixing proportion and huge number of effective parameters [4]. One of the methods for predicting the life time of concrete in the world is gathering the result of experiments and the present structure information for making a model that its usage decreases costs and time, easing in application and flexibility in any concrete structure.

Prediction of concrete diffusion factor is an important issue as a key parameter in life cycle of concrete structures.

System identification and modeling of complex processes using input-output data have always attracted many research efforts. In fact, system identification techniques are applied in many fields in order to model and predict the behaviors of unknown and/or very complex systems based on given input-output data [5]. Theoretically, in order to model a system, it is required to understand the explicit mathematical input-output relationship precisely.

There have been many research efforts for theoretical modeling the properties of concrete to predict the relationship of concrete diffusion factor as a function of the water per cement ratio and silica fume percent [6].

In this paper, experimental data of Construction Materials Institute at the University of Tehran [7] in which parameters water per cement ratio and silica fume percent are considered as input variables are used to find an equation for predicting concrete diffusion factor as output variable using singular value decomposition (SVD) method. In this way, the above mentioned input variables are re-grouped as dimensionless parameters which are then used to obtain the closed-form equation of concrete diffusion factor under aggressive environment in Persian gulf experiments carried out by M.shekarchi zadeh and others [7]. Such hybrid application of SVD and dimensionless analysis modeling is very simple and promising in modeling of the complex processes such as concrete diffusion factor under aggressive environment.

2. EXPERIMENTAL PROCEDURE

Experiments were carried out on result of experimental data on concrete durability in the south of Iran region.

In that regard, 20 concrete prism specimens measuring 15×15×60cm were exposed to the marine environment of Bandar Abbas city in the south of Iran, see Figure 1. All concrete specimens were made with 400kg/m³ Portland cement type II and crushed coarse aggregate (maximum size of 12.5 mm). Among the variables, five percentages of cement replacement with silica fume (0, 5, 7.5, 10, and 12%), and four water to cement ratios (0.35, 0.40, 0.45, and 0.50) were Investigated [7,8].

The data for the chloride diffusion coefficient for the age of 86 days were obtained. In this age, a 10 cm part of the specimens was cut for measuring the chloride ion content at different depths from the surface, see Figure 2, and thereby obtain the associated chloride profile on the top and bottom surfaces of the specimen.
3. DIMENSIONLESS MODELING OF CONCRETE DIFFUSION FACTOR USING SVD

The use of dimensionless parameter in modeling of concrete diffusion factor has been reported by Institute at the University of Tehran [7] using some experimental data, including those used by Beheshti Nezhad and Ranjbar [6]. They applied group method of data handling (GMDH)-type neural networks for modeling and prediction of concrete diffusion factor. However, singular value decomposition (SVD) and dimensionless parameters can be readily used together to obtain a simple equation for predicting the concrete diffusion factor based on the experimental data.

The formal definition of modeling is to find a function $\hat{f}$ so that can be approximately used instead of actual one, $f$, in order to predict output $\hat{y}$ for a given input vector $X = (x_1, x_2, x_3, ..., x_n)$ as close as possible to its actual output $y$. Therefore given $M$ observation of multi-input-single-output data pairs so that...
\[ y_i = f(x_{i1}, x_{i2}, x_{i3}, \ldots, x_{in}) \quad i = 1, 2, \ldots, M \]  

(1)

it is now possible to obtain \( \hat{f} \) to predict the output values \( \hat{y}_i \) for any given input vector

\[ X_i = (x_{i1}, x_{i2}, x_{i3}, \ldots, x_{in}) \]  

(2)

Such that

\[ \hat{y}_i = \hat{f}(x_{i1}, x_{i2}, x_{i3}, \ldots, x_{in}) \quad i = 1, 2, \ldots, M \]  

(3)

The problem is now to determine \( \hat{f} \) so that the square of the difference between the actual output and the predicted one is minimized, i.e.

\[ \sum_{i=1}^{M} [\hat{f}(x_{i1}, x_{i2}, x_{i3}, \ldots, x_{in}) - y_i]^2 \to \text{Min}. \]  

(4)

In dimensionless modeling, however, a dimensionless set, \( \pi = \{\pi_0, \pi_1, \pi_2, \ldots, \pi_k\} \), rather than the set of real physical variables \( \{y, X\} = \{y, x_1, x_2, x_3, \ldots, x_n\} \), is used to obtain \( \hat{f} \), i.e.

\[ \hat{\pi}_{0i} = \hat{f}(\pi_{1j}, \pi_{2j}, \pi_{3j}, \ldots, \pi_{kj}) \quad i = 1, 2, \ldots, M \]  

(5)

Such that

\[ \sum_{i=1}^{M} [\hat{f}(\pi_{1j}, \pi_{2j}, \pi_{3j}, \ldots, \pi_{kj}) - \hat{\pi}_{0i}]^2 \to \text{Min} \]  

(6)

In the order to construct such independent dimensionless parameters for modeling of concrete diffusion factor \( D_f \), total water per cement ratio \( (w/c) \) and silica fume percent \( (SF) \) have been considered. From the set of such input-output parameter, \( K=3 \) independent dimensionless parameters can be constructed according to three main dimensionless (M, L, T) as follows

\[ \pi_1 = \log(1/D) \]  

(7-a)

\[ \pi_2 = \frac{W}{C} \]  

(7-b)

\[ \pi_3 = 1 - \frac{SF}{100} \]  

(7-c)

So that,

\[ \pi_1 = f(\pi_2, \pi_3). \]  

(8)
In order to use SVD to obtain the model, equation (8) can be represented as

$$\pi_i = C(\pi_2)^a (\pi_3)^b$$  \hspace{1cm} (9)

Therefore, the problem of modeling is now to find coefficients $C$, $\alpha$ and $\beta$ so that equation (6) is satisfied. By using natural logarithm, equation (9) can be represented as a linear relation with respect to the coefficients $(\eta = \text{Ln}C)$, $(\alpha)$ and $(\beta)$ as

$$\text{Ln}(\pi_i) = \eta + \alpha \text{Ln}(\pi_2) + \beta \text{Ln}(\pi_3)$$  \hspace{1cm} (10)

Consequently, a system of $M$ Linear algebraic equation with $K=3$ unknown of the above mentioned coefficients is now constructed based on $M$ input-output experimental data pairs as follows

$$\begin{align*}
\eta + \alpha \zeta_{11} + \beta \zeta_{12} &= \zeta_{10} \\
\eta + \alpha \zeta_{21} + \beta \zeta_{22} &= \zeta_{20} \\
&\vdots \\
\eta + \alpha \zeta_{M1} + \beta \zeta_{M2} &= \zeta_{M0}
\end{align*}$$  \hspace{1cm} (11)

where

$$\zeta_{ij} = \text{Ln}(\pi_{ij}) \cdot i = 1,2,\ldots, M \quad j = 1,2$$  \hspace{1cm} (12)

and

$$\zeta_{i0} = \text{Ln}(\pi_{i0}) \cdot i = 1,2,\ldots, M$$  \hspace{1cm} (13)

Such system of linear equations in which $M>>K=3$ can be represented as

$$AX = Y,$$  \hspace{1cm} (14)

Where

$$X = [\eta \quad \alpha \quad \beta]^T,$$  \hspace{1cm} (15)

$$Y = [\zeta_{10} \quad \zeta_{20} \quad \ldots \quad \zeta_{M0}]^T,$$  \hspace{1cm} (16)

And

$$A = \begin{bmatrix}
1 & \zeta_{11} & \zeta_{12} \\
1 & \zeta_{21} & \zeta_{22} \\
\vdots & \vdots & \vdots \\
1 & \zeta_{M1} & \zeta_{M2}
\end{bmatrix}$$  \hspace{1cm} (17)
The least-squares technique from multiple-regression analysis leads to the solution of the normal equation in the form of

$$X = (A^T A)^{-1} A^T Y,$$  

(18)

Which determines the vector of the best \( k = 3 \) unknown of equation (9) for the whole set of \( M \) experimental observation data. However, such solution directly form solving normal equations (18) is rather susceptible to round off error and, more importantly, to the possible singularity of these equations. Therefore, SVD is used to solve equation (14) which leads to better results in comparison with those of using equation (18).

SVD is the method for solving most linear least-squares problems that some singularities may exist in the normal equations. The SVD of a matrix, \( A \in \mathbb{R}^{M \times K} \), is a factorization of the matrix into the product of three matrices, column orthogonal matrix \( U \in \mathbb{R}^{M \times M} \), diagonal matrix \( W \in \mathbb{R}^{K \times K} \) with non-negative elements (singular values), and orthogonal matrix \( V \in \mathbb{R}^{K \times K} \) such that

$$A = U W V^T$$

(19)

The most popular technique for computing the SVD was originally proposed in [9]. The problem of optimal selection of vector of the coefficients in equation (15) and (18) is firstly reduced to the modified inversion of diagonal matrix \( W \) [10] in which the reciprocals of zero or near zero singulars (according to a threshold) are set to zero. Then, such optimal \( X \) are obtained using the following relation

$$X = V \begin{bmatrix} \text{diag} \left( \frac{1}{w_j} \right) \end{bmatrix} U^T Y$$

(20)

In order to demonstrate the prediction ability of SVD in such dimensionless modeling, the data have been divided into two different sets, namely, training and testing sets. The training set, which consists of randomly chosen \( N_t \) input-output data pairs, is used for training the \( K = 3 \) unknown coefficients involved in the dimensionless model of concrete diffusion factor. The testing set, which consists of \( N_p \) unforeseen input-output data samples during the training process, is merely used for testing to show the prediction ability of the obtained simple model.

4. RESULTS AND COMPARISONS

In order to obtain a simple model for concrete diffusion factor under aggressive environment (equation (9)), the experimental data of shekarchi [7] described in section of experimental procedure is now converted into a dimensionless data table based on definitions (7-a)-(7-d) and their natural logarithms equations (12)-(13).
The unknown $K = 3$ coefficient involved in the simple model representing by equation (9) can now be determined by either solving normal equation (SNE) by pseudo-inverse of matrix $A$ given by equation (18) or by SVD approach proposed in this work given by equation (20). Such approach is accomplished by randomly selecting $\hat{N}_i$ data pairs out of total $N=20$ data pairs. The remaining $N_p = N - \hat{N}_i$ data pair is used to show the prediction ability of the obtained simple model in the form of equation (9).

In order to obtain the best possible model (as the amount of $\hat{N}_i$ can vary between $N_i = K = 3$ to $N_i = N$), series of runs in which $N_i$ and $N_p$ vary between (3 to N) and (N to 3), respectively, have been performed. The values of root mean squares of errors (RMSE) obtained using SVD and SNE are 0.959353 and 1.37529, respectively, which demonstrates the superiority of SVD over SNE. In this paper, $N_i = 5$ and $N_p = 15$ has been chosen to represent the simple dimensionless concrete diffusion factor in Persian gulf environment. The corresponding values of parameters are found as $C = 10.628$, $\alpha = -4.353519E - 02$, $\beta = -3.952748E - 01$ Hence, the model can now be given as

$$\log(1/D) = 10.628(w/c)^{-4.353519E-02}(1 - sf/100)^{-3.952748E-01}$$  \hspace{1cm} (21)

Figure (3) shows the comparison of $(\log 1/D)$ given by equation (21) with respect to the experimental values both for training and testing data sets. It is evident from this figure that equation (21) predicts the midpoint concrete diffusion factor successfully for the testing data.

![Figure 3. Variation of concrete diffusion factor using the simplified model (equation 21) in comparison with the experimental values](image)

However, the obtained model given by equation (21) can be further simplified in order to compare with some other models in world.
5. CONCLUSION

Singular value decomposition and dimensionless analysis have been used to model the concrete diffusion under aggressive environment using some experimental input-output data. It has been shown that the simple obtained model can successfully predict the concrete diffusion factor compared with the actual experimental values. The methodology of this paper can be readily applied to find simple closed-form equations of complex real-world processes where some experimental input-output data pairs are available.

REFERENCES

EVALUATION OF MECHANICAL PROPERTIES AND DURABILITY OF CONCRETES CONTAINING RICE HUSK ASH

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\textsuperscript{2}M.Sc. Student, Dept. of Civil Engineering School of Engineering, Amirkabir University of Technology, Tehran, Iran
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ABSTRACT

Replacement of cement with pozzolan in the production of concrete not only improves the mechanical properties and durability of concrete but also decreases the amount of consumed cement in construction projects as well. For many decades, the concretes incorporating Rice Husk Ash (RHA) as an artificial Pozzolan have been noticed for its qualities and properties. The ash remaining from burning rice husk with high specific surface, decreases porosity and permeability and increases durability of concrete as a result of considerable pozzolanic activities and chemical activities with Calcium Hydroxide.

In this paper, in order to supply typical RHA, a special furnace was designed and constructed in Amirkabir University of Technology. XRD and XRF techniques were used to determine the amorphous silica content of the burnt rice husk. Consequently, temperature of 650 degrees centigrade and 60 minutes burning time was found to be the best combination.

Then, various experiments were carried out to determine properties of concretes incorporating optimum RHA. The results show that RHA as an artificial pozzolanic material has increased the strength and reduced chloride permeability leading to higher durability. As an example, the tensile strength increased up to 13\% after 28 days and up to 23\% at 90 days.

Keywords: RHA, durability, special furnace, mechanical properties, RCPT

1. INTRODUCTION

Sustainable development of the cement and concrete industry requires the utilization of industrial and agricultural waste components. At present, for a variety of reasons, the concrete construction industry is not sustainable. Firstly, it consumes huge quantities of virgin materials which can remain for next generations. Secondly, the principal binder in concrete is Portland cement, the production of which is a major contributor to greenhouse gas emissions that are implicated in global warming and climate change. Thirdly, many concrete structures suffer from lack of durability which may waste the natural resources. So,
finding a solution to substitute a practical recycled product for part of the cement seems to be desirable for sustainable development. [1-7]

Recycling of waste components contributes to energy savings in cement production, to conservation of natural resources, and in protection of the environment. Furthermore, the use of certain components with potentially pozzolanic reactivity can significantly improve the properties of concrete [8-14].

One of the most suitable sources of pozzolanic material among agricultural waste components is rice husk, as it is available in large quantities and contains a relatively large amount of silica. When rice husk is burnt, about 20% by weight of the husk is recovered as ash in which more than 75% by weight is silica. Unlike natural pozzolan, the ash is an annually renewable source of silica. It is worth mentioning that the use of RHA in concrete may lead to the improved workability, the reduced heat evolution, the reduced permeability, and the increased strength at longer ages. [15-21]

In Iran, rice production has increased during these years, becoming the most important crop. Rice husks are residue produced in significant quantities. While in some regions, they are utilized as a fuel in the rice paddy milling process, in our country they are treated as waste, causing pollution of environment and disposal problems. Due to increasing environmental concern, and the need to preserve energy and resources, efforts have been made to burn the husks under controlled conditions and to utilize the resultant ash as a building material. In addition, rice husks are able to be an ideal fuel for electricity generation [11-14].

The use of Rice Husk Ash (RHA) in concrete was patented in the year 1924 [14]. Up to 1978, all the researches were concentrated to utilize ash derived from uncontrolled combustion. Mehta published several papers dealing with rice husk ash utilization during this period. He established that burning rice husk under controlled temperature-time conditions produces ash containing silica in amorphous form [22-26].

Depending on produce method, the utilization of rice husk ash as a pozzolanic material in cement and concrete provides several advantages, such as improved strength and durability properties. Rodríguez de Sensale [16] reported that mortars and concrete containing RHA have compressive strength values inferior or superior to that of OPC concrete. In addition, in most of the cases [18, 19, 26], mortars and concrete containing RHA improves durability of concrete at various ages.

Generally, there are two types of RHA in concrete. The type of RHA which is suitable for pozzolanic activity is amorphous rather than crystalline. Therefore, substantial research has been carried out on producing RHA containing high amount of amorphous silica. The results have shown that RHA quality depends on temperature and burning time.

In fact, for an incinerator temperature up to 700°C the silica is in amorphous form and silica crystals grew with time of incineration. The combustion environment also affects specific surface area, so that time, temperature and environment also must be considered in the processing of rice husks to produce ash of maximum reactivity [5, 7].
2. MATERIALS USED

The following materials were used in the preparation of the concrete specimens. Local natural sand according to ASTM Standard with maximum aggregate size of 4.75 mm; Crushed granite according to ASTM Standard with maximum aggregate size of 19 mm; Tehran potable water, Type I Portland cement and homogeneous rice husk ash produced by the special designed furnace at 650°C and 60 minutes burning time. Table 1 shows the physical and chemical characteristics of RHA (RHA-650-60) and cement.

<table>
<thead>
<tr>
<th>Physical Tests</th>
<th>Chemical Analyses, (%)</th>
<th>Bogue Composition, (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific Gravity</td>
<td>Blaine, (cm²/gram)</td>
<td>SiO₂</td>
</tr>
<tr>
<td>RHA</td>
<td>2.15</td>
<td>3600</td>
</tr>
<tr>
<td>Cement</td>
<td>3.21</td>
<td>3200</td>
</tr>
</tbody>
</table>

3. TEST METHODS

A total of 4 concrete mixtures were made; one corresponding to a control concrete (CTL) and three others with 7%, 10% and 15% RHA replaced with cement by weight. Table 2 lists the mix proportions of concrete. Slumps were kept constant at 70 ± 10 mm. Superplasticizer with polycarboxylate base was used at very low percentages according to the results obtained for the slumps. Concrete test specimens were compacted by external vibration and kept protected after casting to avoid water evaporation. After 24 hr. they were demolded and cured in lime-saturated water at 23 ± 2°C to prevent possible leaching of Ca (OH) 2 from these specimens. Concrete cubes of 100×100×100mm dimension were cast for compressive strength and water penetration tests. The results obtained are reported as an average of two tests. While two 150×300 mm cylinder concrete specimens were prepared for the tensile strength test and static modulus of elasticity, samples of rapid chloride permeability tests (RCPT), according to ASTM C 1202, were prepared by cutting and discarding 25mm slices from the top and bottom of 100×200 mm cylinders, and the remaining section cut into three 50mm thick slices. The water permeability test was conducted using a high-pressure permeability cell. The specimens used were cubes of 150×150×150 mm dimension. In addition, 50×50×50 mm mortar samples were prepared for the pozzolanic activity test. All specimens were moist cured until the time of testing.

| Table 1: Physical and chemical characteristics of cement and RHA |
|-----------------------------|--------|--------|--------|--------|--------|--------|--------|--------|--------|--------|--------|--------|--------|
| Aggregate (kg/m³) | RHA (kg/m³) | cement (kg/m³) | Aggregate (kg/m³) | SP/cement (%) | water/cement |
| 7%RHA | 29.4 | 420 | 815 | 995 | 0 | 0.45 |
| 10%RHA | 42 | 390.6 | 815 | 995 | 0.15 | 0.45 |
| 15%RHA | 63 | 378 | 815 | 995 | 0.25 | 0.45 |
4. TEST RESULTS

The results of pozzolanic activity test are shown in Table 3. Results demonstrate high pozzolanic activity index of RHA over that of the control in accordance with ASTM C-311/ASTM C-618 test method. On the other hand, produced rice husk ash is a high reactive pozzolanic material, and entirely satisfies other requirements. Figure 1 shows XRD patterns of the ash.

Table 3: Comparison in chemical and physical specifications of produced RHA with ASTM standard C618-03

<table>
<thead>
<tr>
<th></th>
<th>ASTM</th>
<th>RHA results</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chemical Requirements</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SiO₂ + Al₂O₃ + Fe₂O₃, min., %</td>
<td>70</td>
<td>89.9</td>
</tr>
<tr>
<td>SO₃, max., %</td>
<td>4</td>
<td>0.15</td>
</tr>
<tr>
<td>Moisture Content, max., %</td>
<td>3</td>
<td>0.23</td>
</tr>
<tr>
<td>Loss On Ignition (LOI), max., %</td>
<td>6</td>
<td>5.9</td>
</tr>
<tr>
<td>Physical Requirements</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fineness: Amount retained when wet-sieved on 45 µm sieve, max., %</td>
<td>34</td>
<td>8</td>
</tr>
<tr>
<td>Strength Activity Index (20% RHA) at 3-day, min. % control</td>
<td>---</td>
<td>102</td>
</tr>
<tr>
<td>Strength Activity Index (20% RHA) at 7-day, min. % control</td>
<td>75</td>
<td>106</td>
</tr>
<tr>
<td>Strength Activity Index (20% RHA) at 28-day, min. % control</td>
<td>75</td>
<td>110</td>
</tr>
</tbody>
</table>

Results of the compressive strengths of concretes are given in Figure 2. In general, the RHA concrete had higher compressive strengths at various ages and up to 90 days when compared with the control concrete. The results show that it was possible to obtain a compressive strength of as high as 46.9 MPa after 28 days. In addition, strengths up to 63.2 MPa were obtained at 90 days.

Figure 3 shows that concrete containing RHA has a greater splitting tensile strength than that of the control concrete at all ages. It is clear that, as the amount of RHA increases, the tensile strength increases up to 20%. For instance, at 90 days the 15%RHA concrete had a compressive strength of 5.62 MPa compared with 4.58 MPa for the control concrete.

Figure 4 shows the static modulus of elasticity in compression of concrete mixed with different proportions of RHA at 28 and 90 days. After 90 days, mixture
containing 15% of RHA showed 7% increase in static modulus of elasticity in compression as compared to the control concrete. On the other hand, concrete containing RHA depicts a higher static modulus of elasticity when compared to the control concrete.

Results of the rapid determination of chloride permeability of concrete test (Figure 5) show that using RHA drastically enhances resistance to chloride penetration.
compared to control concrete on average, around 4–5 times higher for the 15% RHA. At 7 days, the control concrete showed the highest value of 6189 coulombs while the charge passed through the 15%RHA concrete was 1749 coulombs. With a continuous moist-curing of up to 91 days, the charge passed through all concretes was reduced. The charge for the 15%RHA concrete was reduced to 576 coulombs, which was well below that of the control concrete (2563 coulombs). According to ASTM C 1202, when the charge passed through concrete is below 1000 coulombs, it is categorized as a very high resistance concrete to chloride ion penetration.

The chloride permeability of the concrete specimens incorporating 15%RHA was “very low”, while that of the concrete specimens with 0%, 7%, 10% RHA were “moderate”, “low” and “low”, respectively, as per ASTM C 1202 criteria.

In addition to RCPT, investigations of water permeability were carried out. In this test, water was forced into the concrete samples from one side for three days and under constant pressure of 0.5 MPa. Then, the samples were split in a plane parallel to the direction of water penetration, and the greatest depth of water penetration into the concrete sample was measured. The depth of water penetration of concrete incorporating RHA specimens is shown in Figure 6. As expected, depth of water penetration of concrete specimens decreased significantly with an increase in RHA content and curing period.

Figure 7 gives a linear relationship between compressive strength and tensile strength, it is significant. A good relationship was also found between RCPT and compressive strength, which is shown in Figure 8.

![Figure 6. Depth of water penetration (mm) at various ages for control (CTL) & RHA mixtures](image-url)
5. DISCUSSION

Improvements in mechanical and durability properties of the concretes containing RHA can be explained by the chemical and physical effects of RHA. Chemical effect is mainly due to the pozzolanic reactions between the amorphous silica of RHA and calcium hydroxide (C-H) produced by the cement hydration to form calcium-silicate-hydrates (C-S-H). The physical effect which can also be considered as filler effect is that RHA particles increase the packing of the solid
materials by filling the spaces between the cement grains in much the same way as cement fills the spaces between fine aggregates, and fine aggregates fill the spaces between coarse aggregates in concrete. Moreover, small particles of additions generate a large number of nucleation sites for the precipitation of the hydration products. This will accelerate the reactions and form smaller C-H crystals. RHA reduces the number of large pores and increases the probability of transforming the continuous pores into discontinuous ones. Therefore, all these mechanisms make the microstructure of the paste more homogeneous and denser.

6. CONCLUSIONS
Based on the results of the present experiments, the following conclusions can be drawn out:

1) The quality of the RHA cement is widely varied due to the differences in the methods of production. So, it is generally advocated to use special incinerators, which can guarantee controlled burning conditions. With the proper production method, rice husk ash of a pozzolanic reactivity comparable to other pozzolans can be obtained. A special furnace which was designed and constructed was able to produce RHA with various qualities.

2) The duration and temperature of furnace are important parameters, influencing the reactivity of RHA pozzolans. Silica in the rice husk initially exists in the amorphous form, but may become crystalline when rice husk is burnt at high temperature. In addition, silica in rice husk ash will not remain porous and amorphous, when combusted for a prolonged period at a temperature above 650°C, or during less than a few minutes at 1100°C, under oxidizing conditions. The results of XRD analysis show that quartz crystal is present in both types of ashes. So, investigation on the influence of combustion conditions on the amorphous silica suggests that the RHA-650-60 can be considered to be non-crystalline RHA and to save the RHA production time.

3) Huge amounts of crystalline silica or higher carbon content are detrimental to the pozzolanic reactivity of the ash. Presence of un-burnt carbon can adversely affect the reactivity even though it is rich in amorphous silica. The results of pozzolanic activity demonstrate high pozzolanic activity index of produced rice husk ash concrete over that of the control. In addition, the produced rice husk ashes containing up to 90 percent amorphous silica entirely satisfy other requirements of ASTM standard C618-03. This shows the high quality of produced rice husk ashes.

4) The RHA concrete showed higher compressive strength at various ages in comparison with that of the concrete without RHA. In addition, the RHA concrete had higher splitting tensile strength and modulus of elasticity in comparison with that of the concrete without RHA. It is concluded that produced RHA provides a positive effect on the compressive strength of concretes.

5) The performance of concrete with cement replacement by RHA is outstanding considering resistance to water and chloride ion penetration which is in many
cases the most important characteristic concerning durability and corrosion prevention.

REFERENCES
INFLUENCE OF WATERPROOFING ADMIXTURE IN WATER PENETRATION OF CONCRETE

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1Power and Water University of Technology, faculty member
2Power and Water University of Technology, faculty member

ABSTRACT
Nowadays, concrete plays an outstanding role in construction industry and variety of engineering infrastructures such as bridges, dams, jetties, piers and channels which are made of concrete. Since from one hand, these huge and critical concrete structures in which some cases are of lifelines of a country are of great importance and on the other hand, the necessity of conservation and durability of these structures is inevitable, the importance of concrete durability will become vividly apparent. One of the factors that can deteriorate the reinforced concrete condition by corrosion and cause cracking in severe cold weather, has turned out to be water penetration into concrete. With this respect, one of the practical and efficient ways ever used to countermeasure this situation is to use waterproof additives in concrete mix in order to reduce water and waste water absorption and penetration into concrete. In this investigation, PN gel, PN liquid and PN liquid along with micro silica powder as waterproofing additives, were used in separate mix designs to make specimens and then these specimens and the witness were tested by two methods in order to determine water absorption and penetration depth in concrete. In one of these two methods, water absorption percentage was just determined and in the other one which was more accurate and was implemented by triaxial apparatus, both water absorption and penetration depth in the specimens were investigated. Finally, the tables and diagrams for gained results concerning specimens containing PN additives and witness were prepared and compared. The results showed about more than 50% improvement in water penetration reduction.

Keywords: concrete, durability, waterproof additives, penetration depth, water absorption

1. INTRODUCTION
In the present research, the methods and results of water absorption and penetration depth tests are presented which have been carried out for four groups of specimens including waterproofing additives PN gel, PN liquid, PN liquid along with micro silica powder. These additives are produced by "Vand Chemie Sakhteman" company, and the tests have been implemented on 28-day concrete specimens in Material and Concrete Technical Unit laboratory of Power & Water University of Technology (PWUT).

General characteristics of materials used in concrete mix designs are presented in
the following tables:

Table 1: Materials used in mix designs

<table>
<thead>
<tr>
<th>Materials</th>
<th>Characteristics</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>Tehran Portland cement Type I</td>
</tr>
<tr>
<td>Aggregates</td>
<td>Materials and Concrete Technical Unit lab</td>
</tr>
<tr>
<td>Water</td>
<td>Tehran drinking water</td>
</tr>
<tr>
<td>Water proofing gel</td>
<td>PN additives (produced by 'van shimi' Co.)</td>
</tr>
<tr>
<td>Water proofing liquid</td>
<td>PN additives (produced by 'van shimi' Co.)</td>
</tr>
<tr>
<td>Micro silica</td>
<td>Powder (produced by 'van shimi' Co.)</td>
</tr>
</tbody>
</table>

Table 2: Mix design characteristics of specimens

<table>
<thead>
<tr>
<th>Group NO.</th>
<th>Mix design</th>
<th>w/c ratio</th>
<th>Slump no. (cm)</th>
<th>Cement content (kg/m³)</th>
<th>Aggregate content (kg/m³)</th>
<th>Coarse aggregates (kg/m³)</th>
<th>Fine aggregates (kg/m³)</th>
<th>Weight ratio of additives to cement (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>witness</td>
<td>0.63</td>
<td>5</td>
<td>350</td>
<td>945</td>
<td>473</td>
<td>473</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>PN gel</td>
<td>0.63</td>
<td>16</td>
<td>350</td>
<td>945</td>
<td>473</td>
<td>473</td>
<td>7</td>
</tr>
<tr>
<td>3</td>
<td>PN liquid</td>
<td>0.63</td>
<td>17</td>
<td>350</td>
<td>945</td>
<td>473</td>
<td>473</td>
<td>0.5</td>
</tr>
<tr>
<td>4</td>
<td>PN liquid + microsilica powder</td>
<td>0.63</td>
<td>16</td>
<td>350</td>
<td>945</td>
<td>473</td>
<td>473</td>
<td>0.5 +10</td>
</tr>
</tbody>
</table>

2. DEFINITIONS AND CALCULATIONS FORMULAS
1. Saturated with dried surface weight or SSD (Saturated Surface Dry):
   Weight of aggregates in saturated state with dried surface or weight of concrete specimen in saturated state with dried surface.
2. Water absorption:
   Weight difference ratio for two states as SSD (Saturated Surface Dry) and fully dried (heated in the oven for at least 48 hours in 110°C) to fully dried specimen weight described in percentage
3. Dry weight:
   Specimen weight after being heated in oven for 48 hours at 110°C.

2.1. Construction and Test Procedure
2.1.1. Specimens construction
   According to the mix design prepared as witness, at first aggregates and half of the mix water were poured into the mixer and then cement and the rest of the water were added into the mixer. The slump was tested in order to reach the specified slump to the amount of 5 cm. therefore, water to cement ratio was determined as 0.63 and this ratio was used in other mix designs of the specimens. The molds were opened 24 to 26 hours after placing the concrete.[1]

2.1.2. Construction standards
   Steps of making and curing of the specimens have been done as presented in the
The following table:

**Table 3: Standards of making the concrete specimens**

<table>
<thead>
<tr>
<th>Row No.</th>
<th>Description</th>
<th>ABA standard (Iran concrete standard)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Making concrete specimens for concrete tests</td>
<td>503ت.د.</td>
</tr>
<tr>
<td>2</td>
<td>Curing the specimens in the laboratory</td>
<td>503ت.د.</td>
</tr>
<tr>
<td>3</td>
<td>Determination of concrete workability (slump)</td>
<td>505ت.د.</td>
</tr>
<tr>
<td>4</td>
<td>Determination of fresh concrete unit weight</td>
<td>509ت.د.</td>
</tr>
</tbody>
</table>

2.2. Curing Condition of the Specimens
The specimens were kept in water with the temperature of 23°C until the age of 28 days.

![Figure 1. Water pool where the specimens were kept](image)

2.3. Implementation of Penetration Test
In order to do the penetration test by triaxial apparatus (Figures. 2-1 to 2-4), since the maximum height of the apparatus was less than 20 cm, the specimens were cut 2 cm in length and their heights reached 18 cm.

![Figure 2.1. General view of triaxial apparatus](image)

![Figure 2.2. Digital sensor for axial force](image)
2.4. Sealing the Specimens and Test Procedure Selection

Since the water must be just entered from top surface of the specimen, its lateral sides ought to be completely sealed. For this purpose, an elastic cover was used and all of the seams were sealed with silicon glue and the cover was fully attached to the top and bottom supports by special elastic rings. Then the confining pressure which was higher than injection pressure was applied to the specimen and sealed it completely (Figure 3).

Then the witness specimen kept in the oven for 48 hours at 110°C was placed in the triaxial apparatus and injection and confining pressures were applied as much as 490Kpa (4.9 bar) and 500Kpa (5 bar), respectively. With these preparations, the time period which lasted until the water reached the other side of the specimen could be determined. Since in the period of one hour, no water output considered, this time period was used as a criterion for time period test of the specimens and this procedure was finalized after two series of tests. [2]
2.5. Test Standard of Water Penetration Depth
According to implemented steps, standard method for the test is summarized as following:
- First all of the specimens should be kept in the oven for 48 hours at 110°C.
- Then the specimens should be placed into the triaxial apparatus for 60 minutes and under injection and confining pressures equal to 490Kpa and 500Kpa, respectively.
- Finally the specimens are brought out of the apparatus and broken in Brazilian way into two pieces. By considering the wet part of the half of the specimen, the wet distance measured from top surface of the specimen can be determined as water penetration depth.

It should be mentioned that the final penetration depth has been determined as average of five readings of middle, 1/2 from sides and 1/4 from sides of diameter of the specimens.

2.6. Characteristics of Test Specimens
2.6.1. Characteristics of Fresh Concrete
Characteristics of fresh concrete such as slump, density and ambient temperature are listed in Table (4).

<table>
<thead>
<tr>
<th>Mixture design</th>
<th>Ambient temperature (°C)</th>
<th>Slump (cm)</th>
<th>Fresh concrete density (gr/cm³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Witness</td>
<td>15</td>
<td>5</td>
<td>2.5</td>
</tr>
<tr>
<td>PN gel</td>
<td>15</td>
<td>16</td>
<td>2.53</td>
</tr>
<tr>
<td>PN liquid</td>
<td>15</td>
<td>17</td>
<td>2.5</td>
</tr>
<tr>
<td>PN liquid + micro silica powder</td>
<td>15</td>
<td>16</td>
<td>2.54</td>
</tr>
</tbody>
</table>

It can be concluded from the above table that slump variation of the concrete containing water proof materials is considerable in comparison to the witness specimen. Figure (4) demonstrates these variations properly. This vividly reveals the improvement of concrete workability.
2.7. Characteristics of Hardened Concrete

2.7.1. Water absorption

Two procedures were used to measure water absorption of the specimens. The first one was mentioned in section 1-1 and the second one was in such a way that having the specimens placed in the oven for 48 hours at 110 °C and let them to be cooled in the air for two hours, they were placed in water for two hours and then water absorption of the specimens were measured.

The results of both methods are presented in the table below:

### Table 5: Characteristics of hardened concrete and water absorption (first method-SSD)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Age (days)</th>
<th>SSD Weight (gr)</th>
<th>Dry weight 48hours (gr)</th>
<th>Specimen dimensions (cm)</th>
<th>Volume (Cm³)</th>
<th>SSD density (gr/cm³)</th>
<th>Dry density</th>
<th>Water absorption (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>28</td>
<td>3538</td>
<td>3244</td>
<td>10 18</td>
<td>1413.7</td>
<td>2.5</td>
<td>2.29</td>
<td>9.06</td>
</tr>
<tr>
<td>2</td>
<td>28</td>
<td>3480</td>
<td>3226</td>
<td>10 18</td>
<td>1413.7</td>
<td>2.46</td>
<td>2.28</td>
<td>7.87</td>
</tr>
<tr>
<td>3</td>
<td>28</td>
<td>3954</td>
<td>3686</td>
<td>10 20</td>
<td>1570.8</td>
<td>2.52</td>
<td>2.35</td>
<td>7.27</td>
</tr>
<tr>
<td>4</td>
<td>28</td>
<td>3835</td>
<td>3586</td>
<td>10 20</td>
<td>1570.8</td>
<td>2.44</td>
<td>2.28</td>
<td>6.94</td>
</tr>
</tbody>
</table>

Calculation results can be seen in figure (5).

### Figure 5. Water absorption comparison of the specimens in the first method

### Table 6: Characteristics of hardened concrete and water absorption in second method

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Age (days)</th>
<th>Wet Weight (gr)</th>
<th>Dry weight 48hours (gr)</th>
<th>Specimen dimensions (cm)</th>
<th>Volume (Cm³)</th>
<th>wet density (gr/cm³)</th>
<th>Dry density</th>
<th>Water absorption (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>28</td>
<td>3428</td>
<td>3244</td>
<td>10 18</td>
<td>1413.7</td>
<td>2.42</td>
<td>2.29</td>
<td>5.67</td>
</tr>
<tr>
<td>2</td>
<td>28</td>
<td>3370</td>
<td>3226</td>
<td>10 18</td>
<td>1413.7</td>
<td>2.4</td>
<td>2.28</td>
<td>4.46</td>
</tr>
<tr>
<td>3</td>
<td>28</td>
<td>3843</td>
<td>3686</td>
<td>10 20</td>
<td>1570.8</td>
<td>2.45</td>
<td>2.35</td>
<td>4.26</td>
</tr>
<tr>
<td>4</td>
<td>28</td>
<td>3701</td>
<td>3586</td>
<td>10 20</td>
<td>1570.8</td>
<td>2.36</td>
<td>2.28</td>
<td>3.21</td>
</tr>
</tbody>
</table>
Water absorption variations in second method are demonstrated in Figure (6).

Figure 6. Water absorption comparison of the specimens in the second method

2.8. Water Penetration Depth
In order to start the test, the specimens were placed in standard conditions and tests were carried out for 4 specimens and the results are presented in the table and figure below:

Table 7: Results of water penetration depth tests

<table>
<thead>
<tr>
<th>Row No.</th>
<th>Specimen</th>
<th>Age (days)</th>
<th>Specimen dimensions (cm)</th>
<th>Water penetration depth (cm)</th>
<th>Average (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>witness 2</td>
<td>28</td>
<td>10</td>
<td>15 14 13 14.5 15</td>
<td>14.3</td>
</tr>
<tr>
<td>2</td>
<td>PN gel 2</td>
<td>28</td>
<td>10</td>
<td>18 * 3.8 4.9 5.1 * 4.6</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>PN liquid 2</td>
<td>28</td>
<td>10</td>
<td>18 6 5.5 6 5.5 6</td>
<td>5.6</td>
</tr>
<tr>
<td>4</td>
<td>PN liquid + Ms 2</td>
<td>28</td>
<td>10</td>
<td>18 5.1 4.5 4 4 5</td>
<td>4.52</td>
</tr>
</tbody>
</table>

* since in the specimens containing PN gel, a small part of upper surface has become water resistant by the glue, therefore the values for the sides were invalid and measurements were made at three middle points.

Figure 7. Water penetration depth comparison based on the mentioned standard
3. CONCLUSION
All of the obtained results for different tests on 28-day specimens have been listed in the following table. In this table, water absorption and penetration depth improvements for two methods are presented.

The results can be interpreted as following:
- Using PN water resistant additives, penetration depth can be reduced by 60% in average.
- Based on the fist method, concrete water absorption decreases by 13% to 23%.
- Based on the second method, concrete water absorption decreases by 21% to 43%.
- Using PN water resistant additives, the workability of concrete increased considerably, i.e. the slump value increased three times compared to the witness.
Table 8: Results add up and Comparison

<table>
<thead>
<tr>
<th>Row NO.</th>
<th>Test type</th>
<th>Specimen age (days)</th>
<th>Improvement percentage</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Water penetration depth in witness specimen</td>
<td>28</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>Water penetration depth in the specimen containing PN gel</td>
<td>28</td>
<td>67.83</td>
</tr>
<tr>
<td>3</td>
<td>Water penetration depth in the specimen containing PN liquid</td>
<td>28</td>
<td>60.84</td>
</tr>
<tr>
<td>4</td>
<td>Water penetration depth in the specimen containing PN liquid + Ms</td>
<td>28</td>
<td>68.39</td>
</tr>
<tr>
<td></td>
<td>Water absorption percentage (first method)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Water absorption depth in witness specimen</td>
<td>28</td>
<td>-</td>
</tr>
<tr>
<td>6</td>
<td>Water absorption depth in the specimen containing PN gel</td>
<td>28</td>
<td>13.13</td>
</tr>
<tr>
<td>7</td>
<td>Water absorption depth in the specimen containing PN liquid</td>
<td>28</td>
<td>19.76</td>
</tr>
<tr>
<td>8</td>
<td>Water absorption depth in the specimen containing PN liquid + Ms</td>
<td>28</td>
<td>23.4</td>
</tr>
<tr>
<td></td>
<td>Water absorption percentage (second method)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>Water absorption depth in witness specimen</td>
<td>28</td>
<td>-</td>
</tr>
<tr>
<td>10</td>
<td>Water absorption depth in the specimen containing PN gel</td>
<td>28</td>
<td>21.34</td>
</tr>
<tr>
<td>11</td>
<td>Water absorption depth in the specimen containing PN liquid</td>
<td>28</td>
<td>24.87</td>
</tr>
<tr>
<td>12</td>
<td>Water absorption depth in the specimen containing PN liquid + Ms</td>
<td>28</td>
<td>43.39</td>
</tr>
</tbody>
</table>

REFERENCES
APPLICATION OF GENETIC ALGORITHM IN OPTIMIZATION OF FIBERS DIRECTION IN COMPOSITE FRP JACKETS FOR LATERAL RETROFITTING OF RC COLUMNS

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ABSTRACT
In the recent years, many researches have been implemented in the field of composite fibers in concrete. One of the effective and efficient methods of rehabilitation and strengthening of RC members is application of composite materials and FRP jackets and laminates. These laminates are made in different ways with respect to the direction and orientation of fibers forming the layers. Generally 1D, 2D or 3D layers are used for this purpose for which the direction of fibers varies for different types. Selection of the best and optimized direction of fibers is of great importance to fully utilize the strength capacity of composite fibers and the layers.

In this paper, genetic algorithm is considered to be used to find the optimized direction of fibers in composite layers. In this way, different angles of fibers in the layer are tested by genetic algorithm to so that the best performance of the composite layers is obtained. It should be considered that by this optimization, lateral resistance of concrete members wrapped by composite layers can be improved.

Keywords: genetic algorithm, fibers direction, composite fibers, optimization, FRP jackets

1. INTRODUCTION
As a result of development of composite materials technology and their cost decrease during last three decades, application of such materials as a good replacement of metallic alloys has been promoted. Generally, a composite is combination of at least two materials which are combined to produce desirable characteristics. In practice, most of composites contain a base material and a strengthening element which is added to the base material to increase its strength. The most common composite materials are those composed of a series of strong fibers tightly packed together by means of a bonding material. In fact, the bonding material acts as a paste or matrix and let the stress transfer from one fiber to another so that a uniform and integrated formation is produced.
Composites offer variety and combinations of characteristics which are not available in traditional materials. It is possible for the fibers in high stress areas to
be in the position, orientation and volume so that maximize the efficiency and functionality and again in another area of low stress in the same member, the characteristics may be changed to satisfy the requirements. Other advantages include: light weight, corrosion resistance, flexibility and desirable functionality in structures.

Fibers form an important element of composite materials. Totally, 30 to 70 percent of composite volume consists of fibers which may be in individual or woven form. Regardless of the geometry, fibers are made of different materials the most common of which in structural applications are glass, carbon and aramid. Also new fibers of high strength have been developed which are known as 'Advanced fibers'. Composites made of such materials are also called 'advanced' composites. Mechanical properties of composites are dependent on different parameters such as fibers type, volume and orientation. These materials are anisotropic and their strength varies in different directions. Their stress-strain curve shows a sudden failure at the yielding point. Although the resin shows visco-elastic behavior under applied loads and bear creep and relaxation, the total composite is design so as to satisfy the needed requirements.

A kind of composites known as FRP are made of different fibers as mentioned earlier such as carbon (CFRP) and glass (GFRP) and also aramid (AFRP). These products are being widely used in different applications such as cylindrical shells and in different forms like jackets and laminates for structures retrofitting. In the latter application, FRP may be used as lateral confinement around the concrete member to enhance its load bearing capacity.

For example in Europe and New Zealand, the seismic philosophy for achieving ductility is different, relying more on increasing the non-linear concrete strains through concrete confinement [3]. The advantage of this philosophy is that apart from smaller cross-sections, a significantly higher amount of lateral reinforcement is required. This lateral reinforcement, which is there to confine the concrete, is also beneficial in resisting additional shear and preventing splice and anchorage failures. The enhancement of concrete ductility by confinement is central to the principles of Eurocode 8. However, researchers dealing with FRP confinement do not always consult the huge wealth of published work, which is derived from the earthquake engineering research.

From Figure 1, it can be easily recognized that lateral confinement has effectively improved the load bearing capacity of the RC columns confined by CFRP and GFRP. Physical and mechanical properties of the composites are presented in the following tables:

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E_{frp}$</td>
<td>Young’s modulus of elasticity</td>
</tr>
<tr>
<td>$f_{frpu}$</td>
<td>Ultimate tensile strength and elongation of pultruded laminate</td>
</tr>
<tr>
<td>$T_{GM}$</td>
<td>Glass transition temperature</td>
</tr>
<tr>
<td>$V_f$</td>
<td>Volumetric fibre content</td>
</tr>
</tbody>
</table>
### Table 1: Physical and Mechanical properties of Fibre sheets [6]

<table>
<thead>
<tr>
<th>Fibre</th>
<th>$E_{frp}$ (GPa)</th>
<th>$f_{frpu}$ (MPa)</th>
<th>$\varepsilon_{frpu}$ (%)</th>
<th>Density (gr/cm$^3$)</th>
<th>$T_G M$ (C°)</th>
<th>$V_f$ (%)</th>
<th>Composition</th>
</tr>
</thead>
<tbody>
<tr>
<td>GFRP AR</td>
<td>65</td>
<td>1700</td>
<td>2.88</td>
<td>2.6</td>
<td>-</td>
<td>100</td>
<td>Bi-Directional</td>
</tr>
<tr>
<td>CFRP 240</td>
<td>240</td>
<td>3900</td>
<td>1.55</td>
<td>1.7</td>
<td>100-130</td>
<td>100</td>
<td>Uni-Directional</td>
</tr>
</tbody>
</table>

### Table 2: Physical and Mechanical properties of Epoxy Plus Structural Adhesive [6]

<table>
<thead>
<tr>
<th>Epoxy</th>
<th>Colour</th>
<th>Strength $E_m$ (GPa)</th>
<th>Density (kg/litre)</th>
<th>$T_{GK}$ (C°)</th>
<th>Manufacturer</th>
<th>England</th>
</tr>
</thead>
<tbody>
<tr>
<td>Adhesive</td>
<td>Mid grey</td>
<td>19</td>
<td>1.535</td>
<td>60</td>
<td>SBD</td>
<td>U.K.</td>
</tr>
<tr>
<td>Primer</td>
<td>Translucent</td>
<td>-</td>
<td>1.12</td>
<td>-</td>
<td>SBD</td>
<td></td>
</tr>
</tbody>
</table>

![Figure 1. Comparison of Ave. longitudinal and lateral strain measured by strain gauges VS. stress](image)

### 2. CLASSIFICATION OF COMPOSITES BASED ON GEOMETRY AND STACKING OF LAYERS

Classification of composites varies depending on geometry and stacking of layers. The basic form is the unidirectional layers with continuous fibers. The one layer may itself be composed of several layers however; all of the fibers are oriented in the same direction, weather in simple or woven form.

Multi layer laminates are made by stacking of unidirectional layers each of which having different fiber orientation. Efficient properties of multi layer laminates vary by fibers orientation, layers thickness and stacking order. Hybrid composites which are used in many applications include more than one kind of fiber or combination of fibers and metallic materials. Different types of fibers are combined in a composite to achieve the best performance and lowest cost. Nevertheless, in many applications one kind of fibers may be used in different layers with various fibers orientation and in order to achieve the best performance, the angle of the fibers should be optimized (Figure 2)
3. GENETIC ALGORITHM (GA) METHOD
Optimization algorithms are divided into two basic types, one based on differential equations and other based on numerical methods. The former one uses gradient-based search with a suitable primary guess. These algorithms search range is local and concerning ill-posed target functions, they are instable and get involved in local optimized points. Many numerical methods have been proposed to overcome the local optimized problem however; these methods increase the calculations volume. Genetic algorithm which belongs to the latter type of optimization algorithms is the most applicable evolutionary algorithms. In spite of many methods, the genetic algorithm utilizes the probabilistic rules instead of deterministic ones for conducting the search process to the more suitable search space.

GA algorithms basically were developed to study intelligent systems nevertheless; they were used from the beginning in optimization of practical engineering problems. GA usually include a population of individuals, fitness function, crossover operator and new generation replacement. Figure 3 shows the cycle of classic GA (CGA) and basic elitist GA (BEGA).

In the GA, first a determined numbers of inputs, which belong to the sample space \( X \) are selected and displayed as a vector, \( X=(x_1, x_2, \ldots x_n) \), which is called 'chromosome'. A group of chromosomes form 'colony' or 'population'. In each step, the population evolves based on determined rules. There is a fitness for each chromosome as \( f(x_i) \). Stronger elements or chromosomes the fitness of which is closer to the optimized of the colony would have more chance to survive in next generations. In other word, inputs closer to the optimized solution will remain and others will be disregarded.

Another important step is the birth which is occurred once in each period. In this process, two suitable chromosomes are combined to generate more optimized chromosome. Furthermore, during each period a series of chromosomes might have mutation.

4. DIFFERENCE BETWEEN GA AND TRADITIONAL OPTIMIZATION METHODS
The difference between GA and other optimization methods may be summarized as follows:
- Despite the other methods, GA uses several search points (as a set of individuals) which are convergent instead of one search point. Therefore, the

![Figure 2. Different fibers orientation in different layers of composite material](image)
The possibility of getting involved in local optimized points decreases significantly and on the other hand, the access probability of the global optimized point will increase.

- GA takes advantage of the data obtained (from fitness function) instead of the derivatives or subordinate data.
- It is applicable to optimization problems with multi-objective functions.
- It is applicable to linear, nonlinear and ill-posed problems.
- It can be combined with other optimization methods.
- It is a powerful algorithm to approach the nearest optimized solution with high probability.

![Diagram of CGA and BEGA cycles](image)

**Figure 3. diagram of CGA and BEGA cycles**
5. THE ROLE OF NEURAL NETWORK IN GA OPTIMIZATION

GA is a powerful method in optimization problems especially multi-objective optimization, however, one of disadvantages of GA is high CPU time. This reason and also stochastic nature of GA in solving the optimization problems make the process of optimization slow down. Using ANN, this problem can be overcome while maintaining the other useful characteristics of GA.

In this study, first the neural network was trained and then tested and the trends obtained from ANN was used in GA to optimize fibers angles and also final value of objective function.

6. SIMPLE OPTIMIZATION OF NATURAL FREQUENCY OF FOUR-LAYER FRP CYLINDRICAL SHELL

Characteristics of the composite cylindrical shell are illustrated in Figure 4.

![Figure 4. Sketch of a Multi-layer composite cylindrical shell](image)

In this study, the ANN trained and tested to be used in GA was one of the back propagation algorithms called Levenberg-Marquardt which had the best results and less errors compared to other algorithms. This four-layer network has 4 neurons in input layer and one neuron in output layer. The characteristics of the network are presented in Table 3.

<table>
<thead>
<tr>
<th>Number of inputs</th>
<th>First layer</th>
<th>Hidden layers</th>
<th>Second layer</th>
<th>Output layer</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Transition function</td>
<td>Number of neurons</td>
<td>Transition function</td>
<td>Number of neurons</td>
</tr>
<tr>
<td>4</td>
<td>Tangent sigmoid</td>
<td>25</td>
<td>Tangent sigmoid</td>
<td>12</td>
</tr>
</tbody>
</table>

In order to assure the appropriate performance of the network, it should be tested
and the training error should be determined. Figure 4 shows MSE curve with respect to the cycles of ANN for modeling of first natural frequency.

![Figure 5. MSE curve with respect to the ANN cycles to train first natural frequency in four-layer shell for h/r=0.01](image)

![Figure 6. Errors of the trained ANN for natural frequency in test space](image)

As can be seen in Figures 4, 5 MSE in the network is around $10^{-8}$ and maximum error in the trained network in test stage is about 2.5% which shows appropriate accuracy of the network.

When the network is tested and its performance is assured, the process of optimization starts. Increase of fitness function of GA using analytical solution and ANN for following state is compared in Figure 7.
According to Figure 7, the best frequency obtained from GA and analytical solution is equal to 7016 (Hz) which was reached after 46 generation and after that no improvement in frequency observed. Angles of layers stacking are as following: 

\[ \theta = [-45^\circ, 46^\circ, 40^\circ, 45^\circ] \]

However, the frequency obtained from GA and neural network is equal to 7152(Hz) which was reached after 42 generation. In this method, Angles of layers stacking are as following:

\[ \theta = [73^\circ, 58^\circ, 48^\circ, 73^\circ] \]

It should be mentioned that in this process, number of 50 generation was considered as stop criterion of the cycle.

As it is obvious in Figure 8 increase of fitness function in both methods are in good agreement and the final result which is the optimized frequency has rather small error. Therefore, using ANN has speeded up the optimization while keeping the error desirably small. The error due to application of ANN in GA instead of analytical solution is presented in Table 4.

Table 4. Results comparison of ANN and analytical solution in GA for frequency calculation of four-layer shell in the case of h/r=0.01

<table>
<thead>
<tr>
<th>Maximum of fitness function (GA and analytical solution)</th>
<th>Optimized fibers angle in the layers</th>
<th>Maximum of fitness function (GA and ANN)</th>
<th>Optimized fibers angle in the layers</th>
<th>Error percentage in frequency calculation</th>
</tr>
</thead>
<tbody>
<tr>
<td>7016</td>
<td>[-45,46,40,45]</td>
<td>46</td>
<td>7152</td>
<td>[73,58,-48,73]</td>
</tr>
</tbody>
</table>
Figure 8 shows average of fitness functions in generation of sequential populations for four-layer shell which is calculated by following equation:

\[
    f_m = \frac{\sum_{i=1}^{n_s} f(c_i)}{n_s}
\]

Where:
- \( c_i \) is \( i^{th} \) member of the population.

Also fitness function increase and fitness functions average in GA for four-layer shell for \( h/r=0.01 \) are presented in the same plane in Figure 9.

It can be observed that average of fitness functions increases at 45\(^{th} \) generation until reaches its maximum which the corresponding frequency of 45 is 4400(Hz) that implies the appropriate performance of the process.

Having made sure the proper performance of ANN in GA for \( h/r=0.01 \), ANN may be used for other conditions such as \( h/r=0.05, 0.1 \) to find the optimized frequency.

Figure 8. average of fitness functions in generation of sequential populations for four-layer shell in the case of \( h/r=0.01 \)

Figure 9. presentation of fitness function increase and fitness functions average in generation of sequential populations for four-layer shell in the case of \( h/r=0.01 \)
The best frequency obtained for four-layer shell for h/r=0.05 was equal to 7222(Hz) that was reached after 20 generations. The corresponding stacking of layers is as below:

\[ \theta = [-74^\circ, 56^\circ, 71^\circ, -18^\circ] \]

Also the best frequency obtained for four-layer shell for h/r=0.1 was equal to 7243(Hz) that was reached after 46 generations. The corresponding stacking of layers is as below:

\[ \theta = [-73^\circ, 73^\circ, -73^\circ, 72^\circ] \]

In both cases number of 50 generations considered as end of the cycle. All of the optimization results for four-layer shell for different thicknesses are presented in Table 5.

<table>
<thead>
<tr>
<th>h/r</th>
<th>Maximum frequency(Hz)</th>
<th>Optimized fibers angle in the layers</th>
<th>Number of generations for maximum frequency(Hz)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.01</td>
<td>7152</td>
<td>[73,62,-48,73]</td>
<td>42</td>
</tr>
<tr>
<td>0.05</td>
<td>7222</td>
<td>[-74,-56,71,-18]</td>
<td>20</td>
</tr>
<tr>
<td>0.1</td>
<td>7243</td>
<td>[-73,73,-73,72]</td>
<td>46</td>
</tr>
</tbody>
</table>

7. CONCLUSION
Since the strength of FRP shell composed of several layers is affected by orientation of fibers in different layers, so finding the best angle of fibers in each layer to optimize the strength of FRP shell is of great importance. With this respect, GA was applied to do this optimization. In the process, ANN was used to speed up the optimization process while keeping the errors small. Results comparison of analytical solution and ANN showed good agreement.

The results of optimization showed that the increase of thickness and number of layers would result in increase of optimized solution and frequency.

REFERENCES
5. C.W. Bert, “Optimal Design of a Composite Material Plate to Maximize its Fundamental Frequency”, J .of sound and vibration vol. 50(2) pp.229-237,
PREDICTION OF COMPRESSIVE STRENGTH OF COMPOSITE FIBER REINFORCED CONCRETE (FRC) USING ARTIFICIAL NEURAL NETWORK

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ABSTRACT
Within the framework of studies on FRC, a series of tests were undertaken in the laboratory in order to better understand the behavior of FRC and composite fibers to characteristic loading. The results obtained in the tests vary according to the type and arrangement of fibers, the water content, the size of grains (grains size distribution) and percentage of composite fibers. Therefore, it is important to estimate the strength of concrete according to available data and in the case of lacking of enough experimental data. For this purpose, neural network technique was used to predict the strength of concrete based on mix proportions. At first the results of experimental tests carried out in PWUT laboratory on fiber reinforced concrete specimens are presented and then the missing experimental data and gaps in compressive strength trends are predicted by back propagation method in neural network. It is worth mentioning that it can also be used to study the different mix parameters on concrete strength.

Keywords: neural network, back propagation, fiber reinforced concrete, composite fibers

1. INTRODUCTION
Compressive strength of the hardened concrete is the most important property that describes its quality and suitability for construction works. Also, it considers the mother strength, where most of other properties and strengths; such as tension, flexural, shear and bond with steel reinforcement are improved with the improvement in compressive strength and vice versa. Most often, an ultimate target in the mixture design is the 28-day compressive strength. This strength is usually determined based on a standard uniaxial compression test, and is accepted universally as a general index of concrete strength. Most research in material modeling aims to construct mathematical models to describe the relationship between components and material behavior. These models consist of mathematical rules and expressions that capture these varied and complex behaviors. Concrete is a highly nonlinear material, so modeling its behavior is a difficult task. However, the artificial neural network (ANN) was proved to be able in predicting the concrete compressive strength, without the need
of specific equations (Yeh 1998, Guang and Zong 2000, Lee 2003, Kim et al. 2004). Also, its application would reduce the time and cost required for making specimens and the 28 day waiting period before they could be tested. The ANNs have recently been widely used to model some of the human activities in many areas of science and engineering. They need sufficient input-output data, which may be theoretical, experimental or empirical. ANNs can deal with incomplete and noisy data, which is the predominant case in engineering applications.

In this study, the back-propagation neural network (BPNN) was used to predict the fiber reinforced concrete compressive strength based on the mix proportions and fiber percentages. Training and testing patterns of the network were prepared using the data sets containing the mix proportions of two ready-mixed concrete companies and fiber percentages from specimens tested in the laboratory. The estimated strengths were compared with those tested in the laboratory.

2. ARTIFICIAL NEURAL NETWORKS

As in the biological neurons, the information processing system of the ANN consists of three main aspects: transmission of information, processing of information and storage of information. The counterparts in ANN are the input layer, one or more hidden layer and the output layer. The input layer consists of number of nodes each of which receives input data of an independent variable. Thus, the total number of nodes in the input layer is equal to the total number of the input variables of the problem.

The hidden layer receives information from input layer, using the applied weights and pre-specified activation functions (Waszczyzn, 1998). The output layer receives the processed information from the hidden layer and sends the results to an external recreant. The number of nodes in the output layer is equal to the number of output variables.

The number of hidden layers and the number of nodes in each hidden layer are important factors in the design of the network, and there are no generally applicable rules to determine these numbers exactly (Flood and Kartam, 1994). However, there are some suggestions, which were proposed to aid in selecting the optimum number of nodes and layers in the hidden part (Flood and Kartam 1994, Hajela and Berke 1991).

The collected data for the problem is divided into training and testing data sets. Depending on the available data, about 60-70% of the total data is used as a training data. The number and distribution of training patterns affect the generalization ability of the ANN (Flood and Kartam, 1994). The training pattern must cover all the possible ranges of the study.

Once the topology of the ANN is determined, the training process is started by assigning values to the training parameters and specifying the activation function and learning algorithm. Different learning algorithms could be used; among of which the back-propagation algorithm is predominant one used in civil engineering applications (Adeli, 2001). This algorithm looks for the minimum of the error function in weight space using the method of gradient decent.
3. PROPOSED ANN MODEL FOR THE PRESENT CASE

The data used in this study consisted of actual mix proportions provided by two different ready mixed concrete companies and also the data of force-deflection curve of the specimens containing different percent of fibers. The back propagation neural network was applied on these data using the MATLAB software. Several MATLAB subroutines were developed and various other commands were used to perform the task. Constructing the proposed ANN model consisted of the following steps:

3.1. Preparation of Training and Testing Data Sets

The total number of records obtained was about 70; table 1 gives the minimum maximum and average values of each parameter of the concrete mix proportions and fiber percentages. Two-third of the data set (i.e., 47 records) was used for training and one-third (i.e., 23 records) was used for testing the ANN. Due to the large difference in the values of the data provided to the ANN and since the activation functions are often varied within 0 to 1 or -1 to 1, the training patterns were normalized before they were applied to the ANN.

<table>
<thead>
<tr>
<th>Slump (cm)</th>
<th>w/c</th>
<th>Unit Water Content (kN/m³)</th>
<th>Unit Cement Content (kN/m³)</th>
<th>Natural Sand (kN/m³)</th>
<th>Coarse Agg. (kN/m³)</th>
<th>Fiber Percentage (%)</th>
<th>Admix (%)</th>
<th>Stress (Mpa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Min</td>
<td>1</td>
<td>0.32</td>
<td>1.3</td>
<td>2.66</td>
<td>2.5</td>
<td>8.9</td>
<td>0</td>
<td>0.74</td>
</tr>
<tr>
<td>Max</td>
<td>9</td>
<td>0.6</td>
<td>1.6</td>
<td>5.67</td>
<td>9.2</td>
<td>10.1</td>
<td>5</td>
<td>2.2</td>
</tr>
<tr>
<td>Ave.</td>
<td>5</td>
<td>0.46</td>
<td>1.45</td>
<td>3.75</td>
<td>5.35</td>
<td>9.5</td>
<td>2.5</td>
<td>1.47</td>
</tr>
</tbody>
</table>

3.2. Determination of Mean Square Error (MSE)

MSE is an important parameter to be specified in training and testing ANNs. To identify its value, an ANN having one hidden layer with 20 nodes was used with MSE decreasing up to about 10⁻⁶. The curves of MSE for different percent of fibers are presented in Figure 1.
3.3. Training of the ANN

Computer subroutines were developed to utilize the ANN tools in MATLAB environment and used in training the suggested ANN. It consisted of the following main steps:

a. Reading the training data set and specifying the input and output variables to the ANN.

b. Normalizing the training data according to the normalization rule selected.

c. Defining the topology of the ANN. This was achieved by specifying the number of hidden layers in excess of the input and output layers. The number of hidden layers was selected to be 1 and 2. In each one of these ANNs, the number of nodes in the input and output layers was 8 and 1, respectively, see figure 2 for the one-hidden layer ANN. The number of nodes in the hidden layer was varied and searching was carried out to determine the optimum number of nodes that satisfies the acceptable value of MSE with a minimum possible time (number of epochs) consumed during the run.

d. Specifying the transfer functions used for the hidden and output layers; they were log-sigmoid ( logsig) and linear ( purline), respectively.

e. Specifying the learning rule used, which was the levenberg_MarqUdt algorithm with the identity acronyms within the MATLAB - Trainlm (Demuth et al., 2006).

f. Specifying the training parameter required for convergence (MSE) and the limiting maximum number of epochs.

g. Training the ANN defined according to the above steps.

h. Computing the normalized target variables, then converting them to the same units of the original targets.

i. Evaluating the ANN training efficiency by plotting the output of ANN versus the original target values.

<table>
<thead>
<tr>
<th>X1</th>
<th>The slump</th>
</tr>
</thead>
<tbody>
<tr>
<td>X2</td>
<td>The water to cement ratio</td>
</tr>
<tr>
<td>X3</td>
<td>The unit water content</td>
</tr>
<tr>
<td>X4</td>
<td>unit natural sand content</td>
</tr>
<tr>
<td>X5</td>
<td>The unit natural sand content</td>
</tr>
<tr>
<td>X6</td>
<td>The unit coarse agg. content</td>
</tr>
</tbody>
</table>
3.4. Determination of the optimum topology for the ANN

At each suggested number of nodes, the maximum error and the total number of epochs required to converge the specified MSE were recorded and. As clarified, the ANN with 22 nodes was considered the optimum for the case of one hidden layer. It gave reasonable maximum % error with very low number of epochs; 1.3526 at 8, respectively. Also, the ANN with 6 nodes in the first hidden layer and 5 nodes in the second hidden layer (6-5) was considered as the optimum for the case of two hidden layers, because it gave the lowest value of the maximum % error with reasonable number of epochs; 1.2268 at 98, respectively.

Comparing the performance of the above mentioned ANNs showed that the both types of ANN were performed well with this type of data. The one-hidden layer ANN with 22 nodes was the optimum in view of the time consumed (number of epochs), however, the two-hidden layers ANN with 6-5 nodes was the optimum in view of the maximum percent of error. In fact the difference in accuracy between the two types of ANN was very small when compared with the higher reduction in the number of epochs, so the one-hidden layer ANN was more preferable.

3.5. Testing of the Optimum ANN

Once trained, the optimum ANN need to be tested to evaluate whether it can successfully estimate the force-deflection curve (stress-strain behavior) and compressive strength of fiber reinforced concrete based on mixing proportions. The testing data set was used for this task. Again this set contained actual data provided by companies and empirical results from laboratory, but completely different from those used for training the ANN. Figure 3 shows the train error curves of ANN method.
4. FACTORS AFFECTING PERFORMANCE OF ANN

4.1. Effect of Learning Algorithm

There are a lot of learning algorithms available in the neural network toolbox for use with the MATLAB (Demuth et. al., 2006). The Trainlm or Levenberg-Marquardt algorithm was used in the training process of the optimum ANN. As well observed the Training algorithm was the best, since it had the lower value of the maximum % error at smaller number of epochs.

4.2. Effect of Normalization Method

The normalization method used in obtaining the optimum ANN was the mean and standard deviation (mean-std) method. To study the effect of the normalization method, the optimum ANN was trained and tested using the minimum and maximum (mm-max) normalization method. The results showed that a lower error in the target was obtained by the use of the mean-std method. This means that the mean-std normalization method can be considered as more suitable for this type of data.

5. APPLICATIONS USING OPTIMUM ANN

The use of ANN extended to be applied for studying the effect of certain
parameters on the 28-day FRC strength and force-deflection curve. For example, figure 4 shows the effect of fibers on the 28-day force-deflection curve and final compressive strength of concrete. Figure 5 shows the fitness curves of experimental and predicted values.

![Figure 4a. predicted force-deflection curve for 1% fiber specimens](image)

![Figure 4b. predicted force-deflection curve for 2% fiber specimens](image)

![Figure 4c. predicted force-deflection curve for 3% fiber specimens](image)

![Figure 4d. predicted force-deflection curve for 5% fiber specimens](image)

![Figure 5a. Fitness curve for 1% of fibers](image)

![Figure 5b. Fitness curve for 2% of fibers](image)
6. CONCLUSION AND PERSPECTIVES

In this study the feed-forward back-propagation ANN was used to predict the force-deflection curve and compressive strength of concrete based on mix proportions and also fibers percentages. The results showed that the one-hidden layer ANN with 22 nodes and the two-hidden layers ANN with 6-5 combination of nodes could accurately estimate the Concrete force-deflection curve and compressive strength. However, the one-layer ANN was more preferable. The optimum ANN was also used to study the effect of certain mix parameters on the 28-day compressive strength and stress-strain behavior; the relations developed showed trends similar to the experimental ones. The standard deviation normalization method and the Levenberg-Marquardt learning algorithm were proved to be more efficient for the present study.

REFERENCES

EXPERIMENTAL STUDY ON SHORT CONCRETE COLUMNS STRENGTHENING BY COMPOSITE FIBERS

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ABSTRACT
Nowadays, the concrete structures are strengthening in several ways. One of these ways that has drawn considerable attentions in the recent years is using composite fibers (FRP) in concrete structures strengthening. There have been many studies on the effects of these fibers on resistance and ductility of the structures. The present study intends to focus on the effects of two types of composite fibers, Carbonic and glass fibers, with three different arrangements that are being currently used in strengthening of the columns. At first, we referred to how the samples have been developed, and then introduce the arrangements that are being used. The results indicate that strengthening the short columns by carbonic composite fibers with complete wrapping which is perpendicular to the direction of loading, along the fibers; have the highest effect on the resistance and formability.

Keywords: composite fibers, short concrete columns, structure strengthening, confinement CFRP, GFRP

1. INTRODUCTION
The studies on the effect of wrapping on resistance and shape of concrete began by introducing enclosed columns with wrapping reinforcement columns. Enclosing or wrapping by cross-sectional reinforcement are inactive. It means that the wrapping pressure is created after the pressure resulting from Poisson's ratio of the enclosed concrete and the strain of the ring material of wrapper. One way which is nowadays used to wrap the concrete is using the FRP composite sheets. The sheets have been drawn considerable attention during the recent years in improvement and repairing the structures especially concrete structures due to their high resistance compared to their high weight (FRP layer with a weight of 20% of steel and stress resistance of app. 200 to 1000% of steel) and Resistance to the scratch and chemicals, resistance to exhaustion arising out of loading and immediate installation. FRP composites are made up of two fibrous and resin materials which may strengthening the column or stake of the related element.
Since this is a new technique of supporting, there has been considerable attempt on the behavior of these polymers in making the concrete buckle resistance by attaching these fibers to the intersection which is under stress which all highlight the mechanical behavior and increased buckling resistance buckling. Present article
reviews the effect of FRP sheets in the both kinds, carbonic and glass, on the compressive resistance of the short concrete columns.

2. GEOMETRICAL SPECIFICATIONS OF CONCRETE COLUMNS
EXPERIMENTAL SAMPLES
The present study uses cylindrical columns with 500mm length and 200mm diameter which fulfills the requirements of short columns. In the improvement process or seismic resisting by composite fibers, the column that was introduced earlier may be slender due to increased loading. Therefore, prior to strengthening and for avoidance of possible breaking as a result of extreme buckling, the column’s behavior was controlled. To review the effect of FRP on the resistance of columns, six cylindrical samples were used in the experiments. Two samples were used without cover of FRP and two other samples had the GFRP, other two samples were wrapped with CFRP. Concrete samples were developed in a standard environment within 28 days from producing. After the development process completed, 2 samples, E and F, remained as controller samples. B and A samples with CFRP sheets were strengthen with one layer in the form of complete ring and spiral cover with null phase and a 1cm cover with approximately 30C angle then tested (Figure 2). D and C samples were strengthen with GFRP sheets as well.
To prepare the surfaces, the concrete surface was cleaned and dried completely, then an even resin layer was distributed on the surface that was going to be confined. The fibers were cut into the various numbers, and then it was put on the resin layer without any compression. Another resin layer was distributed on the surface. Resin development is a phenomenon depending on the heat and time. Contraction of the resin completely takes place normally in standard conditions. In the samples under test, the development of resins in environmental conditions took place under the temperature of 11°C. during this phase, all necessary actions was taken to prevent direct contact of sunlight, rain, dire heat changes as well as any pollution, dust and moisture exposition. The quality of the samples was satisfactory with no spume observed.

3. CONCRETE MIX DESIGN AND SPECIFICATIONS OF COMPOSITE FIBER

Concrete mix design in producing the cylindrical columns per 1 m$^3$ of volume is as follows:

| Table 1: Specification of consumed concrete in tests |
|---------------------------------|-------------------|
| Gravel             | 800 kg            |
| Sand               | 1100 kg           |
| Cement             | 350 kg            |
| Water to cement Ratio | 0.48              |
| Type of cement     | Portland Type 2   |

No additives were used in concrete producing procedure. The results obtained from tests that were conducted on the concretes indicate that the utilized concrete has compression resistance of cylindrical sample of about 25MPa. Also, the specifications of the material of FRP are presented in the following table:

<table>
<thead>
<tr>
<th>Table 2: Material specification of FRP</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material</td>
</tr>
<tr>
<td>----------</td>
</tr>
<tr>
<td>GFRP</td>
</tr>
<tr>
<td>CFRP</td>
</tr>
</tbody>
</table>

4. HOW TO CONDUCT THE AXIAL COMPRESSION TESTS

The final step of the test is breaking the samples under the compression. Prior to breaking the samples, their surface were saturated with melt sulfur in order that even power exert on the surface. Breaking the samples by two jacks of 100 and 300 tons was conducted in the laboratory. Considering the facilities, the test was conducted in the controlled form. The jack of 100tons was calibrate digitally; while, the jack of 300tons lacked this advantage. So it was essential to read the force and displacement changes manually. Consequently, the of human error also affected the results. Unconfined samples which were E and F sample broken by jack of 100tons and confined samples were broken by jack of 300tons due to lack
of needed capacity. In order to registering the displacement and longitude strain of the samples along the cylinder axis, two LVDT devices were used in both sides of the jack. The precision of these devices were 0.01mm.

4.1. Results of Tests
The conducted tests on controlled samples of E and F indicated that the average compression resistance of short columns was 70.5 MPa which has approximately 5% difference with analytical equations.

Diagram 1 shows the stain-stress behavior of the enclosed columns by CFRP. In the confined sample in the form of a ring, the weakness of performance of integrated column and supporter layer causes a local yielding in strain of approximately 0.00135. After the force was increase from this phase onward, in fact, the confined layers are activated. It can create a confinement behavior in the wrapping in a form of spiral. Probably, correct implementing the CFRP is a main reason for good performance of the column that indicates the role and importance of the implementation in practice. Behaviors suggest that elasticity module of diagram A. had decreased but had reached the same after loading with column B. also, the wrapped column in the form of ring indicated resistance and forming. In fact, enclosing the ring exerts better control compared to the lateral strains perpendicular to the axis of the column. Increased strain such as breaking (app.90%) is more significant than the resistance of (app.22%) compared to that of column B.
In diagram 2 the strain-stress graph of the confined columns with GFRP has been indicated. Column B broke just like the columns of wrapped with carbon composite fibers. In continuing, the GFRP was used. In column B, failure strain increased (app.38%) was more significant than resistance increase (app.20%) the column B.

![Diagram 2. Strain-Stress of Sample C&D confined with GFRP](image1)

Finally all 4 Samples strain-stress graph gathered for comparison in diagram 3.

![Diagram 3. Strain-Stress of all 4 Samples](image2)

5. CONCLUSION
Confined columns had resistance with an average of 265MPa. It clearly indicates that Samples A&C in the compression resistance tests whose results are being observed in diagrams 1&2, the failure observed double. This has caused that FRP has not loaded from the beginning and after the early concrete beating, the samples are equipped with FRP, then the loading capacity is increased. This indicates the need to focusing on the proper manner of covering, involvement of composite
fibers with concrete samples when strengthening the columns using these fibers. In both models of GFRP and CFRP, the samples that have been taken a form of complete wrap can be tested compared to other samples. They may have stronger resistance and formability. It, therefore, seems that using these fibers in the form of wrapped may has highest effect on the resistance and formability of the concrete columns.

Although, sample C has the form of wrapped, it may has the least elasticity module, consequently, its resistance may be least. But the results obtained indicate that using these composite fibers, even if they are being used in some sections of the column, may exert considerable effect on the resistance and formability.

Samples A and B as well as C and D have the same slope in most part of the diagram which suggests the effect of FRP on elasticity module of equipped columns and the arrangement and manner of using FRP has little effect on this module.

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ارائه شبکه عصبی مناسب برای طرح اختراع بیانی سدهای بنی گلتكی

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چکیده

یکی از روش‌هایی که در ساخت سدهای بنی و خاکی به عنوان گانگزینی مناسب مطرح است مطرح ساخت سدها به شیوه بتن غلتكی می‌باشد. صرف ورود کمتر استفاده از املاحی آلای دراهاسی، کوته شدن زمان ساخت و تداوم ساخت همگی از ویژگی‌های این نوع روش ساخت سدها می‌باشد که باعث برتری یافتن این روش شده است و در نهایت این شیوه برای ساخت سدها در نقاط مختلف جهان به عنوان گانگزینی مناسب مطرح شده است. از طریق گسترده‌تری و مصالح مورد استفاده در این نوع برن و یپچیدگی طرح اختراع آن و متأثر بودن طرح اختراع آن از پارامترهای مختلف و نیز یافتن روابط بین پارامترهای مختلف طرح اختراع آن باعث پیدا نموده است. این اثرات ممکن است در افزایش صنعت سازه‌های ده‌ها، مدل‌سازی‌های مختلف از جمله مدل‌سازی‌های آزمایشی گردیده باشد که نسبت به مشخصات جدیدی استفاده باید داشته باشد. طرح اختراعی آن از چنین شیوه‌هایی به عنوان گانگزینی‌های مناسب چند نیاز به پرستون (MLP) به همراه الگوریتم آموزش پس از انتشار خطا، که بیشتر در زمینه‌های مدل‌سازی قرار می‌گیرد از آن به عنوان هسته اصلی مدل سازی در این مقاله استفاده شدند.

کلیدواژه‌های اصلی: شبکه‌های عصبی، بتن غلتكی، پیش‌بینی مقاومت قیرا، شبکه‌های عصبی چند نیاز پرستون

1- مقدمه

با کشف این حقیقت که مجز انسان محاسبات را با روشی کاملاً متمایز از کامپیوترهای دیجیتال می‌تواند، این انتظار اینم که مجز در حیطه‌های بی‌سابقه بی‌سابقه، با استفاده از شبکه‌های عصبی مناسب دیجیتال بتواند استفاده کرد و همچنین با استفاده از روشهای پیچیده‌تری با انتقال دست‌هایی با هدف به‌کارگیری دست‌های قوی‌تر، می‌تواند به‌عنوان ابزار اصلی برای حساب‌رسی‌ها و... را با سرعتی
بنن غلتبی یکی از شیوه‌های نسبتاً جدید ساخت سد در ایران است، بیشین و سمال سازی طرح اختلاط و مقاومت این بنن بر اساس پاژامه‌های ورودی به مانند انواع دیگر بنن و همچنین از پیچیدگی خاصی برخوردار است. از طرفی ورود انواع بوزوالنها، می‌توان افزودنی جدید در طرح اختلاط این نوع بنن و همچنین ملت و ساخت این بنن از شیوه‌های مقاومت بنن ریزی، اختلاط و زرگر، پیچیدگی طرح اختلاط این را مسابع نموده است [13]. مدل سازی مقاومت بنن غلتبی توسط شیوه‌های سنتی و مهندسی به یکدیگر می‌یابد در این شیوه، قادر به پیش بنن‌های مناسب نخواهی بود، جرا که فنار مقاومتی بنن تحت تأثیر شرایط غیر خاطی است و از کوکتی‌های جزئی طراحی موجود در مخلوط و آمادگی بین این اجرا مانند می‌یابد [17].

- روش تحقیق

2- شیوه‌های عصبی به کار رفته جهت مدل سازی

شیوه‌های عصبی یکی از شیوه‌های پیشرفته است که به‌طور عمده بر اساس پارامترسازی الگوریتم‌های شبکه‌های حیاتی می‌باشد. شیوه‌های عصبی به کار رفته در این پژوهش شامل شیوه‌های مختلفی می‌باشند که شامل شیوه‌های ورودی و خروجی تکی شده است که به کار رفته در این پژوهش شامل شیوه‌های ورودی و خروجی تکی شده است که به کار رفته در این پژوهش شامل شیوه‌های ورودی و خروجی تکی شده است که به کار رفته در این پژوهش شامل شیوه‌های ورودی و خروجی تکی شده است که به کار رفته در این پژوهش شامل شیوه‌های ورودی و خروجی تکی شده است. شیوه‌های عصبی به کار رفته جهت مدل سازی سد در این پژوهش شامل شیوه‌های ورودی و خروجی تکی شده است که به کار رفته در این پژوهش شامل شیوه‌های ورودی و خروجی تکی شده است که به کار رفته در این پژوهش شامل شیوه‌های ورودی و خروجی تکی شده است که به کار رفته در این پژوهش شامل شیوه‌های ورودی و خروجی تکی شده است.
این شبکه‌ها براساس الگوهای پرداتشکری به یک نام سلول عصبی عمل می‌کنند (شکل 1). سلول‌های لابه‌ورودی مقایسه عمده‌بردار ورودی هر یک از الگوهای آن بی‌سوی پرداتشکری به‌ناهی پهنای منتقل می‌کند و سلول‌های لابه سه پهنای خروجی و پهنای لابه خروجی بر اساس ۲ بر روی مقایسه ورودی حکم به پرداتشکری اطلاعات دست می‌زند. تابع ۲ در این شکل با نام تابع تحريكی در این شکل شناخته می‌شود و می‌تواند از نوع تابع سیگموئید و تابع هیرپولیک در برخی از آخرین و یا اکثر یکند [14].

شکل 2- (چپ) سلول عصبی و عملیات رایانش (راست) (۱) تابع تابع‌های هیرپولیک (راست) (۲) تابع سیگموئید

در این شبکه‌های عصبی دو روال انجام می‌شود. روال تابعی که شامل اعمال الگوهای وظیفه‌ورودی به شبکه و تغییر خروجی‌های سلولی یا ثابت‌ماندگی خروجی‌های لابه به‌ناهی سه پهنای حکم، با مقایسه نتیجه خروجی با مقیاس هدف آن الگو و تعیین خطا و با برآورده یا انتخاب روش (مرجعه ۱) و بر اساس الگوهای مختلف آموزش این خط را از لابه‌های انتخاب‌های به‌ناهی، آموزش خطای کلی انتقال داده و در جهن این انتقال آوران و با پایان‌ها رابه گویند این تنظیم می‌کند که خطای شبکه به پایین ترین سطح برسد [14].

\[
mse = \frac{1}{N.S_0} \sum_{i=1}^{N} \sum_{j=1}^{S_0} (t_{ij} - 0_j)^2
\]

(1)

تابع خط و عملکرد شبکه
در راه‌های:
۱. هدف
۲. خروجی مدل
۳. قابلیت سلول خروجی
۴. زمان الگو
۵. تعداد سلول‌های خروجی
۶. تعداد الگو

پارامترهای مختلف شبکه عصبی در مدل‌سازی مقاومت
شبکه‌های BP با یک پهنای پهنای در ناحیه با عضویت خود در لابه‌های اساسی مدل‌سازی قرار گرفته است. همچنین از تابع تحريكی تابع تابع‌های تابعی رابه و تابع تخليه خروجی (Tanh) در لابه‌های خصوصی استفاده شده است. در ضمن از نرم افزار اکثر (یا)
مجموعه داده‌های مدل سازی

اطلاعات به کار رفته در این مدل سازی از میان طرح اختلال‌های آزمایشگاهی سد زیردان گردآوری شده است.

زکره‌های جمع‌آوری شده شامل ۱۸۰ طرح اختلال می‌باشد که پس از بازیابی آنها داده‌های با خطای چشمگیر و همچنین داده‌های با تکرار در پارامترهای موثر بر طرح اختلال و مقاومت كنار گذاشته شدن و در نهایت ۱۱۱ طرح اختلال به عنوان پایگاه اصلی مدل سازی‌های مختلف به کار گرفته شده‌اند.

مدل‌سازی مقاومت ۷۰ و ۹۰ روزه

با توجه به اینکه طرح‌های اختلال‌های حاصل مجموع مواد سیمانی تا ۱۹۰ کیلگرم بر متر مکعب و با پله‌های ۱۰ کیلگرم ساختمان برد و چند جای مدل پیش‌بینی مقاومت از میان اطلاعات گردآوری شده نیز به این ترتیب عمل شد که برای هر طرح اختلال روابط مقاومت‌هایی ان در سنین مختلف محاسبه شود.

پارامترهای ورودی مدل

پارامترهای متفاوتی بر مقاومت بتن گلتنی تاثیرگذار هستند که ازجمله این پارامترها می‌توان به میزان و نوع سیمان و پوزولان، شن و ماسه، ریزی ذرات سیمان، مقدار آب، مدل ریزی ماسه، مقدار ریزین سنگدانه، مقدار ریزین سنگ‌دانه و میزان و نوع مواد افزودنی اشاره کرد. علاوه بر این مواد پارامترهای ترکیب این و جود دارندگی از آنها به عنوان شناخت‌های موثر بر مقاومت بتن گلتنی [۹] از میان پارامترهای مستقل موثر بر مقاومت بتن گلتنی آن دسته از پارامترها ویژگی‌هایی که در اطلاعات گردآوری شده حضور داشتند انتخاب شده بود تا بتوان از آنها در شرایط گوناگون استفاده نمود. به کار گرفته شدند. این پارامترها به ترتیب عبارتند از:
- ۲۵-۵ ماسه ۰-۵ ماسه ۳-۰ ماسه، پوزولان، شن، آب
- افزودنی‌های Conplast RP264M و Chryso Tard CHR. Chrysos plast CER
- بدون مصالح

آماده‌سازی و استاندارد کردن داده‌ها

جهر انجام محاسبات در ابتدا لازم است که داده‌های خام بین ۰ و ۱ استاندارد سازی شوند [۷]. پنجمین داده‌های ورودی با توجه به میزان داده‌های خنثیک و حذف‌الاستاندارد شدن. این عمل به تقابل‌یاد کردن داده‌ها خواهد بود از روی‌ها استاندارد سازی دیگر کاربردی است. بعد از خروجی گرفتن از شیب‌های خروجی‌های استاندارد شده با پیش‌بینی داده‌های واقع تبدیل شوند تا با مقادیر مشاهده شده مورد مقایسه قرار گیرند. همچنین محدوده ماکریم و مینیمم داده به شرح جدول ۱ می‌باشد:

جدول ۱ محدوده داده‌های طرح اختلال‌ها

برای برنامه نویسی‌های مورد نیاز مدل سازی استفاده شده است. MATLAB
پس از تحلیل داده توسط شیبک عصبی نرم افزار مبلخب خروجی های صوتی و زن هر یک از داده ها برای مقاومت ووزه در جدول ۲ و ۳ برای مقاومت فشاری ۷۰ روژه در جدول ۳ و ۴ برای مقاومت فشاری ۹۰ روژه در جدول ۴ و همچنین برای مقاومت فشاری ۱۸۰ روژه در جدول ۶ و ۲ که به ترتیب در جدول اول شاخص وزن داده است.

### جدول ۳: نتایج (1,1) مقاومت فشاری ۷ روژه

<table>
<thead>
<tr>
<th>مقاومت فشاری</th>
<th>تغییرات وزن</th>
<th>بیزونا</th>
<th>افزودنی</th>
<th>فردی</th>
<th>مطلق</th>
<th>۴/۳</th>
<th>۱/۳</th>
<th>۲/۳</th>
<th>۵/۳</th>
</tr>
</thead>
<tbody>
<tr>
<td>W(1,1)</td>
<td>(kg/m³)</td>
<td>(kg/m³)</td>
<td>(kg/m³)</td>
<td>(kg/m³)</td>
<td>(kg/m³)</td>
<td>(kg/m³)</td>
<td>(kg/m³)</td>
<td>(kg/m³)</td>
<td>(kg/m³)</td>
</tr>
<tr>
<td>110</td>
<td>857.8025</td>
<td>982.765</td>
<td>744.7871</td>
<td>1018.7999</td>
<td>-271.1188</td>
<td>-30.1665</td>
<td>2311.84</td>
<td>2586.4981</td>
<td>960.5746</td>
</tr>
<tr>
<td>130</td>
<td>487.2844</td>
<td>-201.6556</td>
<td>344.5199</td>
<td>359.4187</td>
<td>-95.5121</td>
<td>-10.0123</td>
<td>701.8738</td>
<td>-2178.5897</td>
<td>-326.5085</td>
</tr>
<tr>
<td>140</td>
<td>6212.0142</td>
<td>5919.4822</td>
<td>8349.0382</td>
<td>9462.9423</td>
<td>2589.298</td>
<td>39.0134</td>
<td>22886.0558</td>
<td>23810.0079</td>
<td>1024.390</td>
</tr>
<tr>
<td>150</td>
<td>46.729</td>
<td>-85.2741</td>
<td>-192.0376</td>
<td>54.898</td>
<td>15.6507</td>
<td>1.739</td>
<td>-106.787</td>
<td>664.9755</td>
<td>-209.5466</td>
</tr>
</tbody>
</table>

### جدول ۴: نتایج (2,1) مقاومت فشاری ۳۸ روژه

<table>
<thead>
<tr>
<th>مقاومت فشاری</th>
<th>تغییرات وزن</th>
<th>۲/۲</th>
<th>b2</th>
<th>۳۲</th>
<th>داده ها</th>
</tr>
</thead>
<tbody>
<tr>
<td>W(1,1)</td>
<td>(kg/m³)</td>
<td>(kg/m³)</td>
<td>(kg/m³)</td>
<td>(kg/m³)</td>
<td>(kg/m³)</td>
</tr>
<tr>
<td>110</td>
<td>67.8715</td>
<td>-2455.65</td>
<td>68.6172</td>
<td>59.3</td>
<td></td>
</tr>
<tr>
<td>120</td>
<td>0.18378</td>
<td>1856.336</td>
<td>0.83419</td>
<td>64.9</td>
<td></td>
</tr>
<tr>
<td>130</td>
<td>-0.25585</td>
<td>732.407</td>
<td>0.84413</td>
<td>75.9</td>
<td></td>
</tr>
<tr>
<td>140</td>
<td>-0.08802</td>
<td>-17350</td>
<td>0.87284</td>
<td>88.4</td>
<td></td>
</tr>
<tr>
<td>150</td>
<td>-70.8489</td>
<td>128.3482</td>
<td>71.3546</td>
<td>119.5</td>
<td></td>
</tr>
</tbody>
</table>

### جدول شماره ۴: نتایج (1,1) مقاومت فشاری ۲۸ روژه

<table>
<thead>
<tr>
<th>مقاومت فشاری</th>
<th>تغییرات وزن</th>
<th>۲/۲</th>
<th>b2</th>
<th>۳۲</th>
<th>داده ها</th>
</tr>
</thead>
<tbody>
<tr>
<td>W(1,1)</td>
<td>(kg/m³)</td>
<td>(kg/m³)</td>
<td>(kg/m³)</td>
<td>(kg/m³)</td>
<td>(kg/m³)</td>
</tr>
<tr>
<td>110</td>
<td>67.8715</td>
<td>-2455.65</td>
<td>68.6172</td>
<td>59.3</td>
<td></td>
</tr>
<tr>
<td>120</td>
<td>0.18378</td>
<td>1856.336</td>
<td>0.83419</td>
<td>64.9</td>
<td></td>
</tr>
<tr>
<td>130</td>
<td>-0.25585</td>
<td>732.407</td>
<td>0.84413</td>
<td>75.9</td>
<td></td>
</tr>
<tr>
<td>140</td>
<td>-0.08802</td>
<td>-17350</td>
<td>0.87284</td>
<td>88.4</td>
<td></td>
</tr>
<tr>
<td>150</td>
<td>-70.8489</td>
<td>128.3482</td>
<td>71.3546</td>
<td>119.5</td>
<td></td>
</tr>
</tbody>
</table>
**جدول ۱: تناوبی w(2,1) و b1 و b2 و ماکروکیم داده‌های مقاومت فشاری ۲۸ روزه**

<table>
<thead>
<tr>
<th>مقاومت فشاری</th>
<th>b2</th>
<th>b1</th>
<th>ماکروکیم داده‌های</th>
</tr>
</thead>
<tbody>
<tr>
<td>ورژه</td>
<td>5-25</td>
<td>5-25</td>
<td>0-5</td>
</tr>
<tr>
<td>F(1,1) (kg/m³)</td>
<td>(kg/m³)</td>
<td>(kg/m³)</td>
<td>(kg/m³)</td>
</tr>
<tr>
<td>120</td>
<td>9045.9221</td>
<td>-4026.487</td>
<td>2306.897</td>
</tr>
<tr>
<td>130</td>
<td>-706.109</td>
<td>52.836</td>
<td>15.049</td>
</tr>
<tr>
<td>140</td>
<td>15706.923</td>
<td>151744.384</td>
<td>157072.2375</td>
</tr>
</tbody>
</table>

**جدول ۲: تناوبی W(2,1,1) و b1 و b2 و ماکروکیم داده‌های مقاومت فشاری ۰ روزه**

<table>
<thead>
<tr>
<th>مقاومت فشاری</th>
<th>b2</th>
<th>b1</th>
<th>ماکروکیم داده‌های</th>
</tr>
</thead>
<tbody>
<tr>
<td>ورژه</td>
<td>5-25</td>
<td>5-25</td>
<td>0-5</td>
</tr>
<tr>
<td>F(1,1) (kg/m³)</td>
<td>(kg/m³)</td>
<td>(kg/m³)</td>
<td>(kg/m³)</td>
</tr>
<tr>
<td>110</td>
<td>90.5573</td>
<td>84.833</td>
<td>90.619</td>
</tr>
<tr>
<td>120</td>
<td>2816.2378</td>
<td>-2862.523</td>
<td>-1674.8758</td>
</tr>
<tr>
<td>130</td>
<td>-6148.698</td>
<td>4111.534</td>
<td>-191.898</td>
</tr>
<tr>
<td>140</td>
<td>194.7366</td>
<td>197.6006</td>
<td>191.898</td>
</tr>
<tr>
<td>150</td>
<td>606244</td>
<td>66.632</td>
<td>352.633</td>
</tr>
</tbody>
</table>

**پارامترهای MATLAB**

\[
Purlin(W2,1)*\text{Tansig}(W1,1\circ A+b1)+b2
\]

که به طور مثال اگر مقاومت فشاری ۹۰ روزه طرحی با مجموع مصالح سیمانی ۱۲۰ کیلوگرم برمتر مکعب و با

یافته‌ای که برنامه MATLAB
نستهای اختلاف مقدار جدول ۱۰ مدنظری‌بندی به صورت زیر عمل می‌شود:

جدول ۸: نتایج (W(1,1) مقاومت فشاری) ۱۸۰ روزه

<table>
<thead>
<tr>
<th>جدول</th>
<th>نتایج (w(2,1) و b1 و b2 و ماکزیم مقدارهای مقاومت فشاری) ۱۸۰ روزه</th>
</tr>
</thead>
<tbody>
<tr>
<td>جدول ۹: نتایج (w(2,1) و b1 و b2 و ماکزیم مقدارهای مقاومت فشاری) ۹۰ روزه</td>
<td></td>
</tr>
</tbody>
</table>

جدول ۱۰: داده‌های اولیه مثال

جدول ۱۱: داده‌های مثال W(1,1) مقاومت فشاری ۱۲۰ کیلوگرم مایع سیمان

اسبانه شکل عدسی مناسب برای طرح اختلال به شکل سد شده‌ای...
حال $z$ را در مقدار $(2.1)W$ ضرب شده و با عدد $b_2$ جمع می‌شود و در انتهای به دلیل اینکه روی برابری اعداد نمایش شده است چهت رسمی به مقامات مردم‌نظر بايد حاصل را در ماکزیم داده‌هاکه برای $109.1kg/cm^2$ است ضرب شده که جواب نهایی برای $100.3kg/cm^2$ می‌شود.

---

۴-بحث

استفاده از شکله‌های عصبی مدل سازی مقاومت بین عضلانی را دچار تحول ساخته است و تابیپرسی مسئله و دیقیقی را در برداران است. در محاسبه که توسط سوکن و هماکاران صورت گرفته نیز نتایج سیاسی دیقیقی به واسطه شکله‌های عصبی در رابطه با بیش بین مقاومت شاری بین صورت گرفته است[9] این مدل نیز با کیهانی و دست دبایه به بیش بین مقاومت را به صورت آنی یا نب قطع بسیار ناسیاب اخلاق و هدف و می‌تواند بسیاری از شکله‌های نموداری طرح اختلاط بین عضلانی را کاهش دهد. مقاومت شاری متانه‌ای سیمان شامل انواع مقاوتی از پورولایه بر اساس شکله عصبی بدون نیاز به انجام هیچ گونه مطالعات آزمایشگاهی سبب صرف جویی هزینه‌ها به مقدار بسیار زیادی در پروژه‌ها می‌گردد[13].

یا به کاربردن این مدل‌ها بیش بین مقاومت و استفاده از شکله‌های مینیمم سازی می‌توان با درنظرگرفتن بسیاری از نظریات می‌توان ساخت نمونه‌های آزمایشگاهی به طرح اختلاط بینه‌ها در سازه‌ای و مالهای دست یافتن. به کارگیری این مدل‌ها برای بیشتر بیرش یا بارمترهای می‌توان بر بین عضلانی سفارش‌های آزمایشگاهی است. این‌ها از خصوصیات بیشتری از سیستم‌ها (نوع کاری، شرایط سنگین‌های سریع جلوگیری از اجتناب و...) نوی سیمان مصرفی و شرایط ساخت نمونه‌ها (زمان اختلاط، نحوه اختلاط، فصله زمانی بین اتمام اختلاط و بین روز و...) درکرت دیگر‌سیات‌های ورودی بیش بین مقاومت را دیقیق تر می‌کند.

تشکر و قدردانی

بنیان‌گذاری از نمونه‌سازی جنگ ویلهالو و تکنیک‌های آزمایشگاهی دیوان جهان کوت (بی‌پاکا سردژدان) که در کلیه مراحل ساخت مخلوط‌های بین و انجام آزمایش‌ها، صمیمانه همکاری فراوانی را مبتنی داشته‌اند سپاسگزاری و قدردانی می‌شود.

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بررسی اثر بر کاتیون‌های مختلف کلرید در ایجاد پیوند شیمیایی و روش اندازه‌گیری کلرید آزاد در سیستم‌های سیمانی

فاطمه جعفری‌پور، فهمیه فیروزیار

1. عضو هیات علمی مرکز تحقیقات ساختمان و سیمان
2. کارشناس تحقیقات مصالح ساختمانی مراکز تحقیقات ساختمان و سیمان

چکیده
هنگامی که کلرید محلول در آب در یک سیستم سیمانی مانند مالات یا تین وجود داشته باشد، می‌تواند سبب خورودی فنر در نظر گرفته شود. پرداختنی کلرید یا FA C3A ترکیب می‌شود و کلرور آلومنیا تولید می‌شود. بنابراین سیمان‌هایی که ندارند مقدار بیشتری از کلرید را پیدا می‌کنند، وقتی که پرتو کلرید‌های پیوند یافته و غیر محلول تبدیل شده و پرتو این غیر فعال می‌شوند، به صورت مایع آبسید کرده و باقی می‌مانند. این آب و مایع در نتیجه آنها پرتو کلرید‌های آب‌دار و شدت خورودی به طور قابل توجهی تابع نوع کاتیون است. برخی سنگدانه‌ها حاوی مقدار قابل توجهی کلرید هستند که در سنگدانه مخصوص بوده و در واکنش خورودی شرکت نمی‌کند. در مورد آزمون استاندارد ملی ایران شماره 9637، بخشی از کلریدی که در این سنگدانه‌ها وجود داشته و در واکنش خورودی نشان داده نیز آنها گیاهی می‌شود. با توجه به اینکه مقدار کلرید اندکی کم‌تر است به شدت به سمت گرایش نیمه‌ای شده که عموماً در واکنش خورودی شرکت نمی‌کند. در این مقاله نتایج حاصل از انجام یک پژوهش تحقیقاتی در زمینه «بررسی اثر تأثیر کاتیون‌های مختلف کلرید در ایجاد پیوند شیمیایی و بررسی روش اندازه‌گیری کلرید عموماً در سیستم‌های سیمانی» ارائه شده است.

کلیدواژه‌های مقاله: C3A، کلرید آزاد، کاتیون‌های مختلف، پیوند شیمیایی، کلرید عصاره‌گیری، سیمان‌های سیمانی

مقدمه

بتن ترکیبی است که خاصیت قلیایی سپار زیادی دارد. این خاصیت آن را از سوی اینکه که بر اثر خورود در ایجاد یک پیوند می‌شود. خول شفاف به تین تحت تحت چین شرایط قلیایی در حالی غیر فعال می‌شود. وقتی این چیز یک دیگر محفظه آب و برق و بار و توزیع عامل‌های مانند کلریدها، سولفیدها و دی‌اکسی‌کردن... به بتن قلیایی آن از بین می‌رود و فولاد خطافی شده داخل آن در معرض خورودی واقع می‌شود. از این رو...
جلوگیری از ورود این عوامل به بتن یکی از عوامل ضروری در حفظ دوام بتن است. ازجمله عوامل مهم‌ها به بتن و آرمان‌ های داخل آن، بون کلرید و ترکیباتی است که این ماده را در طول زمان موجود در بتن که در اثر اضافه کردن آب به مخلوط سیمان و شیم می‌تواند شده است وارد بتن شود و در کنار فولاذ قرار گیرد. نتایج از این پژوهش نشان می‌دهد که به نمک فریل مصرف است. مشکل عمده بون‌های این است که بیوند نداده و به طور آزاد در حفظ می‌آید و اطراف فولاد درکننده می‌یابند. بون‌های کلرید با رساندن به فولاذ خلافت شده که لازم به وجود خودکشیدن در اثر بالای بتن بر سطح این تراکم داده است، این‌طور محافظ را از بین برده و فولاد بدون پوشش را در معرض pH محيط خارجی قرار می‌دهد که باعث از بین رفتن شدید فولاد می‌گردد.

در این شرایط، در مرکز تحقیقات ساختمان و مسکن برتریت تحقیق «پرورش C3A سیمان برناردو بر کاترون‌های مختلف کلرید در این پژوهش ایجاد شد و در این بررسی پرورش C3A سیمان در حلال کلریدی از طریق ساخت خالص در آزمایشگاه و انویدان آن به مقدار مختلف سیمان و اضافه کردن نوع نمک کلریدی به نسبت‌های مختلف و همچنین در نمونه‌های ساخته شده با سیمان دارای C3A مختلف که شرایط واقعی گزارش فردی، مواد بررسی قرار گرفته است.

- پرورش ساختار شیمیایی بتن

ساختار شیمیایی بتن نش اتمایی در روند اسید‌سازی دارد و نشان‌های اصولی و منافع ساختار بسیار در کنار این درک، می‌تواند در نتیجه افزایش بتنی از ترکیبی این کلرید های مختلف کلرید در این پژوهش ایجاد شد و در این بررسی پرورش C3A سیمان در حلال کلریدی از طریق ساخت خالص در آزمایشگاه و انویدان آن به مقدار مختلف سیمان و اضافه کردن نوع نمک کلریدی به نسبت‌های مختلف و همچنین در نمونه‌های ساخته شده با سیمان دارای C3A مختلف که شرایط واقعی گزارش فردی، مواد بررسی قرار گرفته است.

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به شرح زیر است:
- سیستم جامد
- سیستم منافذ
- سیستم محلول
- سیستم محلول منافذ

سیستم‌های جامد و محلول از منافذ مربوط به ساختار شیمیایی است، سیستم منافذ محدود به ساختار فیزیکی است. سیستم جامد در محدوده‌های هیدرانتاسیون است که شامل و C- S- H هیدروکسید کلسیم و فازهای آلومنیوم و فریت است. قسمتی از دان می‌زند و در این کلرانت باقی می‌ماند که ناشی از بزرگ بودن آن دراز است.

سیستم منافذ شامل منافذ زل است که انتزاع آن سیستم کوچک و منافذ مویی است که انتزاع آنها بزرگتر از منافذ مویی است که در تحقیق تراکم ناقص بین ایجاد می‌شود.

بختیه با تمام سیستم منافذ با محلول پر می‌شود که در این محلول عمداً شامل هیدروکسی کلسیم (NaOH) و هیدروکسی کلسیم (KOH) و هیدروکسی کلسیم (Ca(OH)2 است. تغییرات در هریک از سه سیستم یاد شده سبب تغییر در دو سیستم دیگر می‌شود.
درآمدی، ستون اصلی می‌باشد، که در اینجا باید به تعداد گرمی که در گروه فلز، نیترات آسیان درجه‌بندی شده است.

<table>
<thead>
<tr>
<th>جدول 1: مقادیر و عملکرد ترکیبات اصلی سیمان</th>
<th>عملکرد</th>
<th>مقدار (درصد)</th>
</tr>
</thead>
<tbody>
<tr>
<td>بسیار فعال و مرواریده‌ای</td>
<td>C2S</td>
<td>50-65</td>
</tr>
<tr>
<td>مزاجی و مرواریده‌ای</td>
<td>C2S</td>
<td>25</td>
</tr>
<tr>
<td>مزوی-پارک پایین و غلیظ</td>
<td>C2S</td>
<td>10</td>
</tr>
<tr>
<td>دو پالایش و مقاومت و بازیابی</td>
<td>C2AF</td>
<td>10-8</td>
</tr>
<tr>
<td>کنترل بسیار سیمان</td>
<td>C2S</td>
<td>5</td>
</tr>
</tbody>
</table>

- خوردنگی کلریدی

خوردنگی بر اثر تفویض بیون کلرید یکی از عوامل اصلی و مهم تخریب سازه‌های بتن مسلح است که در معرض آب دریا با کمک‌هایی مانند دارو، همچنین کلریدها می‌توانند روی سطح بتن رسوب کنند. این رسوبها از طریق قطرات بسیار زیادی دریا و گردشگیر مطلق در هوا تشکیل می‌شود.
هگامی که کلرید در بین سطح شده وجود داشته باشد می‌تواند سبب خوردگی‌های بسیار شدید آرمان‌های شود. منشا کلریدها از دو منبع عده به شرح زیر است:

الف - کلرید با منشا داخلی
این نوع کلریدها در هنگام اختلال به بین وارد می‌شوند. مانند استفاده از افزودنی‌های زودگیر طنین (کلرید کلسیم)، سگدانه‌ها و آب دریا از سایر آب‌های شور.

ب - کلرید با منشا خارجی
این نوع کلریدها در زمان سخت شدن به بین وارد می‌شوند. مانند استفاده از نمک خشیچ در ساخت بزرگراه‌ها و کلریدهای ناشی از آب دریای ساحلی (اسکله).

اتر نمک‌های کلرید تا حدی به شوی وارد شدن آن دارد. چنانچه کلرید هنگام اختلال اجرا مشکل بین وجود داشته باشد؛ فارتری کلرید اولولونتاس (C₃A) سیمان تا حدی با کلرید واکنش داده و مشکل پیوند شیمیایی کلروآلمینیات کلسمی می‌دهد. در حالت، کلرید در محلول منافع غیر محول بوده و در واکنش خوردگی شرکت می‌کند. قابلیت سیمان برای تشکیل کمپلکس کلرید محدود است و بستگی به سیمان دارد. به عنوان مثال، سیمان ضدسولفات که دارای C₃A کم است نمک آرمان‌های کلرید در حد کمی یافت می‌گردد.

همچنین، تجربیات نشان می‌دهد که میزان کلرید بیش از ۴۰ درصد وزنی سیمان، خطر خوردگی را افزایش می‌دهد. شایان ذکر است که این حداً هنگام میزان کلرید بیشتر از حد ذکر شده محتماً سبب خوردگی شدید آرمان‌های می‌شود. خوردگی به نفوذ‌بری بین و عمق کربناتاسیون در ارتباط با پوشش آرمان‌ها بسیار زیاد می‌شود.

هگامی که بین در اثر واکنش با یا اکسید کربن هوای کربناته می‌شود، کلریدهای بیوتین یافت آزاد می‌شوند. در اثر این فرآیند، غلت کلریدهای محلول در محیط منطقه کربناتاسیون زایدتر می‌شود و در نتیجه سبب هماجرت کلرید در داخل بین می‌شود.

۴ - کلریدهای پیوندی و آزاد
مهم‌ترین پیوند یون‌های کلرید، واکنش آنها با C₃A است که تشکیل کلسیم کلروآلمینیات می‌دهد. المشابهی انجام داده و تشکیل کلسیم کلروفریت می‌دهد. بنابراین وقتی مقادیر C₃A سیمان بالای باشد و همچنین وقتی مقادر C₄Af نیز واکنش C₃A سیمان بالای باشد و همچنین وقتی مقادر سیمان در مخلوط زیاد باشد، یون‌های کلرید بیشتری پیوند و قرار می‌گیرد. بنابراین این طور استنابیت می‌شود که استفاده از سیمان با C₃A بالا منجر به ایجاد مقاومت مناسبی در برابر خوردگی می‌شود. این مسئله ممکن است هنگامی صدق کند که این یون‌های کلرید موجود باشد و به سرعت با تبیج و یون‌های کلر کار خارج وارد بین شوند مقادیر کلروآلمینیات نشان می‌دهد.

همچنین ممکن است تحت شرایطی کلروآلمینیات تفکیک شده و یون‌های کلرید آزاد توسط آب منافع به سطح قولاً منتقل شوند. عامل دیگری که تعیین کننده مقدار مطلوب C₃A در سیمان می‌باشد مشکل حمل سولفات‌ها
است. شایان ذکر است که برای مقابله در برای سیمان تا است. است. قرار گرفته در معرض حمل سابقه متفاوتی که در جدول 2 میزان انواع سیمان C3A مشخص شده است.

<table>
<thead>
<tr>
<th>نوع سیمان</th>
<th>C3A</th>
<th>C3S</th>
<th>C2S</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>3.12</td>
<td>21.56</td>
<td>26.45</td>
</tr>
<tr>
<td>II</td>
<td>2.56</td>
<td>21.35</td>
<td>24.67</td>
</tr>
<tr>
<td>III</td>
<td>3.75</td>
<td>20.12</td>
<td>25.33</td>
</tr>
<tr>
<td>IV</td>
<td>3.98</td>
<td>21.35</td>
<td>24.67</td>
</tr>
<tr>
<td>V</td>
<td>3.12</td>
<td>21.56</td>
<td>26.45</td>
</tr>
</tbody>
</table>

اثر نوع و مقدار سیمان در نفوذ کلرید
در صورت نفوذ کلرید به داخل بنی و یا به دلیل آلوده بودن محصول بنی، بی‌حال‌های کلرید به سه حالت آزاد در محلول منافذ، چربی و محصولات هیدراتاسیون و بیوند یافتگان با C3A در بنی یافت می‌شود. از دیدگاه CaO, Al2O3, C3A ترکیبی که در 1/4 به 2 می‌شود و در نتیجه کلرید یا بیوند می‌یابد. بنابراین می‌توان نتیجه گرفت که نفوذ C2S سیمان برلند توسط مقدار C2S ترجمه داده. انتظار می‌رود که سیمان با نفوذ زیاد گردد که سیمان کلرید در بنی شود و در نتیجه زمان شروع نفوذ کلرید می‌باشد. این نظریه شاید درباره سیمان های پوزولانی صادق باشد، زیرا یکی از پوزولان‌ها دارای نفوذ زیادی یک گرد، کم‌یونده، اما توسط سازوکارهای دیگر (مانند نفوذ‌زردرنگ) سبب کاهش نفوذ یونه‌های کلرید می‌شود.

به‌هرحال در محیط‌هایی که بی‌حال‌های کلرید به عناوین مخلب محصول می‌شود، سیمان برلند معمولی نوع 1 با مقدار زیاد در مقایسه با سیمان برلند دیگر مقدار C3A (گم) عامل‌های مهم دارد. تحقیقات انجام شده تا که علاوه بر مقدار C3AF سیمان در بینون دانه بی‌حال‌های کلرید، عامل‌های دیگر نیز مؤثرند:

- ترکیبات سیمان
- ترکیبات محلول منافذ
- ترکیبات محلول نیاز در مقادیر بی‌حال‌های کلرید بی‌حال‌های اثر دارد. به عبارت دیگر، بی‌حال‌های موجود در محلول منافذ مانند OH−, SO3²− و HCO3− که به دیدگاه C3A و C3S نیست، بلاکه C3A و C3S نیست و همچنین C3S و C2S، ۱۵ می‌شود.

تزیاد قسمت‌های ظرفیت بی‌حال سیمان برای بی‌حال‌های OH−, SO3²− و C3A به‌طور کلی در سیمان با تغییرات در محیط‌های مختلف با شیمیایی و مادی و شرایط محیطی بستگی دارد. برای مثل اگر به مقدار SO3²− و OH− در منافذ بین الگو شود، از مقادیر بی‌حال‌های کلرید بی‌حال‌ها کاسته می‌شود، صرف می‌شود.
اثر نوع کاتیون‌ها

با تحقیقات انجام شده مشخص شده است که نوع کاتیون‌های کلرید در خودگردی آرسنات اثر دارد. وقتی مقدار مسالی ای اما با کاتیون‌های مختلف به علاوه سیمان پریلند افزوده شود، اثرات متفاوتی را نشان می‌دهد. در این تحقیق از KCl و NaCl، CaCl2 در ایجاد نشان می‌دهد که شدت خودگردی به‌طور قابل توجهی باید تا کاتیون بوده و این تغییر از استفاده در تغییرات آرامش مختلف به ریز گروه یا جایگزینی می‌تواند در نتیجه انتشار کلرید آسان انجام می‌شود و هدایت الکتریکی افزایش یابد. همچنین براساس تامام تحقیقات انجام شده مشخص شده است که کلرید سدیم باعث کاهش اندازه منافذ می‌شود. بنابراین تخلخل بتین تابع کاتیون‌های نیتروس و نوع مخلوط به می‌باشد.

بررسی‌های انجام شده در مرکز تحقیقات ساختمان و مسکن

بررسی‌های انجام شده در مرکز تحقیقات ساختمان و مسکن در سه زمینه به شرح زیر بوده است:
- بررسی تاثیر میزان C3A سیمان در ایجاد پیوند شیمیایی برون کلرید.
- بررسی تأثیر کاتیون‌های مختلف کلرید در ایجاد پیوند شیمیایی.
- بررسی روش‌های اندازه‌گیری کلرید آزار در سیستم‌های سیمانی.
- بررسی تأثیر بیان‌های مختلف (براساس نسبی‌های اسکوپمری) در آزمایشگاه و افزودن آن به نسبت‌های C3A در این راستا با ساختمان خلال C3A در میزان 92/8 و 10 درصد به سیمان و ساخت ملات با سیمان‌های ساختمان شده و افزایش کلرید سدیم و کلسیم به نسبت‌های C3A (0/1 و 1/2 درصد به آب اختلافی، روند پیوند کلریدی مورد بررسی قرار گرفته، بسیاری از عمل‌آوری نمونه‌های ساختمان و آماده‌سازی آنها براساس روش‌های استاندارد، نمونه کلرید مخلوط در آب و محلول در اسید آنها افزایش گیری شد.

همچنین به‌منظور بررسی اثر C3A در بین کلریدی در شرایط واقعی، از هواپیمای سیمان با دو مقدار مختلف C3A در پیوند کلریدی در 15 درصد، در ساخت نمونه‌ها استفاده شد. عمل‌آوری نمونه‌ها و آماده‌سازی آنها C3A مشابه نمونه‌های ساختمان شده به سیمان دارای C3A خالص بوده است.

اثر میزان C3A سیمان در بین کلریدی

- بررسی تاثیر به دست آمده از اندازه‌گیری کلرید نمونه‌های ساختمان شده با سیمان C3A خالص (ساخته شده در آزمایشگاه) با درصد‌هایی 10 و 12/16 درصد می‌باشد که با افزایش میزان C3A سیمان، پیوند کلریدی C3A می‌باشد.
نیز افزایش یافته است. همچنین کلریدهای محلول در آب (کلریدهای آزاد) نیز با افزایش \( C_3A \) تیز می‌شود. به‌دست داده که مواد کلریدی بیشتر است. در شکل‌های ۱ و ۲ روند افزایش پویوند کلریدی با افزایش میزان \( C_3A \) سیمان، به‌ترتیب در نمونه‌های ساخته‌شده با کلرید سدیم و کلرید کلسیم مشاهده می‌شود.

شکل ۱ - هیستوگرام روند افزایش کلریدهای پویوند با افزایش میزان \( C_3A \) در نمونه‌های ساخته شده با سیمان با \( C_3A \) خالص و کلرید سدیم

شکل ۲ - هیستوگرام روند افزایش کلریدهای پویوند با افزایش میزان \( C_3A \) در نمونه‌های ساخته شده با سیمان با \( C_3A \) خالص و کلرید کلسیم

بررسی نتایج به دست آمده از آنالیز گیری کلرید نمونه‌های ساخته شده با سیمان دارای \( C_3A \) مختلف (با \( C_3A \) درصد \( 30/72, 58/56, 75/57, 84/55 \) و \( 84/55, 75/57, 64/54 \) نشان می‌دهد که با افزایش میزان \( C_3A \) سیمان، پویوند کلریدی نیز افزایش یافته است. همچنین کلریدهای محلول در آب (کلریدهای آزاد) نیز با افزایش \( C_3A \) تیز می‌شود. به‌دست داده که مواد کلریدی بیشتر است.
در شکل‌های ۳ و ۴ روند افزایش یوند کلریدی با افزایش میزان C3A سیمان، به‌ترتیب در نمونه‌های ساختمان‌کرده با کلرید سدیم و کلرید کلسیم مشاهده می‌شود.

![شکل ۳- هیستوگرام روند افزایش کلریدهای یوند با افزایش میزان C3A (در نمونه‌های ساختمان‌کرده با سیمان دارای مختلف و کلرید سدیم)](image)

![شکل ۴- هیستوگرام روند افزایش کلریدهای یوند با افزایش میزان C3A (در نمونه‌های ساختمان‌کرده با سیمان دارای مختلف و کلرید کلسیم)](image)

تأثیر نوع کاتیون کلرید در ایجاد یوند کلریدی

- بررسی نتایج به دست آمده از آنالیز گیری کلرید نمونه‌های ساختمان‌کرده با سیمان با C3A خالص (ساخته شده در آزمایشگاه) با درصد‌های ۰، ۱۰ و ۱۲، نشان می‌دهد که در همه موارد با یک نسبت مشابه کلریدسدیم و کلرید کلسیم، نمونه‌های ساختمان‌کرده با یک کلرید کلسیم و یوند کلریدی بیشتری ایجاد کرده است.
- بررسی نتایج به دست آمده از آنالیز گیری کلرید نمونه‌های ساختمان‌کرده با سیمان دارای C3A مختلف با درصد‌های ۰/۲۴، ۰/۴۵، ۰/۵۷ و ۰/۷۷، نشان می‌دهد که در همه موارد با یک نسبت مشابه کلریدسدیم و
کلرید کلسیم، نمونه‌های دارای کلرید کلسیم، یکی از کلرید‌های بیشتری ایجاد کرده است.

در شکل 7 میزان کلرید یوند یافته در نمونه‌های ساخته شده با کلرید سدیم و کلرید کلسیم به نسبت‌های مختلف (سیمان با 12 درصد C3A خالص) مقایسه شده است.

در شکل 8 میزان کلرید یوند یافته در نمونه‌های ساخته شده با کلرید سدیم و کلرید کلسیم به نسبت‌های مختلف (سیمان دارای 11/57 درصد C3A) مقایسه شده است.

شکل 7- هیستوگرام میزان کلرید یوند یافته در نمونه‌های ساخته شده با کلرید سدیم و کلرید کلسیم به نسبت‌های مختلف (سیمان با 12 درصد C3A خالص).

شکل 8- هیستوگرام میزان کلرید یوند یافته در نمونه‌های ساخته شده با کلرید سدیم و کلرید کلسیم به نسبت‌های مختلف (سیمان دارای 11/57 درصد C3A).

کلرید عصاره گیری شده در سنگدانه و سیستم‌های سیمان

برخی سنگدانه‌ها حاوی مقداری کلرید هستند که در سنگدانه‌های محیط بوده و در واکنش خورشک شرکت نمی‌کنند. این نوع کلرید‌ها از طریق روی عصاره‌گیری اندازه‌گیری می‌شوند.

هنگامی که کلرید محلول در آب به مقدار کافی وجود داشته باشد، می‌تواند سبب خورشک فاقد نظیر فولاد شده

بررسی اثر C3A سیمان پرتند بر کاتیون‌های مختلف ..../ 99
که در داخل یک سیستم سیمان مانند ملات، گروت یا پن تار دارد یا در تماس با آنها یا باشند. این روش آزمون در مورد سنگدانه‌هایی که به طور طبیعی دارای کلرید هستند، قابل عمل است.

روش آزمون شرح داده در استاندارد ایران شماره 1387 بخشی از کلرید را که در این سنگدانه‌ها وجود دارد اندازه‌گیری می‌کند. شیب این که مقادیر کلرید اندازه‌گیری شده به شدت به درجه نرمی سنگدانه‌ها که هنگام نهایه نسبت به موندن سنگین تا نیم یکی سنگی افزایش می‌یابد.

در روش‌های ازون استاندارد ملی ایران شماره‌های 8946 و 8947 و به ترتیب کلریدهای محلول در آب و محلول در آب را تعیین می‌کند. در هر و یا پن نمونه به شکل پودر بسته یا به صورت مواد دانه‌ای ریز آسیاب می‌شود و کلریدهای محیوس در سنگدانه‌ها را که در واکنش خوردگی شرکت ندارند نیز اندازه‌گیری می‌کند.

روش اندازه‌گیری آیین نوع کلریدها در استاندارد ASTM C 1524-02a (ACI 222.1-96) ارائه شده است. در این روش با استفاده از دستگاه عصاره‌گیر Soxhlet مقدار بسیار کم کلرید را که در برخی سنگ‌ها باقی ماند و در واکنش خوردگی شرکت نمی‌کند، عصاره‌گیری می‌شود.

دستگاه عصاره‌گیر

به منظور بررسی کلریدهای محیوس در برخی از سنگدانه‌ها که در واکنش خوردگی شرکت نمی‌کند، از دستگاه عصاره‌گیر Soxhlet استفاده می‌شود.

این دستگاه متشکل از یک گرمکن، یک بانده ته صاف، یک نمونه‌گیر و یک میرد است. علما که دستگاه عصاره‌گیر به کمک یک گرمکن، یک بانده ته صاف، یک نمونه‌گیر و یک میرد است. علما که دستگاه عصاره‌گیر به کمک یک گرمکن، یک بانده ته صاف، یک نمونه‌گیر و یک میرد است. علما که دستگاه عصاره‌گیر به کمک یک گرمکن، یک بانده ته صاف، یک نمونه‌گیر و یک میرد است. علما که دستگاه عصاره‌گیر به کمک یک گرمکن، یک بانده ته صاف، یک نمونه‌گیر و یک میرد است. علما که دستگاه عصاره‌گیر به کمک یک گرمکن، یک بانده ته صاف، یک نمونه‌گیر و یک میرد است.

روشی که در گرمکن یک گرمکن، یک بانده ته صاف، یک نمونه‌گیر و یک میرد است. علما که دستگاه عصاره‌گیر به کمک یک گرمکن، یک بانده ته صاف، یک نمونه‌گیر و یک میرد است. علما که دستگاه عصاره‌گیر به کمک یک گرمکن، یک بانده ته صاف، یک نمونه‌گیر و یک میرد است. علما که دستگاه عصاره‌گیر به کمک یک گرمکن، یک بانده ته صاف، یک نمونه‌گیر و یک میرد است. علما که دستگاه عصاره‌گیر به کمک یک گرمکن، یک بانده ته صاف، یک نمونه‌گیر و یک میرد است.

در شکل 9 شماره این دستگاه مشاهده می‌شود.
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Maintenance
Repairing and
Strengthening
BENDING AND SHEARING STRENGTH OF T-FORMED BEAM MADE OF STRUCTURAL LIGHT CONCRETE USING FRP SHEETS

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ABSTRACT
To study enforcement effect of T-formed beams made of Structural lightweight concrete by binding CFRP with use of epoxy resin on the bending & shearing resistance, nine lab samples were designed, built and tested. These samples were divided into A, B & C groups, regarding their weakness. In all beams, two rods # 12 were used as compressive armature. In A, designing was such that beams have bending weakness. In these beams, two rods #12 were used as tensile armature and rod # 10 were used as shearing armature with a distance of 7.5 cm. In B, designing was such that beams have shearing weakness and 4 rods # 16 were used as tensile armature, the distance of stirrup was 30 cm. In C, designing was such that beams have shearing and also flexural weakness, In these beams, two rods # 12 were used as tensile armature and the distance of stirrups was 30 cm.

According to observed studies, in flexural strengthening, beams have had a strength increase of 60%, although breaking was shearing and CFRP sheets didn’t reach the rupture stage. Studying beam’s shearing strengthening represents loading capacity increase of 20% and breaking was as stratified cutting of concrete with CFRP sheet. Also, we observed 45% increase of beam’s loading capacity from bending and also shearing strengthening together. In this case, failure mode was as stratified cutting of concrete with CFRP sheet from the beam’s side.

Keyword: strengthening, tensile armature, CFRP, flexural

1. INTRODUCTION
Due to several causes such as damages resulting from corrosion or blasting severe wind, members weakness resulting from incorrect maintenance, damages from war or earthquake, usage changes, request for increasing foundation or number of stories and changes of used parameters, maybe the structures of armature concrete don’t have necessary ductility and resistance against imposed loads.[1]

To remove the deficiency of using steel plates in the strengthening of concrete members, using fiber reinforcing polymers (FRP) has some advantages. Several researchers have studied on bending and shearing strengthening by FRP sheets in worldwide scientific centers & universities. The polymeric sheets are not under the effect of corrosive factors, unlike the steel plates and are resistant against
damaging effects and also tolerate the relatively high temperatures well. So, using these sheets doesn’t need special arrangements before attachment and their maintenances are easier after the installation in comparison with steel plates.[2]

2. LABORATORIAL PLAN
In this study, nine T-formed beams of light concrete with a total length of 1900mm were built and tested over supports with a span of 1800mm. Lab samples are in A, B and C groups and each group has 3 beams. For labs, one is as un-strengthened and the other two are strengthened based on the kind of weakness in the beam.

3. SPECIFICATIONS OF USING MATERIALS
3.1. Armature
Internal armature is ribbed bar and from A2 type. The yielding stress for bars of 16 is 3300 kg/cm², for bars of 12, 3800 kg/cm² and for bars of 10 3650 kg/cm². The results of the using steel bars are given in.

<table>
<thead>
<tr>
<th>Number</th>
<th>Steel bar’s diameter mm</th>
<th>Area cm²</th>
<th>Yield stress kg/cm²</th>
<th>Rupture stress kg/cm²</th>
<th>Percentage of length increase</th>
<th>Elasticity modulus kg/cm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>10</td>
<td>0.785</td>
<td>3650</td>
<td>5600</td>
<td>15</td>
<td>2039000</td>
</tr>
<tr>
<td>2</td>
<td>12</td>
<td>1.13</td>
<td>3800</td>
<td>5600</td>
<td>15</td>
<td>2039000</td>
</tr>
<tr>
<td>3</td>
<td>16</td>
<td>2.01</td>
<td>3800</td>
<td>5600</td>
<td>15</td>
<td>2039000</td>
</tr>
</tbody>
</table>

3.2. Concrete
Regarding the first mix designs, an effort has been made to obtain a suitable mixture regarding the effective factors on the compressive resistance increase and also economic and administrative conditions.

To provide needed experimental concrete, the following mixing ratio was used.

<table>
<thead>
<tr>
<th>Material</th>
<th>Material weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>450</td>
</tr>
<tr>
<td>Water</td>
<td>180</td>
</tr>
<tr>
<td>Gravel</td>
<td>234</td>
</tr>
<tr>
<td>Sand</td>
<td>561.6</td>
</tr>
<tr>
<td>Lecca</td>
<td>374.4</td>
</tr>
<tr>
<td>Super Plasticizer</td>
<td>6.75</td>
</tr>
<tr>
<td>Water to cement ratio</td>
<td>0.4</td>
</tr>
</tbody>
</table>

Lecca grains are products of Saveh lecca factory and the fine grains were used in this plan. The used sand was provided from washed one in the Ganjafrooz mine in Babol. The kind of used cement is type, II Portland produced in Neka cement.
factory. The using superplastilizer is PCE from Vandshime factory and the water is from urban potation one.

3.3. Glus
To make glue, a mix including two parts of epoxy resin and one part of hardening is used that will gain its complete stick after one week.

3.4. CFRP Sheet
The CFRP sheets used in beam strengthening are uni-lateral yarn with the yielding stress of 3800 and elasticity modulus of 240,000 MPa. Their thickness is 0.11 mm.

4. LOADING SYSTEM
Loading system is shown in Figure (6-3). In this system, we used rigid metal frames assumed to install jack. Samples are put on the part of this rigid frame for beam support. In this study, beams are tested as Simple beam.

5. MEASURING TOOLS
Measuring special buttons of strain are installed on the beam lateral surface in its upper and lower arrays with a distance of 200mm. These buttons are installed by using concrete glue on the beam in order to measure concrete strain and its stick strengthening sheets by strain gage.

5.1. Data Logger
The available Data Logger is cr10x that has high compatibility in data recording & sending, even through modem & internet.

5.2. Samples Introduction
In this study, nine T-formed beams of light concrete with a total length of 1900mm were loaded and tested on the supports with a span of 1800mm. The compressive strength of used concretes in all beams has been designed for $f_c=410$ kg/cm².
5.3. “A” Group Beams
“A” Group relates to the beams having weakness in bending. In this group, we used two ribbed bars # 12 as tensile armature and two ribbed bars #12 as compressive ones. For shearing armature, we used ribbed bars # 10 as rectangular ones with the distance of 75mm axis to axis.
Selecting the above armature for A was due to strength of these beams in shearing & their weakness in bending. The specifications of “A” are given in figure 5.

5.4. “B” Group Beams
“B” Group relates to the beams having weakness in shearing. In this group, we used four ribbed bars # 16 as tensile armature in two double arrays and two ribbed bars # 12 as compressive ones.
For shearing armatures, we used ribbed bars # 10 as a rectangular form with a distance of 300 mm from axis to axis. The specifications are given in figure 6.

5.5. “C” Group Beams
C Group relates to the beams having both bending & shearing weakness. In this group, we used two ribbed bars # 12 as tensile armature and two ribbed bars # 12 as compressive ones. For shearing armatures, we used ribbed bars # 10 as rectangular ones with a distance of 300 mm axis to axis. The specifications are given in Figure 7.

Figure 2. Specifications relating to “A” beams
Figure 3. Specifications relating to “B” beams
Figure 4. Specifications relating to “C” beams
6. BEAM’S STRENGTHENING METHOD

6.1. Strengthening of A Grap Beams
“A” beams are included in A1, A2 and A3 with the same specifications. In this study, we regard A3 as a reference beam and strengthen A1 and A2. To strengthen A1 and A2, we used uni-lateral CFRP sheets by the yielding stress of 3800 Mpa and elasticity modulus of 240,000 Mpa. The width of reinforced sheet is 16 cm and its length is 160cm. Its pure thickness is 0.11mm and its strengthening method is given in Figure 8.

6.2. Strengthenin of B Grap Beams
“B” beams are included in B1, B2 and B3 with the same specifications. In this study, we regard B1 as a reference beam and strengthen B2 and B3; we used CFRP sheets having a width of 7cm & length of 48cm in U form. Its strengthening method is given in Figure 9.

6.3. Strengthening of “C” Group Beams
“C” beams include C1, C2 and C3 with the same specifications. In this study, we regard C1 as a reference beam and strengthen C2 and C3. To strengthen this beam, we used first the FRP sheet having a width of 16cm & length of 160cm for Flexural strengthening of the beam. Then, we used the 48*7cm sheets for shearing strengthening (figure 10). The cause of installing bending CFRP first and then shearing CFRP is that bending CFRP transfers the tensile forces relatively to the CFRP and this decreases the possibility of debonding risk beneath concrete.
6.4. Results
Cracking ultimate loads and strength failure mode:
Tables 3, 4 and 5 represent cracking loads, and increasing percentage in comparison to the control beam, and also ultimate load that can be carried by beams resulting from carrying out strengthening as well as maximum deflection in beams middle Span while rupturing in 3 different groups.

### Table 3: Results of "A" group beams

<table>
<thead>
<tr>
<th>Sample</th>
<th>Compressive strength kg/cm²</th>
<th>Ultimate load t</th>
<th>Increase of ultimate load relative to control beam %</th>
<th>Maximum deflection mm</th>
<th>Bending cracking load kN</th>
<th>Shearing cracking load kN</th>
<th>Load increase of bending crack %</th>
<th>Load increase of Shearing crack %</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>A₁</td>
<td>525</td>
<td>112</td>
<td>-</td>
<td>14.8</td>
<td>29</td>
<td>59.6</td>
<td>-</td>
<td>-</td>
<td>Bending accompanied with yielding of tensile bars shearing</td>
</tr>
<tr>
<td>A₂</td>
<td>525</td>
<td>175</td>
<td>56%</td>
<td>1.5</td>
<td>43</td>
<td>60</td>
<td>48%</td>
<td>0.67%</td>
<td>Shearing with CFRP removing accompanied with a layer of concrete</td>
</tr>
<tr>
<td>A₃</td>
<td>540</td>
<td>189</td>
<td>69%</td>
<td>10</td>
<td>46</td>
<td>59.6</td>
<td>59%</td>
<td>0</td>
<td>Shearing with bursting compressive flange</td>
</tr>
</tbody>
</table>

### Table 4: Results of "B" group beams

<table>
<thead>
<tr>
<th>Sample</th>
<th>Compressive strength Kg/cm²</th>
<th>Ultimate load kN</th>
<th>Increase of ultimate load relative to control beam %</th>
<th>Maximum deflection mm</th>
<th>Bending cracking load kN</th>
<th>Shearing cracking load kN</th>
<th>Load increase of bending crack %</th>
<th>Load increase of Shearing crack %</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>B₁</td>
<td>550</td>
<td>185</td>
<td>-</td>
<td>8.8</td>
<td>43</td>
<td>55</td>
<td>-</td>
<td>-</td>
<td>Shearing with bursting compressive flange</td>
</tr>
<tr>
<td>B₂</td>
<td>560</td>
<td>210</td>
<td>14%</td>
<td>9</td>
<td>43</td>
<td>111</td>
<td>0</td>
<td>102%</td>
<td>Shearing with CFRP removing accompanied with a layer of concrete</td>
</tr>
<tr>
<td>B₃</td>
<td>560</td>
<td>225</td>
<td>22%</td>
<td>9.4</td>
<td>55</td>
<td>140</td>
<td>28%</td>
<td>155%</td>
<td>Shearing with bursting compressive flange</td>
</tr>
</tbody>
</table>

### Table 5: Results of "C" group beams

<table>
<thead>
<tr>
<th>Sample</th>
<th>Compressive strength Kg/cm²</th>
<th>Ultimate load kN</th>
<th>Increase of ultimate load relative to control beam %</th>
<th>Maximum deflection mm</th>
<th>Bending cracking load kN</th>
<th>Shearing cracking load kN</th>
<th>Load increase of bending crack %</th>
<th>Load increase of Shearing crack %</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>C₁</td>
<td>555</td>
<td>105</td>
<td>-</td>
<td>21.8</td>
<td>32</td>
<td>56</td>
<td>-</td>
<td>-</td>
<td>Shearing</td>
</tr>
<tr>
<td>C₂</td>
<td>545</td>
<td>150</td>
<td>43%</td>
<td>19.6</td>
<td>40</td>
<td>75</td>
<td>25%</td>
<td>34%</td>
<td>Shearing</td>
</tr>
<tr>
<td>C₃</td>
<td>550</td>
<td>141</td>
<td>34%</td>
<td>15.7</td>
<td>38</td>
<td>71</td>
<td>19%</td>
<td>27%</td>
<td>Shearing</td>
</tr>
</tbody>
</table>

Shearing with CFRP removing accompanied with a layer of concrete shearing.
6.5. Comparison of “A” Group Beams
In loading “A” group beams, there are some results. The first created bending crack in A3 was in a load of 2.9 ton, while in A1 and A2 it was 4.3 and 4.6 tons. It was observed that the first crack of these beams is created in a load of 1.5 times for A1 and 1.6 times for A2.
The first created shearing crack in A3 was in a load of 5.96 tons, while in A1 and A2 it was 6 and 5.9 tons. It was expected that these beams would reach the first shearing crack in the same load.
The A3 failing was in a load of 11.2 tons and it was flexural failure with yielding tensile bars, while in A1 and A3, failing was in 17.5 and 18.9 tons and shearing. As it was observed, the created strength increase in A1 was about 56% and in A2, about 70%. We should consider the fact that the real amount of loading capacity increase could be much more than this. Because, first the breaking of A1 and A2 was shearing and the beam fails before using its total flexural capacity. Secondly, if we compare the imposed load in the first crack, we can see that the load of the first crack in A1 and A2 was about 1.5 times for A. So, the flexural strength increase of A2 and A1 is much more than the observed amount.

6.6. Comparing Of “B” Beams
Comparing B1, B2 and B3, we can see that the first bending crack in B1 was in load 4.3 ton, while it was expected that the first bending crack of B1 and B2 is created in the same load. But the first bending crack in B3 was in 5.5 tons that is a little more than the other event. The first shearing crack in B1 was in 5.5 ton, while in B2 and B3; it was in 11.1 and 14 tons that increases the resistance against the first shearing crack. The B1 failure was in 18.5 tons and shearing and for B2 and B1 it was 21 and 22.5 tons and shearing. As we can see, the increase of B2 strength against B1 was about 15% and B3 against B1, was about 22% that is not remarkable. Due to the shearing failure of B2 and B3, we conclude that there is no suitable strengthening, beams still have shearing weakness and CFRP sheets haven’t been broken, but they were removed by a concrete layer. This shows that CFRP sheets can still tolerate more loads.

6.7. Comparing “C” Beams
Comparing C1, C2 and C3, we can see that the first bending crack is created in 3.2 for C1 and 4 and 38 tons for C2 and C3.
The first created bending crack in C1 was about 5.6 tons and in C2 and C3 it was about 7.5 and 7.1 tons, and its increase against C1 was 35% for C1 and 28% for C3. The breaking of C1 was in 10.5 tons and shearing, while for C2 and C3 it was in 15 and 14.1 tons and shearing. As we can see, there is a good strength increase that is 45% for C2 and 35% for C
7. DISCUSSION AND RESULTS
7.1. Strain in Strengthened Frp
Observing figures 8-10 relating to the load-compressive strain and load-tensile strain graph we can see increasing of compressive strain and decreasing of tensile strain in a defined load, that can be due to displacement of neutral cord because of strengthening.
8. CONCLUSION

Based on experiments and calculations, we obtained the following results:

1. The modes of shearing failure in strengthened beam were:
   a) Resulting from FRP failure, b) without FRP failure, c) resulting from FRP debonding.
2. Bending & shearing strengthening on different samples, shows that the compressive strain increases remarkably.
3. With strengthening the beams, the cracking load in beams increases.
4. Deflection of flexural strengthened beams is lower than non- strengthened one.
5. Strengthening the beams, the neutral cord moves upwards at a lower speed.

REFERENCES

INVESTIGATION OF BENDING BEHAVIOR OF REINFORCED CONCRETE BEAMS STRENGTHENED WITH CFRP SHEETS

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1Ph.D Department of Civil Engineering, Islamic Azad University of Lahijan. Iran
2M.Sc. Department of Civil Engineering, Islamic Azad University of Lahijan. Iran

ABSTRACT
The effect of FRP (Fiber Reinforced Polymers) sheets on bending strength of beams is one of the advantages of utilizing carbon fibers in concrete structures. By utilizing FRP sheets, reinforcing bar ratio which is used as longitudinal tensile reinforcements would increase in specimens and bending strength would be improved. In this research study, by testing 12 concrete beam specimens with known dimensions with 3 different reinforcing bar ratios the effect of FRP in flexural behavior strength, displacements, ultimate load and stiffness of the concrete beams have been investigated. The results show that in addition to increased strength, failure may occur with high adequate ductility in reinforced concrete beams.

Keywords: FRP sheets, bending strength, concrete beam, bar ratio

1. INTRODUCTION
Retrofitting and strengthening of a constructed structure are currently very significant in modern civil engineering. One of the modern methods in strengthening concrete structures is utilizing fiber reinforced polymers (FRP) bonded to concrete beams as strips made of carbon fibers [3]. This method has several advantages over traditional ones, especially increasing high strength, decreasing beam weight and creating durability of concrete structures. Based on experimental results obtained by Teng et al, Bonacci, Maalej and Feo [1, 8], the most common failure mode is derived from debonding of FRP plate or ripping of the concrete cover. In addition, some premature failures are generally associated with reduction in deformability of the strengthened tensile members.[ 2]. Numerous experiments have been carried out to determine failure mode and behavior of concrete beams. [5]. Based on existing studies typical failure modes observed in experiments is shown in Figure.1. [4,8]. These failure modes are type (1), type (2), type (3) and type (4) as the following schematic representation [8].
Figure 1. (a) - failure type -1-

Concrete Crushing

Figure 1. (b) failure type -2-

FRP Rupture

Figure 1. (c) failure type - 3-

High Stress zone

Crack Propagation

Figure 1. (d) failure type – 4 (a)

High Stress zone

Crack Propagation

Figure 1. (e) failure type – 4 (b)

High Stress zone

Crack

Load

Figure 1. (f) failure type – 4 (c)

Figure 1. failure modes of concrete beams
According to Sebastian and Teng the corrosion of tensile longitudinal steel bars, changing of reinforcing bar ratio and shear forces may increase the probability of these types of failures [6]. The ratio of reinforcing bar of beams affects the above patterns and bending behavior and the width of cracks.[8]. The influence of FRP, bond around tensile longitudinal bars, on flexural strengthening of reinforced concrete beams and the ductility of beams are investigated in this paper by the test results of 12 beam specimens strengthened by carbon fiber reinforced polymers (CFRP).

2. EXPERIMENTAL SET UP
In order to perform research on the materials some tests carried out have been introduced.

2.1. Materials
For the beam specimens the compressive strength is 240 MPa. The concrete mixture proportions are shown in Table 1.

<table>
<thead>
<tr>
<th>Coarse aggregate</th>
<th>sand</th>
<th>cement</th>
<th>water</th>
</tr>
</thead>
<tbody>
<tr>
<td>750</td>
<td>1000</td>
<td>300</td>
<td>160</td>
</tr>
</tbody>
</table>

*-maximum size of aggregate is 12 mm

Also different sizes of tensile bars which have been used in beams are 8,10,12,16 and 20mm. The yield and ultimate strength of bars is indicated in Table 2.

<table>
<thead>
<tr>
<th>Diameter</th>
<th>8</th>
<th>10</th>
<th>12</th>
<th>16</th>
<th>20</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yield stress (MPa)</td>
<td>350</td>
<td>365</td>
<td>400</td>
<td>420</td>
<td>450</td>
</tr>
<tr>
<td>Ultimate stress (MPa)</td>
<td>460</td>
<td>570</td>
<td>575</td>
<td>585</td>
<td>590</td>
</tr>
</tbody>
</table>

Mechanical properties of CFRP sheets are presented in Table 3.

<table>
<thead>
<tr>
<th>Layer thickness (mm)</th>
<th>Ultimate strain (mm)</th>
<th>Tensile strength (MPa)</th>
<th>Modulus of elasticity (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.170</td>
<td>0.0160</td>
<td>3750</td>
<td>230</td>
</tr>
</tbody>
</table>

Also the stress – strain relationship is sketched in Figure 2. The adhesive used for binding the CFRP sheet on the concrete surface is hand-mixed epoxy and the air between concrete surface and CFRP sheet is removed. The adhesive curing time is 6 days according to instructions of the manufacturer.[9,11]
2.2. Experiments on Specimens
12 concrete beam specimens with dimensions according to Figure 3 are manufactured. The reinforcing bar ratios are 20%, 40% and 70% of the tensile reinforcement balanced ratio.

The dimensions and details of reinforced specimens are shown in Table 4.

<table>
<thead>
<tr>
<th>Specimens</th>
<th>CFRP width (mm)</th>
<th>$f_c$ (MPa)</th>
<th>$A_g$ (mm$^2$)</th>
<th>$A_s$ (mm$^2$)</th>
<th>d (mm)</th>
<th>d$'$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1</td>
<td>0</td>
<td>240</td>
<td>226</td>
<td>157</td>
<td>170</td>
<td>25</td>
</tr>
<tr>
<td>B2</td>
<td>10</td>
<td>240</td>
<td>226</td>
<td>157</td>
<td>170</td>
<td>25</td>
</tr>
<tr>
<td>B3</td>
<td>20</td>
<td>240</td>
<td>226</td>
<td>157</td>
<td>170</td>
<td>25</td>
</tr>
<tr>
<td>B4</td>
<td>25</td>
<td>240</td>
<td>226</td>
<td>157</td>
<td>170</td>
<td>25</td>
</tr>
<tr>
<td>B5</td>
<td>0</td>
<td>240</td>
<td>402</td>
<td>157</td>
<td>170</td>
<td>25</td>
</tr>
<tr>
<td>B6</td>
<td>10</td>
<td>240</td>
<td>402</td>
<td>157</td>
<td>170</td>
<td>25</td>
</tr>
<tr>
<td>B7</td>
<td>20</td>
<td>240</td>
<td>402</td>
<td>157</td>
<td>170</td>
<td>25</td>
</tr>
<tr>
<td>B8</td>
<td>25</td>
<td>240</td>
<td>402</td>
<td>157</td>
<td>170</td>
<td>25</td>
</tr>
<tr>
<td>B9</td>
<td>0</td>
<td>240</td>
<td>628</td>
<td>157</td>
<td>170</td>
<td>25</td>
</tr>
<tr>
<td>B10</td>
<td>10</td>
<td>240</td>
<td>628</td>
<td>157</td>
<td>170</td>
<td>25</td>
</tr>
<tr>
<td>B11</td>
<td>20</td>
<td>240</td>
<td>628</td>
<td>157</td>
<td>170</td>
<td>25</td>
</tr>
<tr>
<td>B12</td>
<td>25</td>
<td>240</td>
<td>628</td>
<td>157</td>
<td>170</td>
<td>25</td>
</tr>
</tbody>
</table>
As shown in Table 4, seven specimens are strengthened by CFRP sheets. Also three specimens are kept as control specimens without strengthening. After loading the deflection of the specimens the strains at the mid-spans are measured by gauge. Also, the strain of concrete at the level of the tensile and compressive reinforcing bars and the strain of CFRP sheets at the mid-span of beam are measured by gauge according to Figure 4:

![Strain gage](https://via.placeholder.com/150)

**Figure 4. Measuring instruments**

The output data are recorded by a computer.

### 2.3. Results and Discussion

The control specimens B1, B5 and B9 failed after straining of tensile bars in a very ductile manner. In B2 failure occurred due to fracture of CFRP sheet but the beam carried a higher load than B1. Specimens B3 and B4 failed in Type three due to high shear and normal stresses at the ends of the CFRP sheets due to debonding of CFRP. B6, B7 and B8 failed due to fracture of CFRP sheets (Type 2) after yielding of reinforcing bars. Specimens B10 failed in Type 2 due to fracture of CFRP sheets around the mid-span after yielding reinforced bars. Specimens B11 and B12 failed in Types (4-b) and (4-c). In both of them debonding of CFRP sheet started due to shearing cracks. Compared to other specimens more shearing cracks with closer spacing occurred in B11 and B12.

Figure 6(a)-(c) shows the load versus mid-span displacement relationship of beams. According to these Figures, at earlier stages, before flexural cracking the load-displacement curves are close to each other. With increasing the load, the strengthened specimens exhibited larger stiffness. After yielding of reinforcing bars, the strength and stiffness of the strengthened specimens were larger compared to the control specimens. In specimens B11 and B12, the load – displacement curves continued without dropping and failure was initiated due to separation of FRP. However, the specimens failed due to crushing of concrete with adequate ductility as indicated in Figure 6 (c).

As shown in Figure 6, by increasing the load, the strengthened specimens demonstrate larger stiffness. After yielding tensile bars the strength and stiffness of specimens reinforced by CFRP are larger compared to the control specimens. Also, after failure of CFRP the load-displacement curves of strengthened drops.
Figure 6 (a). B1, B2, B3 and B4

Figure 6 (b). B5, B6, B7 and B8

Figure 6 (c). B9, B10, B11 and B12
The displacement and ultimate strength $P_u$ of concrete beams are shown in Table 5. The increase in strength of beams reinforced by CFRP sheets which varies with the reinforcing bar ratio are also submitted in this table.

<table>
<thead>
<tr>
<th>Specimens</th>
<th>Bar ratio/</th>
<th>$P_u$ (kN)</th>
<th>$P = P_{u0} - Pu$ (kN)</th>
<th>$P/P_{u0}$ displacement at $Pu$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1</td>
<td>0.2</td>
<td>39.6</td>
<td>0</td>
<td>22.3</td>
</tr>
<tr>
<td>B2</td>
<td>0.2</td>
<td>51.3</td>
<td>11.7</td>
<td>29.5%</td>
</tr>
<tr>
<td>B3</td>
<td>0.2</td>
<td>62.58</td>
<td>22.98</td>
<td>58%</td>
</tr>
<tr>
<td>B4</td>
<td>0.2</td>
<td>64.5</td>
<td>24.9</td>
<td>62.8%</td>
</tr>
<tr>
<td>B5</td>
<td>0.4</td>
<td>65.6</td>
<td>0</td>
<td>-</td>
</tr>
<tr>
<td>B6</td>
<td>0.4</td>
<td>73.5</td>
<td>7.9</td>
<td>12%</td>
</tr>
<tr>
<td>B7</td>
<td>0.4</td>
<td>82.66</td>
<td>17.06</td>
<td>26%</td>
</tr>
<tr>
<td>B8</td>
<td>0.4</td>
<td>95.43</td>
<td>20.83</td>
<td>45.5%</td>
</tr>
<tr>
<td>B9</td>
<td>0.7</td>
<td>85.3</td>
<td>7.91</td>
<td>9.2%</td>
</tr>
<tr>
<td>B10</td>
<td>0.7</td>
<td>93.21</td>
<td>10.33</td>
<td>12.1%</td>
</tr>
<tr>
<td>B11</td>
<td>0.7</td>
<td>95.63</td>
<td>10.33</td>
<td>18.5</td>
</tr>
<tr>
<td>B12</td>
<td>0.7</td>
<td>105.44</td>
<td>20.14</td>
<td>23.6%</td>
</tr>
</tbody>
</table>

2.4. Comparison Between Experimental Results and Theoretical Predictions

According to proposed equations by ISIS Canada, a linear variation over the depth of concrete section and the value of 0.0035 for the maximum concrete strain are being considered [3]. Also ISIS supposes the reduction factors of 0.6, 0.85 and 0.75 for concrete, steel and FRP sheet respectively [3]. The ratios of ultimate test loads to the calculated values supposed by ISIS are given in Table 6.

<table>
<thead>
<tr>
<th>Specimens</th>
<th>bar ratio/bar balance</th>
<th>$P_{test}$ (kN)</th>
<th>$P_{ISIS}$ (kN)</th>
<th>$P_{test}/P_{ISIS}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1</td>
<td>0.4</td>
<td>39.6</td>
<td>32.4</td>
<td>1.22</td>
</tr>
<tr>
<td>B2</td>
<td>0.4</td>
<td>51.3</td>
<td>49.3</td>
<td>1.07</td>
</tr>
<tr>
<td>B3</td>
<td>0.4</td>
<td>62.58</td>
<td>59.5</td>
<td>1.09</td>
</tr>
<tr>
<td>B4</td>
<td>0.4</td>
<td>64.5</td>
<td>53.2</td>
<td>1.09</td>
</tr>
<tr>
<td>B5</td>
<td>0.8</td>
<td>65.6</td>
<td>55.6</td>
<td>1.18</td>
</tr>
<tr>
<td>B6</td>
<td>0.8</td>
<td>73.5</td>
<td>61.7</td>
<td>1.19</td>
</tr>
<tr>
<td>B7</td>
<td>0.8</td>
<td>82.66</td>
<td>71.3</td>
<td>1.16</td>
</tr>
<tr>
<td>B8</td>
<td>0.8</td>
<td>95.43</td>
<td>80.5</td>
<td>1.19</td>
</tr>
<tr>
<td>B9</td>
<td>1.0</td>
<td>95.63</td>
<td>78.2</td>
<td>1.24</td>
</tr>
<tr>
<td>B10</td>
<td>1.0</td>
<td>93.21</td>
<td>83.9</td>
<td>1.11</td>
</tr>
<tr>
<td>B11</td>
<td>1.0</td>
<td>95.63</td>
<td>88.5</td>
<td>1.08</td>
</tr>
<tr>
<td>B12</td>
<td>1.0</td>
<td>105.44</td>
<td>98.5</td>
<td>1.07</td>
</tr>
</tbody>
</table>

Comparing the test results of specimens B2, B6, B7, B8, B10, B11 and B12, ISIS overestimates the ultimate bending strength in the case of strengthened beams with
small reinforcing bar ratios. According to Table 6 by increasing the reinforcing bar ratio in concrete beams, the ratio of $P_{test}$ / $P_{ISIS}$ increases. Therefore, the equations proposed by ISIS are more appropriate for concrete beams with high reinforcing bar ratios. [8,10].

3. CONCLUSION
Generally from the test results and calculated values the following conclusion has been obtained:
1. The flexural strength and stiffness of RC beams increases by CFRP.
2. While the reinforcing bars increases, the ratio of the test load to the Load calculated ($P_{test}$ / $P_{ISIS}$) increase.
3. With high reinforcing bars near balanced reinforcement ratio failure of the concrete beams occurs in either Type - 4 (b) and 4 (c) with adequate ductility.

ACKNOWLEDGMENTS
The authors wish to thank the University of Science and Research Islamic Azad University (Tehran) and the Islamic Azad University of Lahijan for their experimental instruments and scientific labors.

REFERENCES
SHAPE AND DIMENSIONAL EFFECT ON BEHAVIOR OF CONCRETE COLUMNS CONFINED WITH FRP SHEETS

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ABSTRACT

In recent years, the use of FRP jackets for strengthening of weak concrete columns has become increasingly popular. The confinement effect of the externally bonded FRP systems in rectangular and square sections of concrete columns is known to be complicated and less than those in circular sections. In this article, the predictive design equations for calculating the compressive strength of FRP-confined concrete columns in three current international guidelines and five theoretical models have been introduced and effective parameters in each of them have been investigated. For a comparative study, also, a database consists of experimental results of 43 prismatic specimens of different cross-sectional shapes and confinement details have been collected. The analytical results show that the section’s shape and dimensions are effective parameters in diversity of experimental and theoretical results. Also, by defining the effective factors, a model has been expanded for prismatic FRP-confined concrete columns. Based on the analytical results, the expanded model indicates acceptable predictions in comparison with the other models and guidelines.

Keywords: FRP, concrete column, confinement, shape factor, compressive strength

1. INTRODUCTION

In recent years, earthquake damage to many reinforced concrete columns in bridges and buildings has indicated inadequate strength and deformation of reinforced concrete columns built before the 1970’s and urgent need to retrofit them [1]. These structures were rather constructed according to older codes or without an adequate construction practice. The structural members of this type of buildings may experience severe damage due to low deformability and axial capacity (Figure 1). The initial application for retrofitting of weak columns involved the use of steel hoops and straps to provide lateral confinement. Some analytical models have been developed to provide a theoretical base for retrofitting concrete columns [1]. These models are satisfactory for the prediction of strength and ductility of concrete columns confined by steel stirrups.
Rather to some disadvantages of steel jacket, such as heavy weight and high potential for corrosion, the use of fiber reinforced polymer (FRP) composites has been developed in recent decades. This material has some unique properties such as light weight, high stiffness and high strength to weight ratio. Moreover, FRP has great resistance to corrosion. These new materials have shown a great potential in replacing the traditional steel reinforcement as retrofit material. Based on the results of many experimental researches, when reinforced concrete columns confined laterally with FRP sheets, its ductility and axial load capacity will be enhanced [2].

2. CONFINING EFFECT OF FRP

The strength enhancement in columns using lateral FRP sheets may be the confinement effect of transverse fiber sheets. When a concrete column is affected by axial compressive load, concrete core will expand laterally. In jacketed column, however, lateral expansion is limited by the effect of lateral confining material. In these cases concrete core of the column section will be affected by a kind of passive pressure named confining stress. An important aspect of the behavior of confined concrete is that at the rupture of FRP, the hoop strain reached in the jacket is generally considerably smaller than the ultimate tensile strain found from flat coupon tensile tests. The FRP efficiency factor had been suggested for calculation of the actual hoop rupture strain of FRP jacket. According to the stress distribution of confined circular section (Figure 2), confining pressure provided by the transverse FRP sheet ($f_i$) is given by [3 and 4]:

$$f_i = \left(\frac{2}{D}\right)E_{FRP}k_sE_{FRP}t_{FRP}$$  \hspace{1cm} (1)
Where $D =$ diameter of column section, $t_{FRP} =$ whole thickness of FRP sheets, $k_e =$ FRP efficiency factor and $E_{FRP}$ and $\varepsilon_{FRP} =$ the modulus of elasticity and ultimate tension strain of FRP sheet. Based on experimental results the advantages of FRP in circular section column are different from rectangular section. In a circular concrete column, the confining pressure is constant around circumference and small variation due to factors such as in homogeneity of concrete is ignored. In rectangular section of columns, the confining pressure of FRP does not distribute uniformly over the section and only a portion of the section is affected by confining pressure (Figure 3). Because of it the performances of FRP in this kind of sections are different and lower than that of FRP-confined circular sections [4].

In rectangular section, due to stress concentration in FRP jacket, premature failure of FRP occurs and whole capacity of FRP is not used. In this section, the confining pressure provided by the FRP sheet must be decreased by introducing the shape factor that is less than 1.0. In rectangular sections it is given by [5]:

$$f_i = k_s k_e E_{FRP} \varepsilon_{FRP} t_{FRP}$$

(2)

Where $k_s =$ shape factor of section and is related on section’s geometrical
dimensions and often different in each model.

3. EXISTING MODELS FOR FRP-CONFINED CONCRETE
Most of the available models for evaluating the compressive strength of FRP-confined concrete columns are based on the confinement model that was derived experimentally for specimens under active hydrostatic pressure [6]. In this article, five existing models for rectangular and square columns are investigated. Those models had been presented by Mirmiran et al. [7], Pantelides and Yan [8], Al-Salloum [9], Lam and Teng [2], and Ilkki et al. [10] and summarized in Table 1. a, b, and r in this table are large having a small dimension and corner radius of cross section. Lateral confining pressure $f_l$ in each model is calculated from Eq. (2) where FRP efficiency factor and shape factor in each model are given in Table 2.

4. REVIEW OF DESIGN GUIDELINES
The document considered in this article is as follows: “Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures” reported by the American Concrete Institute (ACI Committee 440.2R-02 2002) [11], “Design and Construction of Building Components with Fiber Reinforced Polymers” Reported by the Canadian Standard Association (CSA S806-02 2002) [12], and “Externally Bonded FRP Reinforcement for RC Structures” Technical Report by the Fédération Internationale du béton (fib Bulletin 14 2001) [13]. Each of the design guidelines has some limitations and conditions in nonlinear cross sections, which are related to the type of compressive load application, maximum dimensions, maximum side-aspect-ratio ($a/b$), and minimum corner radius of cross section ($r$). In Table 3 the mentioned limitations in each guideline had been summarized. Approach presented by the current ACI committee 440 (ACI 2002) for compressive strength enhancement is conservative. This guideline specifies that although confining square and rectangular members with FRP jackets can provide marginal increases in the axial compression strength of the member, there are no recommendations provided at this time on the use of FRP. The model provided by ACI guideline for estimation ductility of confined rectangular column is given as follows:

$$f'_{cc} = f'_{c0} \left( 2.25 \sqrt{1 + 7.9 f_l / f'_{c0}} - 1.25 \right) - 2 f_l$$

Eq. (14) had been primary presented for steel-confined concrete. Some research showed that it is applicable for the case of FRP-confined concrete [2]. According to the ACI guideline FRP confining pressure in prismatic column can be obtained by Eq. (2) where the shape factor $k_s$ and FRP efficiency factor $k_e$ are calculated by Eqs. (10) and (15):

$$k_e = MIN \left\{ 0.75, \frac{0.004}{\epsilon_{FRP}} \right\}$$
Table 1: Estimating models for compressive strength of confined concrete column.

<table>
<thead>
<tr>
<th>Author's name</th>
<th>Compressive strength of confined concrete column</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mirmiran et al.</td>
<td>( f_{cc} = \left(1 + 6.0 \frac{f_i^{0.7}}{f_{c0}}\right) f_{c0} ) for ( \frac{f_i}{f_{c0}} \geq 0.15 ) (3)</td>
</tr>
<tr>
<td></td>
<td>( f_{cc} = \left[-4.322 + 4.271 \left(1 + 4.193 \frac{f_i}{f_{c0}} - 2 \frac{f_i}{f_{c0}}\right) f_{c0}\right] )</td>
</tr>
<tr>
<td></td>
<td>Pantelides and Yan</td>
</tr>
<tr>
<td></td>
<td>( f_{cc} = \max \left[\left[-4.322 + 4.271 \left(1 + 4.193 \frac{f_i}{f_{c0}} - 2 \frac{f_i}{f_{c0}}\right) f_{c0}\right], \frac{f_i}{f_{c0}}\right] )</td>
</tr>
<tr>
<td></td>
<td>( f_{cc} = \left(1 + 3.14 \frac{f_i}{f_{c0}}\right) f_{c0} ) (6)</td>
</tr>
<tr>
<td>Al-Salloum</td>
<td>( f_{cc} = \left(1 + 3.3 \frac{f_i}{f_{c0}}\right) f_{c0} ) (7)</td>
</tr>
<tr>
<td>Lam and Teng</td>
<td>( f_{cc} = \left(1 + 2.54 \frac{f_i}{f_{c0}}\right) f_{c0} ) (8)</td>
</tr>
</tbody>
</table>

Table 2: FRP efficiency factor and section's shape factor in each model

<table>
<thead>
<tr>
<th>Author's name</th>
<th>FRP efficiency factor ( k_s )</th>
<th>Section's shape factor ( k_s )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mirmiran et al.</td>
<td>1.0</td>
<td>( k_s = \frac{4r}{a^2} ) (9)</td>
</tr>
<tr>
<td>Pantelides and Yan</td>
<td>0.5</td>
<td>( k_s = \left[1 - \frac{(a - 2r)^2 + (b - 2r)^2}{3ab}\right] \frac{(a + b)}{ab} ) (10)</td>
</tr>
<tr>
<td>Al-Salloum</td>
<td>1.0</td>
<td>( k_s = \frac{2a}{\sqrt{2a - 2 - \sqrt{2 - 1}}} \left[1 - 2 \frac{(1 - 2(a/r))^2}{3(1 - (4 - \pi)(r/a)^2)}\right] ) (11)</td>
</tr>
<tr>
<td>Lam and Teng</td>
<td>0.57</td>
<td>( k_s = \left(\frac{b}{a}\right)^2 \left[\frac{2}{\sqrt{a^2 + b^2}}\right] \left[1 - \frac{(b/a)(a - 2r)^2 + (a/b)(b - 2r)^2}{3(ab - (4 - \pi)r^2)}\right] ) (12)</td>
</tr>
<tr>
<td>Illki et al.</td>
<td>0.85</td>
<td>( k_s = \left(\frac{a + b}{ab}\right) \left[1 - \frac{(b/a)(a - 2r)^2 + (a/b)(b - 2r)^2}{3(ab - (4 - \pi)r^2)}\right] ) (13)</td>
</tr>
</tbody>
</table>

Regarding CSA S806-02 guideline (CSA2002), the maximum confined concrete compressive strength is given by Eq. (16). This equation is similar to well-known equation provided by Richart et al. [2].

\[ f_{cc0}^{*} = 0.85 f_{c0}^{*} + 2.12 f_i^{0.83} \] (16)
Table 3: Design Guidelines Limitations and conditions

<table>
<thead>
<tr>
<th>Guideline</th>
<th>Kind of Loading</th>
<th>Side Dimensions (mm)</th>
<th>Ratio of Side Dimension</th>
<th>Corner Radius (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ACI</td>
<td>Concentric Axial loading</td>
<td>( b, a \leq 900 )</td>
<td>( a/b &lt; 1.5 )</td>
<td>( r \geq 13 )</td>
</tr>
<tr>
<td>CSA</td>
<td>Concentric Axial loading</td>
<td>-</td>
<td>( a/b \leq 1.5 )</td>
<td>( r \geq 20 )</td>
</tr>
<tr>
<td>fib</td>
<td>Concentric Axial loading</td>
<td>-</td>
<td>-</td>
<td>Recommended: ( d: 15 \leq r \leq 25 )</td>
</tr>
</tbody>
</table>

Confining pressure in above equation is calculated by Eq. (2), where FRP efficiency factor is calculated from Eq. (15) and shape factor is given by:

\[
k_s = \left( \frac{2}{D} \right), D = \text{lesser of } a \text{ and } b \quad (17)
\]

The design recommendations provide by *fib* for prismatic columns are based on the model proposed by Spoelstra and Monti [2]. In this code, the maximum amount of confined concrete compressive strength determined from cube calculated is given by:

\[
f'_{cc0} = f'_{c0} + 0.2 + 3 \left( \frac{f_f}{f'_{c0}} \right) \quad (18)
\]

Confining pressure in *fib* is determined from Eq. (2), in which shape factor are given by Eq. (13). *fib* highlights that the hoop rupture strain of the FRP jacket, based on experimental evidence, is lower than the ultimate strain obtained by tensile testing of the material. The guideline points out that this reduction is due to several reasons, such as the quality of construction, the size effect when applying several layers, the effect of wrapping the material on the corners of low radius, and the combined state of stress of the FRP wrapping. Because of the lack of data no appropriate reduction factors are suggested in this guideline. In other words *fib* provides FRP efficiency factor equal to the value of 1.0 for a confinement by full wrapping. In this design guideline, FRP safety factors \( \gamma_f \) are applied individually to each of the material components of the FRP during the computation of lateral confining pressure.

Table 4: FRP safety factors in normal control quality

<table>
<thead>
<tr>
<th>FRP type</th>
<th>CFRP</th>
<th>AFRP</th>
<th>GFRP</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \gamma_f )</td>
<td>1.20</td>
<td>1.25</td>
<td>1.30</td>
</tr>
</tbody>
</table>
––––––––––––––––––––––– 3rd International Conference on Concrete & Development / 897

Table 5: Experimental details of FRP-confined square and rectangular concrete
specimens
t frp
ε frp
E frp
Exp. f cc′
a
b
r
FRP
f c′0
Authors
No.
Parvin and
Wang
Al-Salloum

Kumutha
et al.

Shehata
et al.

Rochette
and
Labossiere

Rochette
and
Labossiere

Pessiki et
al.

1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
21
22
23
24
25
26
27
28
29
30
31
32
33
34
35
36
37
38
39
40
41
42
43

(mm)

(mm)

(mm)

108
108
150
150
150
150
125
125
140
140
161
161
188
188
188
188
152
152
152
152
152
152
152
152
152
152
152
152
152
152
152
152
152
152
152
152
152
203
203
203
203
152
152

108
108
150
150
150
150
125
125
112
112
97
97
94
94
94
94
152
152
152
152
152
152
152
152
152
152
152
152
152
152
152
152
152
152
152
152
152
152
152
152
152
152
152

8.26
8.26
5
25
38
50
0
0
0
0
0
0
10
10
10
10
5
25
25
38
38
5
25
25
25
38
38
5
5
5
5
25
25
25
25
38
38
25.0
38.0
5
25
38
38

(MPa)
21.40
21.40
29.81
30.16
29.00
27.49
34.31
34.31
34.31
34.31
34.31
34.31
23.70
23.70
29.50
29.50
42.00
42.00
42.00
42.00
42.00
43.90
43.90
35.80
35.80
35.80
35.80
43.00
43.00
43.00
43.00
43.00
43.00
43.00
43.00
43.00
43.00
42.00
42.00
43.90
43.90
26.40
26.40

type

CFRP
CFRP
CFRP
CFRP
CFRP
CFRP
GFRP
GFRP
GFRP
GFRP
GFRP
GFRP
CFRP
CFRP
CFRP
CFRP
CFRP
CFRP
CFRP
CFRP
CFRP
CFRP
CFRP
CFRP
CFRP
CFRP
CFRP
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AFRP
AFRP
AFRP
AFRP
AFRP
AFRP
AFRP
AFRP
AFRP
CFRP
CFRP
CFRP
CFRP
CFRP
CFRP

(GPa)
188.9
188.9
75.1
75.1
75.1
75.1
10.5
10.5
10.5
10.5
10.5
10.5
235
235
235
235
82.7
82.7
82.7
82.7
82.7
82.7
82.7
82.7
82.7
82.7
82.7
13.6
13.6
13.6
13.6
13.6
13.6
13.6
13.6
13.6
13.6
82.7
82.7
82.7
82.7
38.1
38.1

(%)
1.60
1.60
1.00
1.00
1.00
1.00
3.50
3.50
3.50
3.50
3.50
3.50
9.10
4.60
9.10
4.60
0.23
0.56
0.63
0.71
1.61
0.44
0.59
0.70
0.65
0.89
0.86
0.79
1.30
1.48
0.90
1.12
1.27
0.94
1.04
1.05
0.97
1.50
1.50
1.50
1.50
0.83
0.90

(mm)
0.138
0.268
1.200
1.200
1.200
1.200
0.680
1.360
0.680
1.360
0.680
1.360
0.165
0.330
0.165
0.330
0.900
0.900
0.900
0.900
0.900
1.500
1.200
1.200
1.500
1.200
1.500
1.260
2.520
3.780
5.040
1.260
2.520
3.780
5.040
2.520
3.780
0.900
0.900
1.500
1.500
1.000
2.000

(MPa)
36.630
45.230
41.840
46.920
55.960
62.680
50.300
60.160
49.410
58.880
49.280
55.040
25.810
33.200
25.710
38.700
39.48
41.58
43.26
47.46
50.40
43.90
50.92
52.27
57.64
59.43
68.74
50.74
51.60
53.75
54.18
51.17
51.17
53.32
55.04
50.74
52.89
42.000
43.680
44.340
44.340
41.40
55.10

These material safety factors are summarized in Table 4 and used as dividers.
These are mainly based on the observed differences on the long term behavior of
composites (basically depending on the type of fibers), as well as influence of
application methods.
4.1. Test Database
A large number of existing studies have been concerned with the compressive
behavior of rectangular concrete columns confined by wrapped FRP. In this article


for a comparative study of mentioned model with actual results, a test database containing a total of 43 FRP-confined plain concrete rectangular specimens have been assembled from experimental studies of Parvin and Wang [14], Kumutha et al. [15], Shehata et al. [16], Rochette and Labossiere [17], Pessiki et al. [18], and Al-Salloum [9] presented in Table 5. Those data are over square and rectangular specimens with the section depth ranging from 94 to 203 mm, and the corner radius from 0 to 50 mm. The unconfined concrete strength of these specimens ranges from 21.4 to 43.9 MPa. Lateral confinement of these specimens was provided by aramid FRP (AFRP), carbon FRP (CFRP) or glass FRP (GFRP).

4.2. Performance of Models and Guidelines

The compressive strength of each experimental case (Table 5) and the theoretical results of each of the models and guidelines are presented in Figure 4. It is observable that if any experimental case doesn’t satisfy the limitation and conditions of each of the models and guidelines, corresponding lateral confining pressure is considered equal to zero. As Figure 4 shows, the models of Mirmiran et al. and ACI and CSA guidelines, significantly underestimate the compressive strength (Figure 4(a) and (h)). According to the Figure 5, models of Lam and Teng, Illki et al, and Al-Salloum predict the correct trend for both square and rectangular specimens with different amount of geometrical dimensions and lateral confining pressure (Figure 4(d), (e), and (c)). Models of Lam and Teng, and Al-Salloum suppose less than 1.0 amount for FRP efficiency factor. But this factor in Illki et al.’s model is equal to 1.0. The results of Panteledis and Yan’s model and fib guidelines are overestimated for rectangular cross sections (Figure 4(b) and (g)). In those models, by increasing the side-aspect-ratio \((a/b)\), the deviation of theoretical results are noticeable. Regarding the amount of the FRP efficiency factor and the shape factor in Panteledis and Yan, it can be concluded that the suggested relation for calculating the compressive strength of confined concrete in this model is affected by cross section of columns and its side-aspect-ratio. Overestimation of fib guideline can depend on the amount of FRP efficiency factor presented in this guideline.

4.3. Proposed Model for Prismatic Columns

Kheyroddin et al. [19] suggested a model for calculating compressive strength of concrete column confined by FRP which is given as follows:

\[
f'_{cc0} = f'_{c0} \left[ 0.622 + \frac{f_t}{f'_{c0}} + 1.577 \sqrt{\frac{f_t}{f'_{c0}} + 0.058} \right]
\]  

(19)

In this model that is suggested for columns with circular cross sections, lateral confining pressure is calculated from Eq. (1). For adaptation of the above equation for prismatic FRP-confined columns, FRP efficiency factor and shape factor of section should be defined. According to the structure of models that have been
investigated, the shape factor $k_s$ in this model suggested is the same as shape factor in Lam and Teng as given by Eq. (12).

![Figure 4. Performance of the models and the guidelines](image)

By a trial and error method the FRP efficiency factor $k_e$ for this model is obtained
as the value equal to 0.650. The performance of the new proposed model is presented in Figure 6. As this figure shows, Kheyroddin et al.’s model with adopted factors predicts acceptable result for prismatic concrete columns confined by FRP in the range of collected experimental database.

4.4. Accuracy of Models

Cusson and Paultre [20] in their comparative studies between experimental and theoretical results applied index error, as follows:

\[
\text{Index Error} = \sqrt{\frac{\sum (\text{Experimental} - \text{Theoretical})^2}{\text{Experimental}}} 
\]  

(20)

This equation has been used for all predicted results of models and guidelines for collected database and obtained results are comparable with the proposed model in Table 6. According to the obtained results, Kheyroddin et al.’s with suggested factors has been indicated acceptable approximations in comparison with other models and guidelines.

<table>
<thead>
<tr>
<th>Models</th>
<th>Mirmiran et al.</th>
<th>Pantelides and Yan</th>
<th>Al-Salloum</th>
<th>Lam and Teng</th>
<th>Ilki et al.</th>
<th>ACI</th>
<th>CSA</th>
<th>fib</th>
<th>Proposed Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Index Error</td>
<td>1.82</td>
<td>3.08</td>
<td>1.65</td>
<td>1.38</td>
<td>1.36</td>
<td>1.90</td>
<td>2.11</td>
<td>4.34</td>
<td>1.37</td>
</tr>
</tbody>
</table>

5. CONCLUSIONS

In this paper, the performance of some current models and guidelines for estimating the compressive strength of FRP-confined concrete columns has been investigated. Studied models and guidelines had been affected by section’s shape. Some of models such as Mirmiran et al.’s model and ACI and CSA underestimate the compressive strength. The results of Panteledis and Yan's model and fib guidelines are overestimated for rectangular cross sections. Models of Ilki et al.,
Lam and Teng, and Al-Salloum had acceptable predictions. Index error in those models was 1.36, 1.38, and 1.65. Moreover, by definition some effective factors, a model had been expanded for prismatic FRP-confined concrete columns showed the proposed model has acceptable predictions in comparing to the other investigated models and guidelines. Index of error for this model obtained is equal to 1.37.

REFERENCES


STUDYING THE EFFECT OF FREEZE AND THAW CYCLES ON BOND STRENGTH OF CONCRETE REPAIR MATERIALS

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ABSTRACT
The mechanisms of damage to concrete from repeated cycles of freezing and thawing are not well understood and continue to be intensively studied. Original research was based on the fact that water expands 9 percent when it freezes. Further researches proposed more mechanisms. Hydraulic pressure theory proposes that destructive stresses can develop if water is displaced to accommodate the advancing ice front in concrete. If the pores are critically saturated, water will begin to flow to make room for the increased ice volume. The concrete will rupture if the hydraulic pressure exceeds its tensile strength. In this paper the results of an experimental study of the effect of freeze & thaw cycles on the bond between repair materials and concrete substrate is presented. The work was aimed at studying the effect of various factors such as initial curing periods and surface preparation method on bond strength. Old concrete samples were made based on BS6319, Part 4 standard. Smooth as-sawn and acid etching methods were used for preparation of concrete substrate surface. Ordinary concrete with cement type II and concrete containing microsilica were used as repair materials. Repaired samples were subjected to 10 to 100 freeze and thaw cycles based on ASTM C666. The bond between repair materials and concrete substrate was evaluated based on slant shear test method (BS, 1984). The obtained results are tabulated and presented in this paper.

Keywords: freeze & thaw, concrete, repair, bond strength

1. INTRODUCTION
The serviceability of construction materials in general is of significant economic importance. This is especially so with structures and materials which are part of the infrastructure of a modern society. Concrete is a material heavily used in urban development, meeting the requirements of codes of practices by means of strength and durable structures. Reduced service life of concrete members in the sense of lack of durability may be due to a number of different reasons, e.g. planning/capacity (over loading), improper structural or material design, construction practice or inadequate maintenance – or lack of knowledge. Widespread use of de-icing salts in many parts of the world is considered one of the major cause of rapid degradation of concrete structures. Further, the de-icing salt together with repeated freezing and thawing may cause failure of the concrete
cover by surface scaling, which combined with steel corrosion may critically reduce the structure’s service life. It is very difficult to estimate the direct repair and maintenance costs caused by freeze-thaw damages of concrete structures. However, due to its still nonrevealed secrets concerning deterioration mechanisms, freeze-thaw resistance has received significant attention for several decades. The mechanisms of damage to concrete from repeated cycles of freezing and thawing are not well understood and continue to be intensively studied. Original research was based on the fact that water expands 9 percent when it freezes. Thus, the term "critical saturation" was coined to describe the point at which the concrete pores were 91.7 percent saturated and, therefore, assumed to be susceptible to damage due to freezing and thawing. Further investigation determined that deterioration due to freezing and thawing can affect concrete with lower degrees of saturation [1].

Four theories have gained wide acceptance in describing the mechanisms of frost action. Although most of these theories were originally used to describe the frost action in cement paste, they are also applicable to concrete [2]. The first was the hydraulic pressure theory Powers proposed in 1945. This was followed by the diffusion and growth of capillary ice theory constructed by Powers and Helmuth in 1953, the dual mechanism theory by Larson and Cady in 1969, and the desorption theory by Litvan in 1972. Other theories have been proposed, but these four form the basis of most research in the area of frost resistance of concrete.

While these theories disagree as to whether water moves toward or away from the point of ice formation, they agree that the amount of water in the pores and the resistance to movement of that water play a role in the frost resistance of concrete. In the case of concrete, it is generally accepted that the pore system is potentially susceptible to damage from freezing and thawing. Efforts to produce frost-resistant concrete have primarily focused on providing a proper system of entrained air voids. In the case of aggregates, some pore systems do not show susceptibility to damage from freezing and thawing while other pore systems do. In addition to the air-entrainment of concrete as mentioned above, efforts have also focused on identifying the aggregates with acceptable pore systems for use in concrete exposed to freezing and thawing.

The causes of concrete deterioration have always been the object of concern and research. This interest is increasing due to the high cost associated with the repair and maintenance of the concrete structure. Repairs, however, are successful in the long-term if the causes of the original damage have been understood and appropriate repair materials are applied to resist future deterioration. Repair materials should be compatible with old concrete and have good adhesion. In repair of concrete, the bond strength between repair materials and old concrete is of vital importance.

The objective of this study was to investigate the effect of freeze and thaw cycles on bond strength of repair materials. Strength and integrity of the bond depends on not only the physical and chemical characteristics of the repair component, but also other factors such as surface preparation method and environmental conditions. The effects of these factors were studied in this work.
2. EXPERIMENT

Old concrete samples were made based on BS6319 Part 4 standard [3]. It is shown that surface preparation method has significant effect on bond strength [4], therefore, two methods including, smooth as-sawn and acid etching were used in order to prepare the surface of old concrete samples. Ordinary concrete, made with type II portland cement, and concrete containing 15 percent microsilica were used as repair materials. Repaired samples were subjected to 10 to 100 freeze and thaw cycles based on ASTM C666 procedure B. The bond between repair materials and concrete substrate was evaluated based on slant shear test method (BS, 1984).

2.1. Mix Proportions

Type II portland cement (ASTM C 150 specification) was used in this research. Crushed stone with a maximum size less than 9.5 mm and sand with a fineness modulus of 2.9 were used for producing concrete. The composition of old concrete mixes (OC mix) was 0.5:1.0:2.35:1.04 (water: cement: sand: gravel) by weight. The uniaxial compressive strength of old concrete samples was 35 MPa. MSOC mix was produced with replacement of 15% of cement in OC mix (by weight) with microsilica in order to investigate the effect of microsilica on bond strength.

2.2. Specimen Preparation

Old concrete samples were made based on BS 6319: Part 4 standard, figure 1. They were cast as 55x100x150 mm prisms and cured in water for 28 days in laboratory. Then cut at 30 deg to the vertical axis using a diamond saw. The acid etching method with use of hydrochloric acid was used to prepare the surface of 1/2 of samples.

For acid etching, with reference to ACI committee 549 [4], a hydrochloric acid solution was chosen. HCl can primarily react with the Ca(OH)_2 of the hydrated cement paste to form CaCl_2, making the substrate more porous. Because no adequate information concerning the influence of acid consistency on bond strength was available in current literature, hydrochloric acid solutions of 5% were chosen for testing [5]. The etching of surface was carried out in such a way that the hydrochloric acid solution was brushed on the surface of concrete substrate with a
2.3. Repair and Test Procedure

Concrete samples were formed in the moulds in which they were cast. The repair material, OC or MSOC mixes, was then applied and hand-compacted. Samples were stripped after 24 hr and placed in curing tank for 7, 14, and 28 days.

To evaluate the effect of freeze and thaw cycles on bond strength of repair materials, concrete samples after curing were subjected to 10 to 100 freeze and thaw cycles based on ASTM C666 Procedure B, figure 2. The loss of the weight in the concrete specimens were also measured and recorded.

A wide range of test method has been proposed to evaluate bond properties and performance of repair materials in general. The slant shear test has become the most widely accepted test for evaluating the bond of resinous repair materials to concrete. However, there seems to be no standard test for testing the bond to concrete of cementitious and modified cementitious repair materials. To compare the bond strength of repair materials, slant shear test method was used in this work. This method, which puts the bond interface into a combined state of compression and shear is adopted in BS6319: Part4 [3], was used as a test method for evaluating bond strength of repair materials, Figure 3.
2.4. Test Results and Discussion

Strength and integrity of the bond depends on not only the physical and chemical characteristics of the repair component, but also other factors such as initial curing periods, surface preparation method and environmental conditions. The effects of these factors were studied in this work. The obtained results are presented and discussed briefly in this section.

The results given in Table 1, shows the effect of initial curing periods on durability of samples subjected to 100 freeze and thaw cycles. Based on the results, the curing period has an important effect on durability of concrete samples subjected to freeze and thaw cycles.

As shown in the above table with increase in curing period weight loss decreases. For concrete samples repaired with ordinary concrete, the weight loss has decreased from 3.05% to 2.11% with increase in curing period from 7 to 28 days, respectively, which means 31% increase in durability, or in other words, increase in resistance to freeze and thaw cycles. With increase in strength of repair material, the weight loss of samples is decreased. Based on the obtained results, samples repaired with MSOC material show about 30% more resistance to freeze and thaw cycles than concrete samples repaired with OC material. For concrete samples repaired with microsilica concrete, the weight loss has decreased from 2.1% to 1.43% with increase in curing period from 7 to 28 days, respectively, which means 32% increase in resistance to freeze and thaw cycles.

To study the effect of freeze and thaw (F&T) cycles on bond strength, samples were cured for 28 days in curing tank and then were subjected to 10 to 100 freeze and thaw cycles based on ASTM C666 Procedure B. Test results are given in Table 2. Based on the obtained results freeze and thaw action decreases the bond strength considerably. With increase in number of F&T cycles bond strength decreases. After 100 cycles of F&T, the bond strength of samples repaired with OC and MSOC materials is reduced by 85.6% and 61.2%, respectively. Moreover, microsilica concrete not only increases the bond strength [6-7], but also increases durability regarding F&T cycles.

<table>
<thead>
<tr>
<th>Repairing Material</th>
<th>OC</th>
<th>MSOC</th>
</tr>
</thead>
<tbody>
<tr>
<td>Curing Period (day)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>14</td>
<td>28</td>
</tr>
<tr>
<td>14</td>
<td></td>
<td></td>
</tr>
<tr>
<td>28</td>
<td>7</td>
<td>14</td>
</tr>
<tr>
<td>Weight Loss (%)</td>
<td>3.05</td>
<td>2.25</td>
</tr>
<tr>
<td></td>
<td>2.1</td>
<td>1.64</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Repairing Material</th>
<th>OC</th>
<th>MSOC</th>
</tr>
</thead>
<tbody>
<tr>
<td>No of F&amp;T cycles</td>
<td>0</td>
<td>10</td>
</tr>
<tr>
<td>Bond Strength (MPa)</td>
<td>21.5</td>
<td>20.4</td>
</tr>
<tr>
<td></td>
<td>17.6</td>
<td>10.3</td>
</tr>
<tr>
<td></td>
<td>24.0</td>
<td>23.3</td>
</tr>
<tr>
<td></td>
<td>19.5</td>
<td>15.1</td>
</tr>
</tbody>
</table>
In Figure 1, the bond strength of samples subjected to F&T cycles is compared to that of samples which were not subjected to F&T cycles (FT/NFT).

![Figure 1. Ratio of bond strength of samples subjected to F&T with that of observation samples](image)

As the above figure shows, the bond strength of repair materials is not much affected in the first 50 cycles of F&T and reduction is less than 20 percent. However, during the second 50 cycles, the bond strength of repair materials reduces sharply in both OC and MSOC materials. As can be seen, reduction of bond strength in OC repair materials is more than that of MSOC repair material.

Two surface preparation methods, smooth as-sawn (SS) and acid etching (AE) were used to prepare the MSOC samples. After repair, samples were cured for 28 days in curing tank and then were subjected to 70 freeze and thaw cycles. Slant shear test was used for evaluating the bond of MSOC repair materials to concrete. In Table 2, the 28-day bond strength of samples is given.

<table>
<thead>
<tr>
<th>Surface Preparation Method</th>
<th>SS</th>
<th>AE</th>
</tr>
</thead>
<tbody>
<tr>
<td>No of F&amp;T cycles</td>
<td>50</td>
<td>70</td>
</tr>
<tr>
<td>Bond Strength (MPa)</td>
<td>19.5</td>
<td>15.1</td>
</tr>
</tbody>
</table>

Based on the obtained results, the surface preparation method has a considerable
effect on bond strength of repair materials subjected to F&T cycles. With use of acid etching method, the bond strength of MSOC repair material is increased by 13%, 16%, and 19% after 50, 70, and 100 F&T cycles, respectively, compare to those of samples prepared by SS method.

3. CONCLUSIONS
In this study the effect of Freeze and thaw cycles on bond strength of cementitious repair material is investigated. The work was aimed at studying the effect of various factors such as initial curing periods and surface preparation method on bond strength. Old concrete samples were made based on BS6319, Part 4 standard. Smooth as-sawn and acid etching methods were used for preparation of concrete substrate surface. Ordinary concrete (OC) with cement type II and concrete containing microsilica (MSOC) were used as repair materials. Repaired samples were subjected to 10 to 100 freeze and thaw cycles based on ASTM C666B. The bond between repair materials and concrete substrate was evaluated based on slant shear test method (BS, 1984). The following conclusions can be drawn from the obtained results:

2) the curing period has an important effect on durability of concrete samples subjected to freeze and thaw cycles. With increase in curing period weight loss of samples decreases. In this study, for concrete samples repaired with ordinary concrete, the weight loss has decreased with increase in curing period from 7 to 28 days by 31%.

3) freeze and thaw phenomena decreases the bond strength considerably. With increase in number of F&T cycles bond strength of repair materials decreases. After 100 cycles of F&T, the bond strength of samples repaired with OC and MSOC materials reduced by 85.6% and 61.2%, respectively. Moreover, microsilica concrete not only increases the bond strength but also increases durability regarding F&T cycles.

4) with application of an effective surface preparation method one can improve the bond strength of repair material considerably. In this study, with use of acid etching method, the bond strength of MSOC repair material could be increased by 13%, 16%, and 19% after 50, 70, and 100 F&T cycles, respectively, compare to that of samples with smooth as sawn surface.

REFERENCES
5. Xiong, G., Cui, Y, Chen, L and Jiang, H. Influence of hydrochloric acid
AN EXPERIMENTAL STUDY ON STRUCTURAL BEHAVIOR OF RECTANGULAR RC COLUMNS DAMAGED BY REBARS CORROSION AND STRENGTHENING THEM WITH FRP

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¹MSc. Student, Dept of Civil Engg., School of Engg., Tehran University, Tehran, Iran
²Assistant Professor, School of Engg. Tehran University, Tehran, Iran

ABSTRACT
This paper presents the results of an experimental study on the structural behavior (strength and ductility) of rectangular RC columns which have been damaged by rebar corrosion. 22 small-scale reinforced concrete columns with dimensions of 160×160 mm in section and 340 mm in height were tested. Ten specimens were conditioned to three levels of accelerated corrosion and six were conditioned to natural corrosion. The specimens were subjected to concentric compression load in order to assess the change of their mechanical properties due to the corrosion effects. Twelve specimens were strengthened with carbon and glass fiber reinforced polymer (CFRP and GFRP) to see the efficiency of different strengthening schemes. Based on this research it was concluded that the damaged columns show less strength and ductility in comparison with two undamaged columns and, FRP wraps could greatly enhance the strength and ductility of damaged specimens.

Keywords: corrosion, fibre reinforced polymer (FRP), strength, ductility, strengthening

1. INTRODUCTION
The structural degradation of concrete structures, due to reinforcement corrosion is a major worldwide problem. For instance, corrosion of reinforcement in bridge piers is encouraged by chloride contamination from exposure to marine environment and from deicing salts used in bridges during winter. Premature failure of RC structures due to corrosion of reinforcement is a significant issue. Corrosion products generally occupy greater volume than the original material; expansive forces are generated in concrete leading to cracks and spalling of the cover, reducing steel cross-section, deterioration of bond between reinforcement and concrete and finally further acceleration of the reinforcement disintegration [2] (Figure 1). Jacketing of such structures by fiber reinforced composite sheets is an effective remedy, not only as a means of slowing down the rate of the reaction, but also by confining the concrete core thereby imparting to it ductility and strength [3]. Fiber-reinforced polymers (FRP), consisting of continuous carbon (C), glass (G), or aramid (A) fibers bonded together in a matrix of epoxy, vinyl ester, or polyester, are being employed extensively for rehabilitation of concrete structure.
Despite their relatively high material costs, the high strength-to-weight ratio of FRP, their immunity to corrosion, and easy handling and installation are making them the material of choice in an increasingly large number of rehabilitation projects [7].

In order to ensure safety of reinforced concrete structures whose reinforcing steel has been severely corroded, it is necessary not only to repair the damage appropriately, but also to evaluate the strength of RC members according to the degree of rebar corrosion [3, 4]. RC columns, are the main members of RC structures, usually sustain axial forces of dead loads and live loads. There are currently some reports on beams subjected to bending, but only a few attempts have so far been made at columns, in which axial force is not predominant [4]. The authors of this paper try to carry out some experimental studies on the structural behaviour of rectangular RC columns damaged by accelerated rebar corrosion in different levels and natural rebar corrosion under concentric axial load. In this research also strengthening method using carbon fibre and glass fibre sheets (CFS and GFS) and the comparison between them is considered.

2. EXPERIMENTAL PROGRAM

2.1. Material and Design

Twenty two specimens were made in the laboratory. All the specimens had a 160 mm square cross-section and a 340 mm height between the test-region. The mix proportion and the mechanical properties of the specimens are given in table 1. The rebars mechanical properties are given in table 2. Figure 2 shows the geometric details of the specimens.

![Figure 1. Spalling of the covers and Corrosion cracks.](image)

<table>
<thead>
<tr>
<th>Table 1: Mixing and mechanical properties of concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>W/C (%)</td>
</tr>
<tr>
<td>W</td>
</tr>
<tr>
<td>-----------------</td>
</tr>
<tr>
<td>68</td>
</tr>
</tbody>
</table>
Table 2: Mechanical properties of rebars

<table>
<thead>
<tr>
<th>Type</th>
<th>Yield Strength (MPa)</th>
<th>Tensile Strength (MPa)</th>
<th>Elastic modulus (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>D16</td>
<td>575</td>
<td>696</td>
<td>197</td>
</tr>
<tr>
<td>D6</td>
<td>260</td>
<td>435</td>
<td>172</td>
</tr>
</tbody>
</table>

2.2. Conditioning to Accelerated and Natural Corrosion

After 28 days of curing, 10 of the 22 specimens were connected to the electrochemical corrosion cell aiming to stimulate the corrosion. The chemical effects can be created naturally around the reinforcement and concrete cover by depletion of iron and rust accumulation. Although admittedly, the exact chemical composition of rust produced in natural conditions is hard to duplicate under electrochemical conditions, by securing adequate supply of oxygen to the corrosion cell, the aim here was to generate expansive rust products that would cause a network of fine cracks to build upon the specimen surface similar to what is seen in the field. Based on earlier research, a mass loss of approximately 5% calculated with Faraday’s Law, assuming constant rate of steel consumption and uniform corrosion over the reinforcing cage is a critical threshold for generating crack widths of 0.2-0.4 mm that are thought to correspond to the Serviceability Limit State of a structure [8]. To this end, specimens were placed in a corrosion basin containing 3% by weight water solution of NaCl. The reinforcement cage of each specimen was connected to the circuit so as to serve as the anode in the corrosion cell, whereas an external steel bar immersed in the basin was used as cathode. Anode and cathode were connected to a constant power supply of 6 V. This voltage has been found to be suitable for generation of similar corrosion products as would occur in nature in a realistic time period so as to enable systematic study of the depletion process and rust accumulation in the laboratory [3, 6]. The electric current passing through each specimen was measured by interpolating ampere.
meters between anode and the power supply. Three levels of corrosion were considered by the volume of integrated electric current. The categories of the levels were as follows: level one 435 hours, level two 653 hours and level three 870 hours were considered. Figure 3 shows the mechanism of accelerated corrosion applied through each specimen.

![Figure 3. Mechanism of accelerated corrosion.](image)

The propagation of cracks due to corrosion at each level was observed. Figure 4 shows the overall views of cracking in the specimens after carrying out electrolytic corrosion. Although map cracking has been observed, most of the cracks were alongside of the reinforcement and it looks as cracks in the longitudinal directions were more than the lateral directions. The characteristic of the corrosion observed was that the rust concentrated on or near corrosion cracks and on the corners of hoops. The reason for this seems to be that cracks are prone to water infiltration and that the corners of the hoops are under high stresses induced when being bent in the preparation of rebars [4].

![Figure 4. Overall views of cracking in the specimens](image)

The depleted mass of iron $\Delta W$ (gr) consumed over the time $\Delta t$ (s) was estimated from the current $I_{corr}$ flowing through the cell using Faraday’s Law (Equation 1), which assumes a constant rate of iron depletion:

$$
\Delta W = \frac{nFE}{M} \times I_{corr} \times \Delta t
$$
\[
\Delta W = \frac{I_{\text{corr}} \Delta t A_m}{Z F}
\]

Where \( A \) is the atomic mass of iron (55.87 gr), \( Z \) is the valence of the reaction usually taken as 2 (assuming the corrosion product is Fe(OH)2) and \( F \) is Faraday’s constant equal to 96490 C (g/equivalent)[1]. The experimental values concerning steel mass consumption are listed in Table 5.

Six of the specimens were subjected to long term (one year) natural corrosion. In order to carry out the natural corrosion the specimens were placed in a basin of salt solution (NaCl 3%) and conditioning to a dry and wet situation once in a day. To evaluate the rebars corrosion activity, corrosion potential readings of the specimens were registered by using calomel electrode once a month. Table 3 shows the results of half cell potential test. The results of the table indicate that corrosion activity was started from the fifth month with 90% probability according to [1].

<table>
<thead>
<tr>
<th>Month</th>
<th>Potential (mV)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>260</td>
</tr>
<tr>
<td>2</td>
<td>320</td>
</tr>
<tr>
<td>3</td>
<td>360</td>
</tr>
<tr>
<td>4</td>
<td>440</td>
</tr>
<tr>
<td>5</td>
<td>530</td>
</tr>
<tr>
<td>6</td>
<td>530</td>
</tr>
<tr>
<td>7</td>
<td>540</td>
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<td>8</td>
<td>550</td>
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<tr>
<td>9</td>
<td>570</td>
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<tr>
<td>11</td>
<td>570</td>
</tr>
<tr>
<td>12</td>
<td>570</td>
</tr>
</tbody>
</table>

2.3. Repair Procedures

Two types of FRP sheets were used for the retrofitting of 12 specimens, with different fibre materials: Carbon versus Glass. The carbon fibre sheet (CFS) consisted of fibres arranged in a uniform direction and the glass fibre sheet (GFS) consisted of fibres arranged in two directions with proportions of 100% and 10%. Table 4 gives the mechanical properties of CFS and GFS in the main direction. Six of the specimens wrapped with CFS and the other six specimens wrapped with GFS. Initially the corner of specimens were rounded by a radius of 25 mm. Surface of the specimens were cleaned from rust and dust and then coated by resin primer. After 24 hours, columns were wrapped with 2 layers of FRP using resin glue.

<table>
<thead>
<tr>
<th>Type</th>
<th>Weight (g/m²)</th>
<th>Thickness (mm)</th>
<th>Tensile strength (MPa)</th>
<th>Elastic modulus (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CFS</td>
<td>300</td>
<td>0.176</td>
<td>4000</td>
<td>240</td>
</tr>
<tr>
<td>GFS</td>
<td>440</td>
<td>0.15</td>
<td>3450</td>
<td>77</td>
</tr>
</tbody>
</table>

3. EXPERIMENTAL RESULTS AND DISCUSSIONS

In the fourth phase of the experimental program all specimens were tested to failure under monotonically increasing concentric compression. Axial strain was measured as the average of two LVDTs placed on opposite sides of the specimen. In Table 5, columns 6–8 outline the most important indices of mechanical response measured during the load tests. In particular the ratio of \( P_{\text{max}}/P_{\text{cont}} \) quantifies the
increase and decrease in load carrying capacity ($P_{\text{max}}$) as compared to that of identical uncorroded-control specimens ($P_{\text{cont}}$). The specimens named as: $U =$ uncorroded, $A_{i} =$ accelerated corrosion with different levels, $N =$ natural corrosion, $W =$ without confinement, $C =$ CFRP confinement and $G =$ GFRP confinement. Figures 5 plot is a representative of histories of compressive stresses versus axial strain for the specimens.

Table 5: Experimental Result

<table>
<thead>
<tr>
<th>specimen ID.</th>
<th>number of specimen</th>
<th>level of corrosion</th>
<th>Mass loss $\Delta M_{\text{cage}}$ (%)</th>
<th>Layer and type of FRP</th>
<th>$P_{\text{max}}$ (kN)</th>
<th>$P_{\text{max}}/P_{\text{cont}}$</th>
<th>$\varepsilon_{\text{ax,peak}}$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>U-W 2</td>
<td>−</td>
<td>−</td>
<td>−</td>
<td>−</td>
<td>690</td>
<td>1</td>
<td>0.42</td>
</tr>
<tr>
<td>U-C 2</td>
<td>−</td>
<td>−</td>
<td>−</td>
<td>2-CFRP</td>
<td>1530</td>
<td>1.59</td>
<td>0.81</td>
</tr>
<tr>
<td>U-G 2</td>
<td>−</td>
<td>−</td>
<td>−</td>
<td>2-GFRP</td>
<td>1310</td>
<td>1.36</td>
<td>1.03</td>
</tr>
<tr>
<td>AC1-W 2</td>
<td>1</td>
<td>6.08</td>
<td>−</td>
<td>2-CFRP</td>
<td>840</td>
<td>0.91</td>
<td>0.34</td>
</tr>
<tr>
<td>AC2-W 2</td>
<td>2</td>
<td>9.35</td>
<td>−</td>
<td>2-CFRP</td>
<td>820</td>
<td>0.85</td>
<td>0.27</td>
</tr>
<tr>
<td>AC3-W 2</td>
<td>3</td>
<td>12.15</td>
<td>−</td>
<td>2-GFRP</td>
<td>790</td>
<td>0.82</td>
<td>0.08</td>
</tr>
<tr>
<td>AC3-C 2</td>
<td>3</td>
<td>12.15</td>
<td>2-CFRP</td>
<td>1470</td>
<td>1.53</td>
<td>0.66</td>
<td></td>
</tr>
<tr>
<td>AC3-G 2</td>
<td>3</td>
<td>12.15</td>
<td>2-GFRP</td>
<td>1205</td>
<td>1.25</td>
<td>0.90</td>
<td></td>
</tr>
<tr>
<td>N-W 2</td>
<td>Natural</td>
<td>−</td>
<td>−</td>
<td>−</td>
<td>925</td>
<td>0.96</td>
<td>0.39</td>
</tr>
<tr>
<td>N-C 2</td>
<td>Natural</td>
<td>−</td>
<td>2-CFRP</td>
<td>1505</td>
<td>1.56</td>
<td>0.78</td>
<td></td>
</tr>
<tr>
<td>N-G 2</td>
<td>Natural</td>
<td>−</td>
<td>2-GFRP</td>
<td>1300</td>
<td>1.35</td>
<td>0.98</td>
<td></td>
</tr>
</tbody>
</table>

Figure 5. Stress-strain curves of compressive tests

Generally, the FRP jackets improved the performance of columns under compression in terms of load carrying capacity, deformation capacity up to peak load and ductility. Their passive confining action was mobilized progressively in response to lateral dilation of the concrete core. In most cases failure started in the corroded portion and extended over the entire specimen with abrupt rupture of the jacket and disintegration of the concrete cover and core. This premature mode of failure was accompanied by simultaneous buckling of longitudinal reinforcement owing to the failure of corroded stirrups.

According to the experimental results in table 5, it is concluded that corrosion level 3 of the specimens reduces 18% strength and 80% ductility with respect to the
reference specimens (U-W). The sever loss of ductility is due to the effect stirrups being more corroded than the longitudinal reinforcements, due to the fact that cover on the stirrups is less than the longitudinal bars.

Strength and ductility of specimens that are corroded naturally are between the results of U-W and AC1-W specimens. Strengthening of the specimens corroded intensively using CFRP and GFRP wrap increase the strength and ductility more than the U-W specimens. This strength and ductility values could reach also close to U-C and U-G values. Regarding all the cases CFRP gives better performance towards strength and GFRP gives better performance towards ductility. According to the last reports [3, 5] and results of this paper it is concluded that because of geometry of column’s section and stresses concentration on the corner of rectangular section, FRP wrap has the lower performance in comparing with columns with circular section.

4. CONCLUSIONS

The following conclusions are deduced from the experimental results:

- Intensive corrosion of the specimens reduces 18% strength and 80% ductility with respect to the reference specimens (uncorroded).
- The sever loss of ductility is due to the effect stirrups being more corroded than the longitudinal reinforcements.
- Generally, the FRP jackets improved the performance of columns under compression in terms of load carrying capacity, deformation capacity up to peak load and ductility. Their passive confining action was mobilized progressively in response to lateral dilation of the concrete core.
- Regarding all the cases CFRP gives better performance towards strength and GFRP gives better performance towards ductility.

REFERENCES


DESIGN AND DEVELOPMENT OF RESIN CAPSULE ANCHORING SYSTEMS FOR STRENGTHENING THE CONCRETE SURFACE USING DESIGN-EXPERT SOFTWARE

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ABSTRACT
The demand for strengthening of concrete structures has resulted in an increased use of chemical bonded anchors. Therefore, investigating the formulation and mechanical properties of adhesive grout layer has become an important issue. Different formulations of polyester resin grout were designed by Design-Expert software and their uniaxial compressive strength, volume shrinkage, gel time and maximum exotherm temperature were obtained. In addition, their rheological behaviors once the reinforcing bar is inserted were evaluated. Having optimized the above parameters by means of mixture D-optimal method, the most efficient formulation of chemical bonded anchors was developed. The effect of adding thixotropic additives and different rotational speed of anchors on the rheological behavior of optimum mixture were investigated and the resin capsule was produced. The most significant factors increasing compressive strength and volume shrinkage are resin and monomer content. Inhibitor concentration decreases both responses dramatically. Silica fume not only improves the rheological behavior of the grout, but also increases its storage time.

Keywords: chemical bonded anchors, polyester resin grout, mixture d-optimal, thixotropic behavior

1. INTRODUCTION
Grouting is a widely used method for strengthening and sealing rock, soil and concrete. Grouts application in the construction and repair of structures include returning disintegrated concrete and masonry into a monolithic mass, repair and welding of cracks in structural concrete members, securing of bolts, rods and anchors in drilled hole, casting of preplaced aggregate concrete and corrosion protection for anchors and tendons [1]. The demand for more flexibility in planning, design and strengthening concrete structures has resulted in an increased consumption of chemical grouted anchors. In addition, due to reducing the average-time of construction, chemical bonded anchors or grouted anchors provide a viable and economical method for adding new concrete sections or steel members to existing concrete structures.

The chemical bonded anchor consists of a structural adhesive grout such as
unsaturated polyester resin and a threaded rod or a reinforcing bar which is inserted in a drilled hole. They develop their holding capacities by bonding of the adhesive grout to both the anchor and concrete [2]. Resin adhesive grouts are available as prepackaged glass or plastic film capsules or dual cartridge injection systems. Plastic films capsules are better suited for use on construction sites since they are more robust. Because of their flexibility they adapt themselves to the hole geometry and can easily be installed overhead [3]. The capsules contain two separated compartments. The outer compartment contains the resin mixture while the inner compartment contains the catalyst mixture. The capsule is inserted into the hole, the threaded rod is then rotary-hammered into the capsule, rupturing the plastic film and mixing the two compartments. Chemical reaction between resin and catalyst mixture hardens the resin adhesive and creates a high strength bond. Factors influencing the performance of load transfer in and strength of chemical bonded anchor systems are reinforcing bar properties, adhesive grout characteristics and installation conditions. Resinous grout properties can be determined by its compositions. The cross-linking reaction between unsaturated polyester resins and vinyl monomers allows one polymer chain to connect with other polymer chains and produce a three dimensional network, which converts the resin from a viscous liquid into a hard, thermoset solid. Unsaturated polyester resins (UP) are cured in the presence of free radicals that are derived from a catalyst such as organic peroxides.

The resin adhesive should have good mechanical properties after setting. Viscosity alteration is also needed especially during installation in order to ensure proper mixing; moreover, maximize contact with concrete and rod surfaces. Residual stresses and possible formation of cracks and voids due to volume shrinkage can present serious problems for chemical bonded anchors. In vertical concrete surfaces, Thixotropic behavior of the resin can reduce run out and sagging before gelation takes place.

The main objective of the present work is to design different adhesive grout formulations and analyze their influence on uniaxial compressive strength, volume shrinkage and rheological behavior of the resinous grout with the latest version of Design-Expert software. Various compounds were designed by mixture D-optimal method. Finally the effect of thixotropic agent on rheological properties of the optimized compound will be evaluated.

2. EXPERIMENTAL PROCEDURE

2.1. Materials

Design variables include unsaturated polyester resin (A), mineral filler (B), monomer (C) and initiator-inhibitor mixture (D). Amount of plasticizer and promoter are kept constant in order to increase the efficiency of the predicted models which are suggested by software. The ratio of the initiator to inhibitor for the mixtures with more than 2% peroxide is 0.25. The amount of orthophthalic polyester resin -provided by Resitan Co., containing 30 wt. % styrene- in the formulation was kept between 20 to 30 wt. %. Limestone powder (200 mesh) was chosen as filler having a mean particle size of about 30 – 50 microns from Iran.
Micronized Powder Co. (55-68 wt. %). Styrene monomer was used as crosslink agent in the curing reaction of unsaturated polyester resin (4-9 wt. %). The initiator used in this study was 55 wt. % solution of benzoyl peroxide (BPO) in phthalate solvent (2-5 wt. %). N, N dimethyl aniline (DMA) was employed as promoter because of its high reactivity to decompose the initiator at low temperatures. Inhibitor was 3 wt. % solution of hydroquinone in dipropylene glycol. Constant amount of 4% Dibutyl phthalate was added as plasticizer in order to adjust the viscosity of the resinous grout. Silica fume from I.F.I Co. was used as thixotropic agent in three different percentages (0.2%, 0.7% and 1.5%) with mean particle size of 0.05-0.015 microns.

2.2. Methods

Depending on the number of design variables and applied constraints, 22 combinations were designed by the software; their results are presented in Table 1. Among these combinations, 5 are allocated to replicates; another 5 are for testing the lack of fitness. Test method for gel time and maximum exotherm temperature during curing of each designed mixture was performed using ASTM D2471-99 [4]. The compressive strength test was performed according to ASTM-C579-01 [5]. Three cylindrical test specimens were cast for each composition; each specimen was Design and Development of Resin Capsule Anchoring Systems for Strengthening the Concrete Surface Using Design-Expert Software.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Gel Time (s)</th>
<th>Compressive Strength (MPa)</th>
<th>Volume Shrinkage (%)</th>
<th>Max Temp. (°C)</th>
<th>Density g/cm²</th>
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<td>78.1337</td>
<td>9.42276</td>
<td>79.2</td>
<td>1.84384</td>
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<td>69.56</td>
<td>60.6298</td>
<td>4.79809</td>
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<td>1.9272</td>
</tr>
<tr>
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<td>304.50</td>
<td>56.8142</td>
<td>6.43913</td>
<td>74</td>
<td>1.7577</td>
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<tr>
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<td>91.80</td>
<td>76.2515</td>
<td>7.84562</td>
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<td>1.85682</td>
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<td>81.4337</td>
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<td>67</td>
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<td>193.37</td>
<td>72.2084</td>
<td>9.102</td>
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<td>73.6807</td>
<td>9.14743</td>
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<td>9</td>
<td>231.52</td>
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<td>7.75762</td>
<td>73.9</td>
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<td>4.0159</td>
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<td>47.6118</td>
<td>6.32899</td>
<td>71</td>
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<td>40.8754</td>
<td>4.03017</td>
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<td>8.40159</td>
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<td>33.3691</td>
<td>2.55093</td>
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<td>57.9861</td>
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<tr>
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<td>81.4481</td>
<td>8.40954</td>
<td>78.8</td>
<td>1.81908</td>
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3. RESULTS AND DISCUSSIONS

3.1. Compressive Strength

In accordance with the obtained results of compressive strength for each grout mixture (Table 1), analysis of variance (ANOVA) is conducted by the software and the quadratic model is recommended for estimating the outcomes. As is demonstrated in Table 2, each design variable solely has a significant role in determining the compressive strength of grout. Moreover, the CD factor which represents the interaction effect between monomer and initiator-inhibitor mixture is known as a fairly influential factor in verifying compressive strength of resinous grout.

According to Figure 1 and Figure 2, increase in percentages of resin and initiator-inhibitor mixture has the most significant effect on compressive strength of the grout. By increasing the concentration of initiator-inhibitor mixture, compressive strength is declined dramatically; this is originated from the increase of inhibitor content in the mixture, despite the increment of initiator concentration. Similar results have been observed in the Cook and Lau’s research concerning the curing process of polyester resin in presence of different percentages of initiator and inhibitor [6].

Adding the promoter leads to decomposition of initiator into free radicals. Inhibitor consumes the generated free radicals; as a result the efficiency of initiator will be diminished.

The increase of resin percentage is similar to increase of matrix component of the grout. Therefore, filler particles are dispersed better in the resin matrix. The reduction of viscosity due to higher resin content and lower filler content assist in better wetting and screening of the particles. Consequently, by curing the matrix component a more densified network is formed and the compressive strength is increased. Figure 2 illustrates a decrease in compressive strength in high percentages of filler. Increasing the monomer concentration has led to a higher compressive strength. This is due to increase of probability of reaction between styrene and free radicals in polyester resin chains.

### Table 2. Analysis of variance (ANOVA) for compressive strength results

<table>
<thead>
<tr>
<th>Source</th>
<th>SS</th>
<th>DF</th>
<th>MS</th>
<th>F-value</th>
<th>P-value</th>
<th>Significance</th>
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<tbody>
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<td>Mean</td>
<td>5213.35</td>
<td>9</td>
<td>579.26</td>
<td>115.56</td>
<td>&lt;0.0001</td>
<td>S.</td>
</tr>
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<td>Linear</td>
<td>5035.37</td>
<td>3</td>
<td>1678.46</td>
<td>334.86</td>
<td>&lt;0.0001</td>
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</tr>
<tr>
<td>AB</td>
<td>5.59</td>
<td>1</td>
<td>5.59</td>
<td>1.12</td>
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<td></td>
</tr>
<tr>
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<td>3.53</td>
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<td>3.53</td>
<td>0.7</td>
<td>0.418</td>
<td></td>
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<tr>
<td>AD</td>
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<td>11.65</td>
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<td>0.1532</td>
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<tr>
<td>BC</td>
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<td>1</td>
<td>0.6</td>
<td>0.12</td>
<td>0.736</td>
<td></td>
</tr>
<tr>
<td>BD</td>
<td>9.4</td>
<td>1</td>
<td>9.4</td>
<td>1.87</td>
<td>0.1961</td>
<td></td>
</tr>
<tr>
<td>CD</td>
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<td>1</td>
<td>25.84</td>
<td>5.15</td>
<td>0.0424</td>
<td></td>
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<tr>
<td>Residual</td>
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<td>12</td>
<td>5.01</td>
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<tr>
<td>LOF</td>
<td>44.49</td>
<td>7</td>
<td>6.36</td>
<td>2.03</td>
<td>0.2267</td>
<td>Not S.</td>
</tr>
<tr>
<td>Error</td>
<td>15.66</td>
<td>5</td>
<td>3.13</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>5273.5</td>
<td>21</td>
<td></td>
<td></td>
<td></td>
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</table>
When the curing reaction begins, viscosity increases and after gelation a three dimensional network is produced in resinous grout bulk. Therefore, polyester resin chain's motions are limited, resulting in increased rate reaction between small molecules of styrene monomer and free residual radicals. The result is autoacceleration of the reaction at higher conversions known as Norish-Trommsdof effect. Van Assache concluded that the autoacceleration is mainly an acceleration of the styrene consumption. Also the autoacceleration is because of depletion of the styrene monomer that causes a further increase in the viscosity [7]. Hence, as a result of autoacceleration, higher degrees of conversion of the grout are achieved, thus raising the compressive strength of the samples.

### 3.2. Volume Shrinkage

Considering the empirical and predicted results, the Design-Expert software suggested a linear model for estimating the volume shrinkage of the grouts (Table 3). According to trace plot in Figure 3, the slope of design variables shows that the resin and monomer concentration have the most significant influence on increase of the volume shrinkage of the resinous grout.

<table>
<thead>
<tr>
<th>Source</th>
<th>SS</th>
<th>DF</th>
<th>MS</th>
<th>F value</th>
<th>P value</th>
<th>Significance</th>
</tr>
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<tr>
<td>Model</td>
<td>85.96</td>
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<td>28.65</td>
<td>37.24</td>
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</tr>
<tr>
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<td>85.96</td>
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<td>28.65</td>
<td>37.24</td>
<td>&lt;0.0001</td>
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<td>13.85</td>
<td>18</td>
<td>0.77</td>
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<td>LOF</td>
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<td>0.61</td>
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<td>0.843</td>
<td>Not S.</td>
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<td>Error</td>
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<td>1.18</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>99.81</td>
<td>21</td>
<td></td>
<td></td>
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</table>
The opposite effect is observed while increasing the concentration of filler and initiator-inhibitor mixture (D). Higher amount of inhibitor in the mixture consumes more generated free radicals which lessens the degree of conversion. Higher conversion would result in a higher amount of shrinkage. Therefore, the polyester resin network shrinks less compared to the state which no inhibitor exists. Monomer’s effect on the shrinkage of the grout is more considerable than that of the resin. This is because of the major role of the styrene monomer in increasing the conversion during curing of the polyester resin. As was mentioned before, small molecules of styrene can diffuse easily among the long crosslinked polyester chains and react with free radicals, making the network denser. Increasing the filler content reduces volume shrinkage linearly. This is because of the decrease in the resin content in the mixture (Figure 4). Filler component acts similar to thermal insulation in the resin matrix and prevents heat transfer through the matrix bulk. By absorbing the exotherm heat, filler particles can hinder the volume shrinkage of the grout and prevent the development of internal stresses. Rate of relaxation of the internal stresses in polymers can be increased by adding substances such as fillers [8].

3.3. Comparing the Results
In Figure 5, higher compressive strengths are seen in the samples which have higher maximum exotherm temperature; this represents higher conversion in these samples. Therefore the resinous matrix of the grout is stiffer and can resist more loads before failure. Figure 6 confirms that higher volume shrinkage would occur in the samples which have higher exotherm temperatures. By examining different formulations, it is clarified that filler content cannot increase the compressive strength, but it even reduces the compressive strength by decreasing the concentration of the resin and monomer.
According to previous results, it is essential to estimate the optimized formulation in order to gain the highest compressive strength and the lowest volume shrinkage. The software introduces the compound formulations as ideal grouts with optimized responses as mentioned in Table 4. In order to compare the outcomes with predicted results achieved by the software, it is necessary to implement the conducted experiments on the optimized compound #1 as well.

One of the significant characteristics of chemical bonded anchors is their high compressive strength in the beginning of the installation of the anchors. Hence, the compressive strength of the optimized compound was examined in the first one hour and 24 hours right after grout curing in order to reassure its strength from the beginning of the installation.

According to Table 5, about 79% of the final strength was achieved after 24 hours of installation the anchor in concrete hole. The outcomes of the designed models for evaluating each response show a good correlation with empirical results. This means that the suggested models can perfectly predict the different formulation properties. Therefore, by using these models, ideal formulations can be suggested and different kinds of grouts can be designed based on consumer demands. This will considerably decrease the price of the final product.

### 3.4. Thixotropic Agent

It is essential to evaluate rheological behavior of the optimized compound in the presence of three different percentages of fumed silica (0.2%, 0.7% and 1.5%) as a thixotropic agent. Therefore, the rheological measurements were designed in such a way that stimulates similar circumstance which grout has in a concrete hole. That is, first the grout sample was subjected to shear rate of 0.01 s\(^{-1}\) in 100 seconds (stage1), then it was promptly exposed to rotation speed of 100 rpm for another 100 seconds (stage2). Then, the sample was remained in holding time period in which the rotation was stopped and the shear rate returns to 0.01 s\(^{-1}\) (stage3). This
test determines the thixotropic and flow behavior of the optimized grout compound in each of the three defined intervals.

Table 4: Optimized formulations and their predicted results are suggested by the software

<table>
<thead>
<tr>
<th>V.Shrinkage (%)</th>
<th>Compressive (MPa)</th>
<th>Max.Temp. (°C)</th>
<th>Density (g/cm³)</th>
<th>Gel Time (s)</th>
<th>Desirability</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>7.60287</td>
<td>71.2102</td>
<td>74.6882</td>
<td>1.87189</td>
<td>63.1655</td>
</tr>
<tr>
<td>2</td>
<td>7.7851</td>
<td>69.0389</td>
<td>68.7493</td>
<td>1.90448</td>
<td>107.2</td>
</tr>
<tr>
<td>3</td>
<td>8.51495</td>
<td>75.8757</td>
<td>80.5386</td>
<td>1.83724</td>
<td>108.495</td>
</tr>
<tr>
<td>4</td>
<td>8.65867</td>
<td>74.6901</td>
<td>75.6227</td>
<td>1.8573</td>
<td>95.09888</td>
</tr>
</tbody>
</table>

Table 5: The actual properties of optimized compound

<table>
<thead>
<tr>
<th>Gel Time</th>
<th>V.Shrinkage</th>
<th>Compressive Strength (MPa)</th>
<th>Max.Temp (°C)</th>
<th>Density (g/cm³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Optimized compound</td>
<td>79.56</td>
<td>7.40816</td>
<td>49.3914</td>
<td>61.04594</td>
</tr>
</tbody>
</table>

Figure 7 shows how the optimized compound behaves in the presence of three different percentages of silica fume. In spite of the high amount of filler in the compound during shearing interval (stage 2), viscosity decreases dramatically. Furthermore, it is observed that increase in the silica fume content enhances the thixotropic properties of the samples and after the rotation of anchor is terminated, the viscosity returns to plateau state rapidly. Silica fume particles are reactive pozzolanic materials and in consequence of their method of producing, they have silanol groups on their surfaces. Researchers have shown the interaction and forming of hydrogen bridging between carbonyl groups and hydroxyl end groups of the polyester resin with silanol groups [9]. Structures based on hydrogen bond through silanol groups are temporary and they will break if exposed to high shear mixing. In order to examine the grout structure, frequency sweep test is conducted on grout samples (Figure 8). It is observed that by adding silica fume to the optimized compound, storage modulus is increased. According to the trend of storage modulus and loss modulus in Figure 8, all the samples except the one with 1.5% silica fume, show viscous behavior at low frequencies. However, in the sample with 1.5% silica fume, by increasing frequency, the storage and loss modulus remain approximately unchanged. Hence, the mentioned sample would resist against deformation. When the storage modulus as a function of frequency turns into horizontal line, a three dimensional structure is formed; this is occurred in the sample with 1.5% silica fume. In fact by increasing the amount of silica fume in grout, interactions between filler’s metal oxides and silanol groups with acid groups in polyester resin chains would increase and this would result in a network structure as shown in frequency sweep test. The elastic behavior of sample with 1.5% silica fume would provide longer storage time. This structure is temporary and according to Figure 7 during shearing interval resinous grout represents the good thixotropic behavior.
Figure 7. The thixotropic behavior of the optimized grout compound (■) in presence of three different percentages of silica fume. 0.2% (■), 0.7% (■), 1.5% (■).

Figure 8. Loss and storage modulus of optimized compound as a function of frequency in presence of different percentages of silica fume (0.2%, 0.7% and 1.5%) in frequency sweep test.

The rheological test, designed for observing the thixotropic properties in three different rotation speeds (100 rpm, 300 rpm and 600 rpm), has been evaluated. In Figure 9, it is illustrated that in all three presumed rotation speeds, grout samples show good thixotropic behavior and would return to the plateau state in a short while. Viscosity drop in higher shear rates is more considerable and it assists to better mixing of components. Because all three samples return to plateau state approximately at the same time, the higher rotation speed of the anchor is preferred due to the more viscosity drop it causes in second interval.

The plastic film used for packaging the resinous grout into the capsule shape is called Myler film.
The Myler film is polyethylene terephthalate transparent barrier film with 30-40 microns thickness. The catalyst formulation compartment would be located inside the resin formulation compartment. (Figure 10)

4. CONCLUSION
Polyester resin, as the matrix component of the adhesive, has the most significant effect on increasing the compressive strength of the grout. Increasing the initiator-inhibitor mixture decreased the compressive strength dramatically because the inhibitor consumed the free radicals. The low filler content had no effect on compressive strength of the grout but higher amounts of the filler reduced the compressive strength. It was observed that monomer content has the most significant influence on increasing the volume shrinkage. Increasing the initiator-inhibitor mixture and filler content reduced the volume shrinkage linearly while higher resin concentration led to increased shrinkage.

Good correlation between the maximum exotherm temperatures and other responses were obtained. Moreover, the results of the experiments for the optimized compound are in good agreement with the estimated values. According to the designed rheological test, improved thixotropic behavior was achieved by adding fume silica to polyester resin grout. The higher rotation speed of anchor caused better mixing and thixotropic behavior.

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PERFORMANCE INVESTIGATION OF RC SHORT COLUMNS RETROFITTED WITH FRP COMPOSITES IN PASSIVE & ACTIVE STATES

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ABSTRACT
Short columns and columns having weakness in transverse reinforcement have shown inappropriate behaviour in confronting with shear forces during different earthquakes and the shear failure mode of these columns is the main failure mode. One of the most conventional methods for improving the behaviour of these columns is the application of wrapped FRP jacket. These FRP jackets are lightweight, high strength and have a convenient installation process. These jackets can be utilized both in active and passive states. In this study, weak and retrofitted columns have been modelled using “Seismostruct” software and have been compared with experimental conclusions as well. In passive retrofitted specimens, the improvement in value of energy dissipation observed was about 1.7 to 4 times greater than control specimens. Furthermore, drift angle was increased about 1.4 to 2.5 times greater. Moreover, active retrofitted specimens have demonstrated a better behaviour than passive ones. In most of the specimens the growth was about 20% and 30 to 40 percent in shear strength and quantity of energy dissipation, respectively. Finally, AFRP composites have demonstrated higher ductility than CFRP composites, despite the fact that the shear strength of CFRP was higher than AFRP type.

Keywords: AFRP, CFRP, confinement, fibre analysis, drift angle, wrapped jackets

1. INTRODUCTION
Shear failure has been one of the most common failure modes for RC structures subjected to earthquakes. Recent studies and experimental researches have shown that short RC columns of buildings are vulnerable in brittle failure. Due to this weakness, a great number of studies have been done to find appropriate methods for increasing the shear strength of such columns in order to change the brittle failure mode to a ductile one. A majority of columns are primarily designed as flexural members but unexpectedly change into short columns by adding walls and infill to the structure. In addition, most of the columns, which have been designed according to codes before 1970 (strength based design) lack enough transverse bars, so their dominant failure mode is the shear mode. This failure mode happens
suddenly and the brittle shear failure decreases the capacity of energy dissipation of the column. Shear capacity of short RC columns is a function of parameters like: 1- Area of longitudinal and transverse bars, 2- Compressive strength and confinement of concrete and 3- Coherence between steel and concrete [1 & 2].

2. HISTORY
A large number of numerical and experimental researches have been done for strengthening and retrofittig RC columns using passive confinement so far, such as the works of Galal et al. [1&3], Saatcioglu [4] and Ehsani [5]. Nevertheless, a few studies have been done using active confinement. The researches of Yamakava et al. [6] and Saadatmanesh [7] are the sample of these studies. In this study, both active and passive confinements were investigated. The active confinement has been considered with the use of prestressed Carbon & Aramid fibres.

3. METHODOLOGY
In the present study, 5 short cantilever RC columns have been chosen and retrofitted in both active and passive states. The ratio of their shear span to depth is 2.5. For considering confinement effect in the concerned software, the relationships of Mander et al. [8] have been used for evident specimens. For passive retrofitted specimens, the relationships introduced by Galal et al. [1&9] and for active retrofitted specimens, those introduced by Yamakava et al. [6] have been used for obtaining confinement factor. The mentioned specimens have been subjected to a constant perpendicular and cyclic lateral load with the help of “Seismostruct” software [10]. The results of analyses have been evaluated and then compared with those of experimental studies [11] held in advance. There is a satisfying compatibility between the hysteresis curves obtained from either analytical specimens or experimental studies.

4. GENERAL PROPERTIES OF THE SPECIMENS
All the specimens have been considered rectangular, having 250x250 mm² dimensions and a height of 620 mm. The compressive strength of concrete is 18MPa. As shown in Figure (1), the longitudinal bars used for reinforcement is 12φ12 and φ4 bars are used for transverse reinforcement. The spacing of transverse reinforcement was considered to be 50 mm at the beginning of the column and at the column-foundation connection, and 100 mm for the rest. For the passive retrofitted specimens, Aramid & Carbon FRP have been used and wrapped up to the height of 30 mm of the columns. The mechanical properties of these composites are shown in Table (1). For the active retrofitted specimens, prestressed FRP fibres having 40 mm width and spacing of 70 mm have been utilized.

<table>
<thead>
<tr>
<th>Fiber Types</th>
<th>Tensile Strength (MPa)</th>
<th>Ultimate Strain (%)</th>
<th>Modulus of Elasticity (GPa)</th>
<th>Thickness (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Carbon</td>
<td>3800</td>
<td>1.55</td>
<td>240</td>
<td>0.165</td>
</tr>
<tr>
<td>Aramid</td>
<td>2900</td>
<td>2.50</td>
<td>120</td>
<td>0.440</td>
</tr>
</tbody>
</table>

Table 1: Mechanical properties of used FRP
For convenience of studying, the specimens have been nominated as regarded in Table (2).

Table 2: Names adopted for the specimens

<table>
<thead>
<tr>
<th>Description</th>
<th>Parameter</th>
<th>Description</th>
<th>Parameter</th>
</tr>
</thead>
<tbody>
<tr>
<td>One layer CFRP</td>
<td>C1</td>
<td>Evident Column</td>
<td>C</td>
</tr>
<tr>
<td>One layer AFRP</td>
<td>A1</td>
<td>Passive Retrofit</td>
<td>PR</td>
</tr>
<tr>
<td>Prestressed CFRP fibre</td>
<td>SC</td>
<td>Active Retrofit</td>
<td>AR</td>
</tr>
<tr>
<td>Prestressed AFRP fibre</td>
<td>SA</td>
<td>Level 1 axial force</td>
<td>L1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(N=0.2 × A_p × f'c)</td>
<td></td>
</tr>
<tr>
<td>Prestressed strain (ratio of ultimate strain of FRP)</td>
<td>1/n</td>
<td>Level 2 axial force</td>
<td>L2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(N=0.4 × A_p × f'c)</td>
<td></td>
</tr>
</tbody>
</table>

5. INTRODUCING SEISMOSTRUCT SOFTWARE

SeismoStruct is a Finite Element program capable of predicting large displacement behaviour of space frames under static or dynamic loading, taking into account both geometric nonlinearities and material inelasticity. The spread of inelasticity along the member length and across the section depth is explicitly modelled, allowing for accurate estimation of damage distribution. The sectional stress-strain state of beam-column elements is obtained through the integration of a nonlinear uniaxial stress-strain response of the individual fibres in which the section has been subdivided. The subdivision of a typical reinforced concrete cross-section is depicted in the Figure (2). By employing a sufficient number of fibres (200-400 in spatial analysis), the distribution of material nonlinearity across the section area could be accurately modelled, even in the highly inelastic range. In this study, the number of fibres has been chosen to be 250 [10].
5.1. Loading
The aforementioned models have been subjected to two simultaneous loads; 1- Constant perpendicular axial load and 2- Time history lateral load. The analysis operation is based on time history static procedure. The constant perpendicular load is considered as a ratio of compressive strength of concrete column \(0.2 \times A_g \times f'c\) which is equal to 225 KN. The time history lateral load is employed as “applied displacement”, where the corresponding values are the displacement of column and time. These values are obtained by dividing lateral displacement of column \(\Delta\) over its height (h) which is defined as “drift angle”. At each drift angle (R), 3 successive cycles with the values of 0.5, 1.0, 1.5, 2.0 and so on have been applied as shown in Figure (3).

5.2. Analysis Results
5.2.1. Hysteresis Curves of Shear-Displacement Index
The obtained hysteresis curves of retrofitted columns wrapped with Carbon (PR-C1-L1) and Aramid (PR-A1-L1) in the passive state and the evident specimen (C-L1) have been shown in Figures (4 and 5). By the comparison of the given curves, the high efficiency of application of wrapped FRP can be observed on increasing the ductility and shear strength of retrofitted ones. Furthermore, the specimens retrofitted with Aramid have shown more adequate ductility than the Carbon though there is a negligible difference for shear strengths.
The obtained hysteresis curves of retrofitted columns wrapped with prestressed Aramid fibres (AR-CA-SA-1/2-L1) and unwrapped prestressed Carbon fibres (AR-SC-1/6-L1) where the prestressing values are considered as $\frac{1}{2} \varepsilon_{fp}$ and $\frac{1}{6} \varepsilon_{fp}$ respectively have been shown in the Figures (6 and 7). Regarding to the hysteresis curves, it can be found that prestressing of AFRP fibres results in more efficient to increase ductility and shear strength than the passive retrofitted one, especially for shear strength enhancement.
Regarding Figure (8), it is obvious that the active retrofitted specimen (AR-C1-SA-1/2-L1) in comparison to passive retrofitted one experiences more ductility and shear strength.

5.3. Energy Dissipation Diagrams
The amount of energy dissipation of the studied specimens, have been shown in Figure (9). The energy dissipation has a direct relation to the ductility of a structure. Regarding the mentioned figure, the amount of energy dissipation for active retrofitted specimen is more than the passive one.
5.4. Shear Force Envelop Curve
The decrement or increment of the column shear force versus drift angle has been illustrated in Figure (10). The active retrofitted specimens have shown more suitable performance than the passive ones.

5.5. Comparison of Analytical Hysteresis with the Experimental Studies
On account of probable errors in analysis and also the differences between material properties in theory and experiment, a comparison has been done between the two analytical models and two concerned experimental ones. As shown in Figures (11 and 12), there is an acceptable conformity among analytical and experimental models.
6. CONCLUSIONS
The results derived from the abovementioned analyses are stated as below:

1. The amount of ductility, energy dissipation and shear strength of the columns would increase by retrofitting them using passive wrapped FRP.
2. AFRP composites have shown more ductility than CFRP composites as a consequence of higher strain capacity, though the shear strength of CFRP was higher than AFRP type.
3. By prestressing the FRP fibres especially AFRP, the amount of confinement and accordingly the shear and compressive strength of concrete would increase.
4. The ratio of increment of energy dissipation for retrofitted specimens varies from 1.7 to 4 times than evident specimens. Also the drift angle increases from 1.4 to 2.5 times.
5. Active retrofitted specimens have shown better behaviour than passive ones, around 20% increment in shear strength and 30-40% in energy dissipation.

REFERENCES
NUMERICAL MODELING OF UPLIFT RESISTANCE OF BURIED CONCRETE DUCTS & PIPES

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2Graduated Student, School of Civil Eng., Semnan University, Semnan, Iran

ABSTRACT
This paper presents simulation of the pipe and soil system behavior during uplift displacement of pipelines in dense and loose sand by a 2D Finite Element modelling. As the first part of an ongoing research, this study focuses on the type of the soil failure mechanism which occurs in saturated condition at different burial depths and soil densities for constant pipe diameter. Then, at this stage, pipe is considered as a linear elastic but very stiff material (compare to the soil). Using conventional continuum elements for the soil, its material behavior is modelled by the Drucker-Prager criterion. The numerical results compared to the laboratory observations and results as well as theoretical aspects.

Different laboratory failure mechanisms in loose and dense soils and also load-displacement curves impressively reproduced by the numerical modellings. Even, development of a gap below the pipeline in dense sand well captured by the FEM results. The essential effect of the soil density on the resistance force against uplift displacement is also well illustrated by the numerical results. To obtain a normalized upheaval load-displacement curve for this phenomenon, displacements were normalized by the pipe diameter and also burial depth, however, another more suitable length parameter is under investigation.

Keywords: buried pipeline, uplift, sand, numerical modeling, FEM

1. INTRODUCTION
Lifeline pipes and ducts are commonly buried to provide environmental stability, thermal isolation and mechanical protection. In shallow trenches of saturated sand soils, buried pipelines and ducts are under threaten of progressive upheaval creep failure, during long term operation. This phenomenon called as upheaval buckling. This type of failure can cause the pipe to resurface at its bed-line and causes probable fracturing and damages, specially on the concrete ducts. This presents considerable operational problems and would have significant costs. The backfill soil in the trench and the pipe weight contribute to prevent the upheaval buckling load imposed to the pipeline. However, the resistance to the upheaval load provided by the soil is difficult to calculate.
A number of theoretical models has been developed to predict the resistance to upward movement provided by the pipeline/soil system. Generally, the models
have considered the backfill soil without considering imperfections of the material during pipe placement. Also, no roughness effect of the pipe-soil contact surface was considered. It is widely accepted that the uplift resistance is complex and it is related directly to the geotechnical properties of the soil. Most theoretical analyses assume some failure surfaces extending through the soil above the pipe. The simplest of these, reported by Matyas and Davies (1983) [1], is to assume a vertical slip surface extending above the pipe, Figure 1(a). The uplift resistance per unit length derived from the weight of soil above the pipe, the weight of soil displaced by the upper half of the pipe and the shearing resistance along the vertical slip surface, is given by the expression (1):

$$F_v = (1 - \frac{\pi D}{8H} + K \tan \phi \frac{H}{D}) \gamma HD$$

(1)

in which $\gamma$ = effective soil unit weight, $H$ = depth to centre of pipe, $D$ = pipe diameter, $\phi_{ps}$ = angle of soil friction in plain strain, and $K$ = lateral earth pressure coefficient. The value of $K$ is often taken as $K_0$, the at rest coefficient for loose sand, but its value in case of the dense sand is difficult to assess and can often be greater the one. Trautmann et al (1986) found [3], this theory has good agreement with the experimental data with $K$ values of 0.5, 0.65 and 0.75 for pipes in loose, medium and dense sand, respectively. However, rupture surface above pipes are generally curved in broad agreement with the pyramidal shaped geometry analyzed by Meyerhof and Adams (1968) [2], shown in Figure 1(b). For shallow embedment, they ignored the second term in Eq. 1, and assumed $K=0.95$, while at greater depths the Eq. 2 was proposed:

$$F_v = \left(1 + \left(\frac{2H}{D} \frac{H_s}{D_s}\right) \frac{H}{H_s} K \tan \phi \frac{H}{D}\right) \gamma HD$$

(2)
where \( H_e \) is the vertical extent of the rupture surface and depends on \( \varphi_{ps} \) and \( D \).

The uplift force, \( N_u \) can be non-dimensionalised and re-expressed as uplift factor, \( f_u \), as suggested by Schaminee et al (1990) [4]. This is calculated using:

\[
\frac{f_u}{D} = \frac{N_u - 1}{H},
\]

in which:

\[
N_u = \frac{F_u}{\gamma H D L},
\]

(3)

where \( H \) is the instantaneous embedment depth measured to the crown of the pipe.

2. PREVIOUS MODELLINGS’ RESULTS

Over the last decade, the geotechnical aspects associated with upheaval resistance of the buried pipes have received considerable attention by the researchers. The focus of this attention has been aimed at the mechanism of soil failure and measurement of uplift load for various soil types. Because of practical difficulties and the high cost of conducting full scale field tests, the majority of these works has been done at small scale, or using geotechnical centrifuge modelling to simulate full scale conditions [5]. Barnsby et al (2001) [6] undertook a combined study using numerical FE analysis and scaled physical model testing to investigate soil resistance to upwards pipeline movement. Rezaee et al (2005) [7] carried out full scaled laboratory tests by improving the conditions of the experiments done by the Trautmann et al (1986) [3], which the FEM modellings of these experiments are presented in here.

3. EXPERIMENTAL OBSERVATION

The laboratory model built by Rezaee et al (2005) [7][8], was including four parts: test box, coarse sub-grade, transducers & gauges, and water intake system to apply loading.

Rigid boundaries have to be located remote from the pipe so as not to interfere with failure mechanisms or effect on the effective stresses in deforming zone. According to the Trautmann suggestion [3], the width and height of the model should be chosen at least five times and its length nine times of the pipe diameter. Then, the test box dimensions- in toughened glass material of 1 cm thickness- considered 69cm of width and height, and 177cm of length, comparing to 11cm of the P.V.C pipe diameter. The test box was well braced not to deform during the loading. As it was necessary to increase the water surface uniformly, a coarse graded layer in 10 cm thickness spread over the test box floor. This layer also prevented cavitations and water wore effect on the soil. In Trautmann’s test load was applied mechanically to the pipe, while in here, real uplift force of the water is applied to.

According to the tests done on the local area soil which used in this study, it classified as a poor aggregated sand (SP) with 2.66 of the solid density and 0.00225 cm/sec of the permeability rate. Its minimum and maximum specific weight were 1.38 and 1.78gr/cm³, respectively. The internal friction angle for its loose case was equal to 32.9° and in dense case was 37.6°.
3.1. Laboratory Failure Mechanisms

In initial tests [7] using dense sand (γ=1.78 gr/cm³), pipe was placed in depth equal to its diameter, i.e. 11 cm- from its center level to the soil surface. During the loading, two mechanisms of failure was observed; first, an **angled sliding block mechanism**, when the soil resistance reached to its utmost strength against upheaval forces, an inclined slip surface (about 20° diversion from the normal direction) created. This failure mechanism happened under small upheaval displacement (about 2mm)- as shown in Figure 2(a)- and this followed by a quick reduction in resistance upheaval force. By increasing the upheaval displacement, uplift resistance more decreased, and a second mechanism called **circulation mechanism**, observed around the pipe. At this stage, two gaps below the pipe formed [Figure 2(b)] and by progressing the upward moving of the pipe, these gaps were filled by flow dropping of the upper soils [Figure 2(c)]. As the curve in Figure 2 depicts after 10mm of displacement, uplift resistance force reaches to a constant amount, called as the residual force, with breaking of the inter-locking between the soil particles and make the pipe buoyant, and a considerable uplift displacement.

![Figure 2. Load-displ. diagram of dense soil on the top of the buried pipe under upheaval force [7]](image-url)
For the loose sand, only a circular mechanism observed as the failure mechanism of the soil. In this case, there was no sign of the sliding surfaces (trivial interlocking) and from the beginning, only the two gaps created under the pipe, and further on, they were filled by flow dropping of the upper soil. Figure 3, illustrates the variation curves of the resistance load against upheaval displacement for different burial depths as well as sand densities.

As the curves depict, for embedment ratio of 0.7 and 1, the residual force in dense sand is less compared to the loose one. Its reason can be stated by this fact when the angled sliding block happened in dense soil, the soil remains above the pipe is less than the amount in case of loose sand. As the resistance shear force depends on the surcharge loads, then the experimental results is justified. Also, as the curves show the effect of the density is more than the burial depth ratio in uplift load carrying capacity of the soil.

Figure 3. Upheaval load-displ. curves for different burial depths of loose and dense sand [8]

4. FINITE ELEMENT MODELLING

2D finite element plane-strain analyses was carried out to investigate the uplift behavior of circular pipeline to identify the effect of the most important parameters- i.e. the soil density and the embedment depth ratio- for further studies as well as to understand the mechanics of the soil around the pipeline in more details. Uplift capacities and soil failure mechanisms were found for different soil conditions and burial depth ratios. The influence of the mesh size, soil stiffness, ratio between the soil permeability and the loading rate as well as the soil/pipe contact surface effect has not been considered at this instance.

4.1. Soil & Pipe Characteristics

Conventional 8-noded quadrilateral serendipity element used to model the pipe and
soil. Each node has two degrees of freedom of displacement. A non-associated Drucker-Prager elastic- perfectly plastic criterion chose to model soil material. This has a constant friction angle $\phi$, a non-associated dilatancy angle $\psi$, and cohesion stress $c$, apart of its elastic parameters $E$, $G$ and $\nu$. For this example some of the parameters have been speculated to fit suitably the behavior of the soil/pipe system. This can be supported by the facts which are explained in below;

1. Experimental data in the literature are not enough to provide all values of parameters are needed in the numerical modelling. Then some of the values (listed in Table 1) have been picked from other sources. This, would be the main source of discrepancy in the FEM results compared with the experiments, as the predictions are sensitive to in situ stresses and soil stiffness.

2. It should be pointed out that all analyses were run under plane-strain formulation which results more stiffer and higher load-carrying capacity structure compared to the plane-stress one, which is the more realistic option in 2D analyses for the tested buried pipe.

3. No potential cracking and fracture is modelled or allowed in the soil elements by loading progress. This also results a stiffer response of the numerical model.

<table>
<thead>
<tr>
<th>Material characteristics</th>
<th>soil</th>
<th>pipe</th>
</tr>
</thead>
<tbody>
<tr>
<td>E</td>
<td>3-4  MPa</td>
<td>8-10 MPa</td>
</tr>
<tr>
<td>$\gamma$</td>
<td>1.45 gr/cm$^3$</td>
<td>1.78 gr/cm$^3$</td>
</tr>
<tr>
<td>$\nu$</td>
<td>0.33</td>
<td>0.33</td>
</tr>
<tr>
<td>$\phi$</td>
<td>32.9°</td>
<td>37.6°</td>
</tr>
<tr>
<td>$c$</td>
<td>2 KPa</td>
<td>5-10 KPa</td>
</tr>
<tr>
<td>$\psi$</td>
<td>0°</td>
<td>0°</td>
</tr>
</tbody>
</table>

4. Dilatancy angle, measured by $\tan \psi$, was constant and set to zero. This means that soil material can slide over the other without producing any vertical displacement and this should affect only marginally the results. However, this is reasonable for the loose soil, but for the dense one, more explanation is presented. As laboratory tests show, dilatancy angle decreases to zero with increasing plastic shear slipping or increasing normal confining pressure. These phenomena occur often combined, particularly in confined structures (for instance deep soil, in here), because shear slip with dilatancy necessarily induces normal compressive stresses. For the concerned case in here, the combined action of these two factors will also produce a faster degradation of the dilatancy angle under increasing confining pressure on the soil.

5. NUMERICAL RESULTS

5.1. Dense Sand
Figure 4-left gives upheaval load-displacement curve of FEM analysis for the dense soil with embedment depth ratio (H/D) equal to 1, which well correlates with
the experimental results, with about 10 percent difference in peak load. The numerical results show a steeper initial slope of the curve compare to that obtained from the experimental results. This may be due to a much higher values of the stiffness parameter E, which has not been provided in the reference text [7]. The higher initial stiffness caused increase in resistance of the soil against more shear deformation by resulting delay in forming the sliding surface, and increasing the stiffness of the soil. Also, Figure 5-left depicts contours of $\varepsilon_{yy}$ on deformed mesh and Figure 5-right shows contours of the upheaval displacement, both at 0.4cm of vertical applied displacement. These pictures well demonstrate the sliding surface in dense sand which complies with the experiment.

More over, it is clear that none of the experimental and numerical results of the failure mechanism in dense soil, obeyed the vertical sliding surface theory.

As it was explained before, for dense soil, uplift resistance force reaches to its peak amount in small upheaval deflections. At this stage, the soil above the pipe has failed and this causes a progressive reduction in uplift force (the descending branch of the curve), to get its residual value in large displacements. Figure 4-right
compares load-upheaval displacement curves for other ratios of H/D, obtained from the numerical analyses. These curves show the amount of deduction in resistance force varies with the ratio of H/D. Numerical results give lesser reduction in the peak resistance load compare to the experimental ones.

5.2. Loose Sand

Figure 6 draws load-displacement curves obtained numerically for the loose soil.

![Figure 6](image)

All show by increasing the burial depth, upheaval load capacity has increased, but upheaval displacement corresponding to the peak load has decreased. However, numerical results in Figure 6-left gives a stiffer model compare to the experimental one with a steeper initial slope of the curve as well as a higher resistance peak load, but its failure mechanism has been well captured by the FEM results. As Figure 7.right shows, in loose sand, a larger region of deformed soil under circulation mechanism of the soil flow underneath the pipe has created (from beginning of the failure procedure). Figure 6-right compares load-upheaval displacement curves for other ratios of H/D, obtained by the numerical analyses which have the same trend explained for the dense sand. Figure 7-left also pictures deformed mesh at 0.4 cm of upheaval displacement for H/D = 1.3 (deflections magnified).

![Figure 7](image)
5.3. Further Studies
Figure 8-left depicts non-dimensional curves of the upheaval load-displacement variations for different ratios of the burial depth. The load-displacement results are re-plotted normalized by peak load and embedment depth, respectively. There is an excellent agreement between the results for the different embedment ratios, with all showing similar normalized stiffness and that d/H equal to 1% for dense sand and 1.5% for the loose sand. However, there is a bit concern for the correlation of the curves after peak load, specially in dense sand, which is, of course, of some shortcomings data, explained earlier. Also, there is less good agreement between the normalized curves when the results are normalized by pipe diameter (not presented here).

![Figure 9. Normalized load-displacement curves (left) for dense sand and; (right) for loose sand](image)

![Figure 9. Normalized uplift peak load for various H/D ratios of dense and loose sand](image)

Also, according to Eq. 3, normalized uplift peak load (N_u) has drawn as a function of H/D, in Figure 9, for dense and loose sand. As this figure shows, normalized peak load is increased by increasing the burial depth and it is more in dense sand compare to loose one for an equal depth ratio.

6. CONCLUSION
A series of Finite Element analyses have been described to examine the soil density and the embedment depth ratio on the uplift capacity and corresponding displacement on the failure of a circular buried pipeline subjected to vertical uplift. Finite Element model gave a very good approximation to the system behavior and
was able to reproduce the complete deformation pattern of the system up to and beyond the peak load until total degradation of strength, without major numerical difficulties. The following conclusions have been drawn from this study:

1. Type and mechanism of failure differs in dense and loose sand. In dense sand, only sliding block mechanism with angled surfaces is formed, in small upheaval deformations, while in loose sand, deformation of wide range of the soil above the pipe is happening.

2. Corresponding displacement for the uplift peak load is much more in loose sand compare to the dense one, in the same H/D ratio.

3. Effect of the increase in soil density on uplift capacity is more than the effect of increase in burial depth ratio, which is very important of economical view.

4. Deduction in peak resistance force capacity to reach the residual force in dense sand is more then the loose one, as a result of different failure mechanisms.

5. Normalized $N_u$ is increased with the embedment ratio, and its peak amount is more in the dense sand compare to loose one, for an equal H/D.

6. FEM results suggest that displacement should normalized by embedment depth, H, rather than pipe diameter, D.

7. Obtaining some crucial parameters carefully and some more refinement, FEM has potential for use in defining advanced design parameters and rules.

REFERENCES


A SENSITIVITY ANALYSIS ON THE CHLORIDE-INDUCED CORROSION INITIATION TIME OF RC ELEMENTS

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ABSTRACT
One of the most important causes for reinforcing steel corrosion is the presence, ingress and attack of chloride ions on RC elements. Reinforcement corrosion has been widely reported in the literature over the last two to three decades. They cause localized breakdown of the passive film that initially formed on steel as a result of the alkaline nature of the pore solution in concrete. In this paper, a sensitivity study was carried out on the influence of the effective parameters of corrosion on the corrosion initiation time, both in uncracked and cracked concrete elements. The results of the study shows a high sensitivity of the corrosion initiation time, regarding \(C_s\) (chloride surface concentration) and \(C\) (cover depth), in uncracked concrete, and on the other side, more sensitivity of the ratio of the \(w/l\) (crack width to crack spacing) in cracked concrete elements.

Keywords: reinforcement corrosion; sensitivity analysis; electrochemical process; corrosion initiation time cracked concrete

1. INTRODUCTION
Reinforcement corrosion has been widely reported in the literature over the last two to three decades \(^1\), \(^1\). Chloride-induced corrosion of the reinforcing steel is known to be a major cause of premature rehabilitation of many RC structures like bridge decks \(^3\). Moreover, reinforcement corrosion in concrete is the predominant causal factor for the premature deterioration of reinforced concrete structures, leading to structural failure. Failure does not necessarily mean structural collapse only, but also includes loss of serviceability, characterized by concrete cracking, spalling, and excessive deflection. Clearly detection and monitoring of reinforcement corrosion in concrete is of significant practical importance if premature failure of RC structures is to be prevented \(^3\). In this regard, the present paper tries to define the corrosion of reinforcements in concrete, mostly due to the chloride ion ingresses, develop and describe the advanced models for chloride-induced corrosion both in cracked and uncracked elements and finally, study the influence of various factors on the corrosion initiation life period.
2. CORROSION OF REINFORCEMENTS
Because of high alkalinity of concrete media 1, 1, 4, 5, (pH >13.5), steel remains passive within the concrete by forming a thin oxide layer on steel rebar surface. In addition, well-consolidated and properly cured concrete with a low w/c ratio has a low permeability, which minimizes ingress of detrimental stuffs either gaseous or liquid, such as chloride, carbon dioxide, oxygen, moisture, and etc. to the steel surface. Furthermore, high electrical resistivity of concrete 1 restricts the rate of corrosion by reducing the flow of electrical current from the anodic to the cathodic sites. The passive film remains stable as long as the composition of the pore solution remains constant. The protective film is destroyed when there is a sufficient concentration of chloride ions and or carbonation around the reinforcement. Therese ions and other detrimental, stemming from environment, deicing salts, and etc penetrate into concrete. Once the reinforcing steel was depassivated and it was supplied with oxygen and water (humidity), metal dissolution (corrosion in the form of rust formation, loss in cross section) may start. 6

3. ELECTROCHEMICAL PROCESS OF CORROSION
Reinforcement corrosion is an electrochemical process. Similar to a flash battery, corrosion take place by coupled cathodic and anodic reactions in an electrolyte like concrete pore water (known as complex electrolyte). The surface of the corroding steel functions as a mixed electrode that is a composite of anodes and cathodes electrically connected through the body of steel itself, upon which coupled anodic and cathodic reactions take place. Concrete pore water functions as an aqueous medium, i.e., a complex electrolyte. Therefore, a reinforcement corrosion cell is formed.

3.1. Anodic and Cathodic Reactions
Generally, anodic and cathodic reactions are referred as oxidation and reduction process respectively. At anode, dissolution of metallic steel occurs, while at the cathode the reaction leads to reduction of dissolved oxygen to form hydroxyl ions. By the way, the possible anodic and cathodic reactions are as follows 1, 7, 1, and 7. At anode;

\[ 3Fe + 4H_2O \rightarrow Fe_3O_4 + 8H^+ + 8e^- \]  (1)

\[ 2Fe + 3H_2O \rightarrow Fe_2O_3 + 6H^+ + 6e^- \]  (2)

\[ Fe + 2H_2O \rightarrow HFeO_4^- + 3H^+ + 2e^- \]  (3)

\[ Fe \rightarrow Fe^{2+} + 2e^- \]  (4)

At cathode;

\[ 2H_2O + 4O_2 + 4e^- \rightarrow 4OH^- \]  (5)

Or

\[ 2H^+ + 2e^- \rightarrow H_2 \]  (6)
Table 1: State of reinforcement corrosion at various pH levels

<table>
<thead>
<tr>
<th>pH of concrete</th>
<th>State of reinforcement corrosion</th>
</tr>
</thead>
<tbody>
<tr>
<td>Below 9.5</td>
<td>Commencement of steel corrosion</td>
</tr>
<tr>
<td>At 8.0</td>
<td>Passive film on the steel surface disappears</td>
</tr>
<tr>
<td>Below 7</td>
<td>Catastrophic corrosion occurs</td>
</tr>
</tbody>
</table>

Table 2: Corrosion risk in concrete containing chlorides

<table>
<thead>
<tr>
<th>Total chloride (wt.% of cement)</th>
<th>Condition of concrete adjacent to reinforcement</th>
<th>Corrosion risk</th>
</tr>
</thead>
<tbody>
<tr>
<td>Less than 0.4% Carbonated</td>
<td>Uncarbonated, made with cement containing less than 8% C₃A</td>
<td>Moderate</td>
</tr>
<tr>
<td>High</td>
<td>As above</td>
<td>High</td>
</tr>
<tr>
<td>High</td>
<td>Uncarbonated, made with cement containing 8% or more C₃A</td>
<td>Low</td>
</tr>
<tr>
<td>0.4% - 1.0 %</td>
<td>As above</td>
<td>High</td>
</tr>
<tr>
<td>More than 1.0%</td>
<td>As above</td>
<td>Moderate</td>
</tr>
<tr>
<td></td>
<td>All cases</td>
<td>High</td>
</tr>
</tbody>
</table>

4. EFFECT OF ACIDIC GASEOUS AGENT IN CORROSION

The corrosive effect of carbonation and other acidic gases, such as SO₂ and NO₂ are due to their tendency to reduce the pH of the concrete. The fall of pH to a certain level may cause commencement of reinforcement corrosion, loss of passivity of concrete against reinforcement corrosion, and catastrophic reinforcement corrosion as indicated in Table 1.

Chloride in concrete may be in the forms of: i) Acid soluble chloride, which is equal the total amount of chloride present in the concrete or that is soluble in nitric acid, ii) Bound chloride, which is the sum of chemically bound chloride with hydration products of the cement, such as the C₃A or C₄AF phases, and loosely bound chloride with C–S–H gel, and iii) Free or water-soluble chloride, which is the concentration of free chloride ions (Cl⁻) within the pore solution of concrete, and is extractable in water under defined conditions. It is generally recognized that only the free chloride ions influence the corrosion process. It is reported that the receptivity decreases and corrosion rate increases with an increase in the chloride content. However, the change in pH is found to be insignificant due to a change in the chloride content of concrete. The risk of reinforcement corrosion associated with the levels of chloride content in both uncarbonated and carbonated concrete is presented in Table 2.

5. CHLORIDE-INDUCED CORROSION MODELING

5.1. Chloride Diffusion Model

Chloride (or ion) diffusion is a specific case of scalar field problem that are encountered in almost all branches of engineering and physics. Most of them can viewed as special forms of the general Helmholtz equation given by...
\[
\frac{\partial}{\partial x} \left( k_x \frac{\partial \phi}{\partial x} \right) + \frac{\partial}{\partial y} \left( k_y \frac{\partial \phi}{\partial y} \right) + \frac{\partial}{\partial z} \left( k_z \frac{\partial \phi}{\partial z} \right) + Q = \rho \frac{\partial \phi}{\partial t}
\]  

(7)

where \( \phi(x, y, z) \) is the field variable to be solved.

For the diffusion of Chloride in one dimension, it can be shown that the governing equation reduced to Fick's Second Law of Diffusion:

\[
\frac{\partial c}{\partial t} = D \frac{\partial^2 c}{\partial x^2}
\]  

(8)

Clifton (1993) derived a closed form solution to Fick’s Law using an apparent diffusion constant \( D_{ac} \). Beginning with Fick’s Law and assuming a constant surface chloride concentration and an initial chloride-free surface, the closed form solution is

\[
C(x, t) = C_0 \left( 1 - \text{erf} \left( \frac{x}{2 \sqrt{D_{ac} t}} \right) \right)
\]  

(9)

Where \( C_0 \) is the constant surface concentration, \( \text{kg/m}^3 \), \( t \) is the time of exposure to \( C_0 \), \( \text{years} \) and \( D_{ac} \) is the apparent diffusion constant, \( \text{cm}^2/\text{year} \). By solving the Eq. 9 on a continuum media like a RC bridge deck shown in Figure 1, the time to corrosion initiated could be gained as follows:

\[
T_i = \frac{C^2}{4D_c} \left[ \text{erf}^{-1} \left( \frac{C_s - C_{cr}}{C_s} \right) \right]^2
\]  

(10)

Where \( T_i \) is the corrosion initiation time and other terms are defined before.

5.2. Time-Dependent Diffusion Coefficient

Midgley et al. (1984) conducted experiments using samples of hardened cement pastes 12. They calculated apparent diffusion coefficient from the quantity of

\[
\frac{\partial c}{\partial t} = D_t \frac{\partial^2 c}{\partial x^2}
\]  

(11)

Where \( D_t \) is a time dependent diffusion coefficient and \( m \) is an empirical coefficient based on the concrete water-cement ratio (w/c). They cited work by Conjeaud and Buenfeld and Newman that indicated that the rate of ingress of chloride ions into concrete decreased with time. This finding has been strengthened by the work of Weyers et al. (1994) reported previously 12. Mangat and Molloy concluded that the underlying common cause of the time-dependence of chloride ion diffusion on the pore structure of concrete, which changes with time 13. They exposed mixes of concrete-to-sea water and monitored the chloride diffusion. All results showed that \( D_{ac} \) decreased with time 13.

5.3. Effect of Chloride Binding

Clifton, (1993) expressed that because chloride ions may react with the tricalcium aluminate of Portland cement (C₃A), the concentration has two components: concentration of bound chloride ions \( C_b \) and concentration of free ions \( C_f \) related by constant \( R \) 14

\[ C_b = R.C_f \]  

(12)

However, carbonation or sulfate ions can release the bound chloride ions, and usually \( R \) assumed to be 0 14. Moreover, Oh and Jang (2003) consider the effect of chloride binding in the ingress of the chloride into the concrete 15. They study the general form of Fick's law in the form as follows 15

\[
\frac{dC_f}{dt} = \frac{dC_f}{dC_i} \text{div}[D_{cl}\text{grad}(C_f)]
\]  

(13)

where \( C_i \) is the total chloride ions (per weight g/g), \( C_f \) is free chloride ions (per weight g/g), \( t \) is time, \( D_{cl} \) is diffusion coefficient, and \( dC_f / dC_i \) is the binding capacity 15. The binding capacity is herein defined as the ratio between the free chloride and total chloride ions and can be rewritten as follows 15

\[ C_i = C_f + C_b \]  

(14)

\[
\frac{dC_f}{dC_i} = \frac{1}{1 + (dC_b / dC_f) b}
\]  

(15)
where $C_b$ is the bound chloride ions (per concrete weight $g_{cl}/g_{con}$), and $dC_b/dC_f$ represent binding isotherm is usually affected by concrete mixture characteristics, and some features have been considered in their study. The base of chloride isotherm is that some chloride ions penetrated in to the concrete are bound and do not affect the corrosion of steel bars. This isotherm is directly related to the amount of calcium silicate hydrate (C-S-H) that is found during the hydration process of cement. Xi, and Bazant (1999) 16 and Jennings and Tennis (1994) 17 are also proposed modified Powers Model to represent the chloride binding isotherm. The overall form of the model is 15, 16, and 17

$$C_b' = (C_f')^t 10^8$$

where $C_b'$ is the bound chloride ions (per gel weight $mg_{cl}/mg_{gel}$), $C_f'$ is the free chloride ions (per gel weight $mg_{cl}/mg_{gel}$), A and B are constants 15.

5.4. Cracked Concrete
Boulfiza et al. (2003) [19] used Darcy's Law to predict the chloride ions ingress in uncracked and cracked concrete. They developed a rational model for this purpose [19]. This model considers water flow properties in matrix and crack, both in saturated and unsaturated cases. Boulfiza et al, proposed a Simplified Smeared Approach (SSA) to model the chloride ions ingress into the cracked concrete 18. In this approach, it is assumed that chloride ions ingress into cracked concrete can be approximated using Fick's Second Law of Diffusion in which the following average coefficient is used

$$D_{av} = D_0 + \frac{w}{l} D_{cr}$$

where $D_{av}$ is the average diffusion coefficient, $D_0$ is the diffusion coefficient of uncracked concrete, $w$ is the crack width, $l$ is the crack spacing, and $D_{cr}$ is the diffusion coefficient inside the crack, as shown in Figure 3 18. Hence, the average diffusion coefficient is proportional to the crack width and inversely proportional to the crack spacing. Also, they proposed a Simplified Discrete Approach (SDA) 18 to evaluate the chloride ingress into the concrete. In this approach, chloride diffusion through the crack has been shown to obey the following equation which proposed by Tsukahara and Umoto (2000) 18 , and Kato et al. (2005) 19.

$$C_c(x,t) = \alpha\sqrt{x} + s\sqrt{t}$$

where, $C_c$ is the chloride concentration, $x$ is the location along the crack wall starting from spacemen surface, $t$ is time, and $\alpha$ and $s$ are two empirical constants. Chloride diffusion through the matrix is assumed to obey Fick's law under variable conations and given by
Boulfiza et al. showed that the two methods (13 and 14) have a good agreement between in their predictions 18.

Li et al. (2003) 20, proposed a new solution for prediction of chloride ingress in reinforced concrete flexural members. They use fundamental mechanism of diffusion of chloride ions in concrete. Based on a combination of Knudsen flow and viscous flow, an analytical solution is derived for the prediction of chloride ingress in concrete flexural members (Figure 5) 20. Also they verified the proposed model by experimental results. By applying some boundary and initial condition on Fick's Law, Li et al., proposed the following equations for chloride ingress in the cracked-concrete:

when \( C_s(t) < C_r \)

\[
C(x,t) = C_s(t) + \left[ C_i - C_s(0) \right] \sum_{n=1}^{\infty} U_n(x,t,D_{c1}) - \sum_{n=1}^{\infty} \int_0^t C_i(\tau) U_n(x,t-\tau,D_{c1}) d\tau
\]  \hspace{1cm} (20)

if \( C_s(t) \) assumed to be constant as for almost all current solution, then the equation (14) becomes

\[
C(x,t) = C_s + \left[ C_i - C_s(0) \right] \sum_{n=1}^{\infty} U_n(x,t,D_{c1})
\]  \hspace{1cm} (21)

when \( C_s(t) > C_r \),

\[
C(x,t) = C_s(t) + \sum_{n=1}^{\infty} \overline{h}_n U_n(x,t,D_{c2}) - \sum_{n=1}^{\infty} \int_0^t C_i(\tau) \left[ U_n(x,T_s-\tau,D_{c2}) - U_n(x,T_s-\tau,D_{c2}) \right] d\tau
\]  \hspace{1cm} (22)

where,

\[
\overline{h}_n = [C_i(0) - C_s(0)] \frac{U_n(x,T_s,D_{c2})}{U_n(x,T_s,D_{c2})} - \sum_{n=1}^{\infty} \int_0^t C_i(\tau) \left[ \frac{U_n(x,T_s-\tau,D_{c2}) - U_n(x,T_s-\tau,D_{c2})}{U_n(x,T_s,D_{c2})} \right] d\tau
\]  \hspace{1cm} (23)

and when \( C_s(t) \) assumed to be constant, then the equation (17) becomes
\[
C(x,t) = C_i + (C_s - C_i) \sum_{n=1}^{\infty} \frac{U_n(x,T_r,D_{c,i})U_n(x,t,D_{c,2})}{U_n(x,T_r,D_{c,2})}
\]

where, in the above equations, \(C_i\) is the initial chloride concentration, \(C_s\) is the surface chloride concentration, \(C_r\) is the critical chloride concentration, \(t\) is the time, \(T_r\) is the time at which the critical chloride concentration is attained and, \(x\) is the location. And,

\[
U_n(x,t,D_c) = \frac{4}{(2n-1)\pi} \frac{D_{c,i} \ln{2}}{4^n} \sin \left( \frac{(2n-1)\pi x}{2l} \right)
\]

where, \(D_c = 2\rho_c \lambda_c\), \(D_{c,i} = 2\rho_c^0 \lambda_c\), where \(\lambda_c\) is the mean free path of chloride ions, and \(\rho_c\) is the average velocity of chloride ions, and \(D_c = \xi w^2\); in which \(\xi\) is an empirical calibration factor, \(w\) is the width of crack, and \(l\) is the length of crack 20[21].

5.5. Non-Constant Surface Chloride Concentration

In concrete bridge decks, surface chlorides are derived from the deicing salts used during winter maintenance operations, and in particular locations, from exposure to sea water 21. Kassir and Ghosn 2002), showed that surface chloride concentration may not be constant 21. Phurkhaa and Kassir (2005) modeled surface chloride by a ramp-type variation 22, so in this case the boundary condition change to

\[
C(0,t) = f(t)
\]

where

\[
f(t) = \begin{cases} 
  C_0 t, & 0 \leq t \leq t_0 \\
  C_0, & t \geq t_0 
\end{cases}
\]

They solved the Fick's law (Eq. 7) for the above boundary condition 22. By applying some mathematical techniques, they derived a closed form solution to initiation time of chloride-induced corrosion as follows

\[
\frac{C(x,t)}{C_0} = g(x,t), \quad t \leq t_0
\]

\[
\frac{C(x,t)}{C_0} = g(x,t) - g(x,t-t_0), \quad t \geq t_0
\]

where

\[
g(x,t) = t \left[ -\frac{x}{\sqrt{\pi D_0 t}} e^{-x^2/(4D_0 t)} + \left( 1 + \frac{x^2}{2D_0 t} \right) \text{erfc} \left( \frac{x}{\sqrt{4D_0 t}} \right) \right]
\]
The result of numerical study showed that in the ramped-type chloride case, the time predicted to initiate corrosion is greater than the constant chloride concentration one [21-23].

6. SENSITIVITY ANALYSIS STUDY
Here some numerical results are presented to verify the above mentioned models. The effect of the various terms in the diffusion of chloride ions into the uncracked and cracked RC structures are evaluated respectively:

6.1. Uncracked Concrete
Sensitivity of diffusion coefficient ($D_c$)
Figure 3 shows the effect of diffusion coefficient on the corrosion initiation time. It can be seen that by 10 times decrease in diffusion coefficient the initiation time increased up to 10 times. This shows a linear inverse relationship between $T_i$ and $D_c$ as in model Eq. 9.

![Figure 3. Effect of $D_c$ on corrosion initiation time ($C_s=0.5\%$, $C_{cr}=0.275\%$)](image)

6.2. Sensitivity of Surface Chloride Concentration ($C_s$)
The effects of surface concentration of chloride ions are shown in Figure 4. As seen in this figure, two times increasing in $C_s$, leads to 224 times decrease in corrosion initiation time. This finding shows a strong relation between $C_s$ and $T_i$. 
6.3. Sensitivity of Critical Chloride Concentration ($C_{CR}$)

It is a natural effect that by increasing the threshold value for corrosion initiation which here, is the critical chloride concentration, leads in longer time for corrosion initiation. This effect is shown in Figure 5. It can be seen that three times increase of $C_{cr}$ would increase $T_i$ about 3.9 times.

6.4. Sensitivity of Cover Depth Thickness (C)

Figure 6 shows the effect of cover depth on the corrosion initiation time. It is obvious that the more concrete available at the outer surface of the reinforcements, the time to chloride ions to reach the steel surface would be reduced. This can be verified in Figure 6, when 3 times increase in cover depth leads to 9 times decrease in corrosion initiation time.
6.2. Sensitivity of crack effect

Based on the Simplified Smeared Approach (SSA) developed by Boulfiza et al., three crack types assumed for RC elements (Figure 2) and the corrosion initiation time was estimated as shown in Figure 7. As depicted here, by increasing the crack width and crack spacing, the corrosion initiation time would be reduced in a logarithmic manner. By doubling the crack width, $T_i$ increase up to 1.5 times, however, by 10 times increase in crack spacing, cause $T_i$ to be decrease up to about 6 times.

7. CONCLUSION

In this paper, the corrosion of steel embedded in concrete as the reinforcements was studied. The main factors affecting the electrochemical corrosion process of reinforcements were described and anodic-cathodic reactions of corrosion were
introduced. Various chloride-induce corrosion models were explained based on the conceptual differential equations. The time to corrosion initiation was explored both in uncracked and cracked concrete. Moreover a numerical study was carried out on the effects of the main factors of the corrosion process on the corrosion initiation time in intact concrete structures and also in cracked concrete structures. It was concluded that the surface chloride concentration has main influence on the corrosion initiation time. Moreover it was found that corrosion initiation time is very sensitive to the cover depth of concrete and the ratio of crack width to the crack spacing.

REFERENCES
EFFECT OF REBAR ON COMPRESSIVE STRENGTH OF CONCRETE CORES

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ABSTRACT
Concrete coring is used for determining compressive strength of hardened concrete of elements to evaluate low-strength-test-result concrete or to understand concrete placing quality of existing structure. In safety evaluation of existing structures that need rehabilitation and retrofitting, it's necessary to have some information about existing concrete. The best and most accurate method to determine compressive strength of existing concrete is coring. Rebars always make difficulty in coring of reinforced concrete structures. Sometimes it is not possible to coring from plain concrete areas. There are serious considerations for effects of rebar on compressive strength of concrete cores in some countries. There is no strength correction for the effect of rebars in ACI and also in Iranian related codes and specifications. There is only an Concrete Society equation in publication no.207 of BHRC based on result corrections of cores contained rebars perpendicular to cores longitudinal axes. This correction does not exceed 10%. Dealing with such problems in a project and because there was sureness about strength of placed concrete, but strength of cores was highly lower and it was not possible to obtain plain core as there was heavy rebar concentration, it was necessary to study rebar effects on core strength. In this research, strength of cores of an element contained rebars and plain ones were compared to evaluate above mentioned equations. Also some cylindrical samples were made and tested with placed rebars. In addition if the core has uncut rebars, there will be no significant reduction in its strength. It seems that cutting of rebars in coring process has little effects on concrete quality. It is due to cracks formation between concrete and rebars that reduce the concrete strength.

Keywords: compressive strength, core, rebar, reinforced concrete, structure

1. INTRODUCTION
If any strength test of laboratory-cured cylinders falls below f'c by more than the given values or if tests of field-cured cylinders indicate deficiencies in protection and curing, steps shall be taken to assure that load-carrying capacity of the structure is not jeopardized [1].
If the likelihood of low-strength concrete is confirmed and calculations indicate that load-carrying capacity is significantly reduced, tests of cores drilled from the area in question in accordance with “Method of Obtaining and Testing Drilled
Cores and Sawed Beams of Concrete” (ASTM C 42) shall be permitted [1,3,4]. If by structural analysis, again it can not be accepted, ordinary solution to accept concrete structurally and according to strength criterion, especially in Iran, is coring critical parts of concrete structure [3,4].

Nondestructive tests of the concrete in place, such as by probe penetration, impact hammer, ultrasonic pulse velocity or pull out may be useful in determining whether or not a portion of the structure actually contains low-strength concrete. Such tests are of value primarily for comparisons within the same job rather than as quantitative measures of strength. For cores, if required, conservatively safe acceptance criteria are provided that should ensure structural adequacy for virtually any type of construction [1].

Some specific consideration such as preparing suitable diameter and height, not contacting with reinforcements if possible, positioning critical areas, and some nondestructive tests such as rebar locating test, impact hammer and ultrasonic pulse velocity are needed for coring.

In any case, if rebars are closely spaced or rebars positions can not be determined, rebars will be cut and core will contain them.

Rebar existing in the core has always been debatable subject. Some consider that rebar increase sample strength; others believe that strength will be reduced when rebar exist; the others believe in very low effect of rebar presentation [6].

In some codes and standards like ACI, ASTM, EN, Iranian Concrete code and other Iranian codes, there is no consideration for rebar effect on core, therefore there is not any correction for core test result [1,2,3,4]. In some European countries there is no correction, but BS 1881:1983 and Concrete Society presented corrections for rebar existing in concrete cores [6].

Equation (1) generally works for samples that rebar are parallel to ends of sample.

\[
\text{StrengthCorrectionFactor} = 1 + \frac{3 \sum_{i=1}^{n} (d_{r,i} h_i)}{2d_c L}
\]

Where, \(d_{r,i}\) is rebar diameter, \(d_c\) is core diameter, \(h_i\) is rebar axis distance to next core surface and \(L\) is core length[5].

In equation (1), effects of rebar depend on diameter, location and number of rebar, but this correction does not exceed 25% of core strength [7].

If it is possible, the best solution is obtaining cores from plain concrete areas. Perhaps rebars cutting leads to adverse effects on structural strength, concrete quality and core strength [6].

Existing no quality control in precast concrete beams construction for bridges of a road in the South of Iran, supervisor selected coring as only practical solution for quality control of concrete of these beams. Compressive strength of cores was astonishing because there was enough confidence in strength of placed concrete, but strength of cores was too low. Accordingly, it was necessary to study rebar effects on core strength.
2. EXPERIMENTAL PROGRAM AND MATERIALS

To determine rebar effect on concrete core strength, two groups of concrete beams is used. In the first group, there are beams without rebar, and beams with rebars were the second group (Figure 1). The concrete that is used for construction of each group, is similar and also with same consolidation and curing conditions. By coring plain beams and determining their strengths, it is possible to compare results. In some beams, one, two and even three rebar was placed. Beam is a concrete block with dimension of $15 \times 15 \times 60$ cm and core is a cylinder with 10cm diameter and 15cm height (Figure 2 and 3). As shown in Table 1, two different kinds of concrete mix proportion were used in this research. Nominal maximum size of aggregates was 20 mm.

![Figure 1. Rebars placement in beams](image)

![Figure 2. Coring of beams](image)

![Figure 3. Cored beams](image)

<table>
<thead>
<tr>
<th>Mix No.</th>
<th>Cement kg/m³</th>
<th>SilicaFume kg/m³</th>
<th>W/C</th>
<th>Sand (SSD) kg/m³</th>
<th>Coarse Aggregate (SSD) kg/m³</th>
<th>Fresh Concrete Density kg/m³</th>
<th>Slump mm.</th>
<th>HRWRA Admixture kg/m³</th>
<th>Air Content %</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>370</td>
<td>30</td>
<td>0.4</td>
<td>925</td>
<td>850</td>
<td>2328</td>
<td>120</td>
<td>3.2</td>
<td>2.4</td>
</tr>
<tr>
<td>2</td>
<td>400</td>
<td>-</td>
<td>0.43</td>
<td>980</td>
<td>775</td>
<td>2328</td>
<td>140</td>
<td>2.8</td>
<td>2.1</td>
</tr>
</tbody>
</table>

Concretes without rebar are determined by CP and ones with rebar by CR. Three cores were prepared for each mix proportion. Rebar sizes are 10, 12, 16, 20, 32 mm.
diameter and different rebar arrangement, as shown in table 2, was used. These arrangements tried to be same as real rebar arrangements in real beams. In another part of this research, some cylindrical samples with mix proportion no.2 and with or without rebar were prepared. Rebars placed in some samples are shorter than sample diameter and they are not cut. After 28-day same curing, tests were fulfilled. Cylindrical samples with rebar are distinguished with P and ones without rebar with letter R.

3. EXPERIMENTS RESULTS AND ANALYSIS
28-day Compressive strength results of cores in saturated state after capping as well as weights, number, diameter and arrangement of rebars are presented in Table 2. Also corrected strength of cores with rebar by using Concrete Society equation (mentioned above) is presented in this table. Also results of cylindrical samples with and without rebar are shown in table 3. Cylindrical samples are with 150 mm diameter and 300 mm height. According to Tables 2 and 3, in all cores with rebar there is a reduction of 25 to 60 percent than cores without rebar. It seems that existing of rebars results in weakening of cores. In cylindrical samples, existence of rebar has reduced sample strength 16 to 24 percent. After core breaking-up, it observed that usually rebar were separated from adjacent concrete matrix (Figure 4). It also seems that in addition to weak connection resulting from bleeding that water gathers under rebar, act of rebar cutting weakens connection of rebar and concrete. Cores strength variation does not follow any rule and there is much variation. Cores strength correction result from Concrete Society equation does not show good proximity to strength of cores without rebar except CR-2-1, CR-2-2 and CR-2-4 cores. Results of cylindrical samples with rebar after correction by Concrete Society equation show better correlation with samples without rebar. This demonstrates that rebar cutting weakens concrete samples.

![Figure 4. Separation of rebars after compression test of cores](image-url)
### Table 2: Concrete Cores Test Results

<table>
<thead>
<tr>
<th>Core No.</th>
<th>Rebar quantity and diameter (mm)</th>
<th>Rebar location (mm)</th>
<th>Core weight (kg)</th>
<th>Measured compressive strength (kg/cm²)</th>
<th>Corrected strength by Concrete Society equation (kg/cm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CP-1-1</td>
<td>-</td>
<td>-</td>
<td>2.240</td>
<td>490</td>
<td>-</td>
</tr>
<tr>
<td>CP-1-2</td>
<td>-</td>
<td>-</td>
<td>2.235</td>
<td>461</td>
<td>-</td>
</tr>
<tr>
<td>CP-1-3</td>
<td>-</td>
<td>-</td>
<td>2.235</td>
<td>475</td>
<td>-</td>
</tr>
<tr>
<td>CR-1-1</td>
<td>1Ø32</td>
<td>175</td>
<td>2.380</td>
<td>260</td>
<td>322.5</td>
</tr>
<tr>
<td>CR-1-2</td>
<td>1Ø32</td>
<td>175</td>
<td>2.365</td>
<td>231</td>
<td>286.5</td>
</tr>
<tr>
<td>CR-1-3</td>
<td>1Ø12, 1Ø20</td>
<td>150</td>
<td>2.370</td>
<td>260</td>
<td>301.5</td>
</tr>
<tr>
<td>CR-1-4</td>
<td>1Ø12, 1Ø20</td>
<td>150</td>
<td>2.375</td>
<td>212</td>
<td>246</td>
</tr>
<tr>
<td>CR-1-5</td>
<td>2Ø16</td>
<td>150</td>
<td>2.380</td>
<td>360</td>
<td>417.5</td>
</tr>
<tr>
<td>CR-1-6</td>
<td>2Ø16</td>
<td>150</td>
<td>2.375</td>
<td>320</td>
<td>371</td>
</tr>
<tr>
<td>CP-2-1</td>
<td>-</td>
<td>-</td>
<td>2.220</td>
<td>375</td>
<td>-</td>
</tr>
<tr>
<td>CP-2-2</td>
<td>-</td>
<td>-</td>
<td>2.235</td>
<td>404</td>
<td>-</td>
</tr>
<tr>
<td>CP-2-3</td>
<td>-</td>
<td>-</td>
<td>2.280</td>
<td>388</td>
<td>-</td>
</tr>
<tr>
<td>CR-2-1</td>
<td>1Ø32</td>
<td>175</td>
<td>2.310</td>
<td>303</td>
<td>375.5</td>
</tr>
<tr>
<td>CR-2-2</td>
<td>1Ø32</td>
<td>175</td>
<td>2.325</td>
<td>303</td>
<td>375.5</td>
</tr>
<tr>
<td>CR-2-3</td>
<td>1Ø32</td>
<td>150</td>
<td>2.310</td>
<td>245</td>
<td>284</td>
</tr>
<tr>
<td>CR-2-4</td>
<td>1Ø32</td>
<td>175</td>
<td>2.310</td>
<td>303</td>
<td>375.5</td>
</tr>
<tr>
<td>CR-2-5</td>
<td>2Ø10, 1Ø20</td>
<td>150</td>
<td>2.360</td>
<td>231</td>
<td>277</td>
</tr>
<tr>
<td>CR-2-6</td>
<td>2Ø10, 1Ø20</td>
<td>150</td>
<td>2.320</td>
<td>274</td>
<td>323</td>
</tr>
<tr>
<td>CR-2-7</td>
<td>1Ø12, 1Ø20</td>
<td>150</td>
<td>2.330</td>
<td>260</td>
<td>301.5</td>
</tr>
<tr>
<td>CR-2-8</td>
<td>1Ø12, 1Ø20</td>
<td>175</td>
<td>2.330</td>
<td>144</td>
<td>178.5</td>
</tr>
<tr>
<td>CR-2-9</td>
<td>1Ø16, 1Ø20</td>
<td>150</td>
<td>2.300</td>
<td>216</td>
<td>255</td>
</tr>
<tr>
<td>CR-2-10</td>
<td>1Ø16, 1Ø20</td>
<td>175</td>
<td>2.315</td>
<td>122</td>
<td>155</td>
</tr>
<tr>
<td>CR-2-11</td>
<td>2Ø16</td>
<td>175</td>
<td>2.325</td>
<td>260</td>
<td>322.5</td>
</tr>
<tr>
<td>CR-2-12</td>
<td>2Ø12</td>
<td>175</td>
<td>2.320</td>
<td>231</td>
<td>259</td>
</tr>
</tbody>
</table>
Table 3: Cylindrical Concrete Samples Test Results

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Rebar quantity and diameter (mm)</th>
<th>Rebar location (mm)</th>
<th>Measured compressive strength (kg/cm²)</th>
<th>Corrected strength by Concrete Society equation (kg/cm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>P-2-1</td>
<td>-</td>
<td>-</td>
<td>342</td>
<td>-</td>
</tr>
<tr>
<td>P-2-2</td>
<td>-</td>
<td>-</td>
<td>328</td>
<td>-</td>
</tr>
<tr>
<td>P-2-3</td>
<td>-</td>
<td>-</td>
<td>323</td>
<td>-</td>
</tr>
<tr>
<td>P-2-4</td>
<td>-</td>
<td>-</td>
<td>337</td>
<td>-</td>
</tr>
<tr>
<td>R-2-1</td>
<td>1Φ16 1Φ12</td>
<td>150</td>
<td>253</td>
<td>288.5</td>
</tr>
<tr>
<td>R-2-2</td>
<td>1Φ16 1Φ12</td>
<td>150</td>
<td>267</td>
<td>304.5</td>
</tr>
<tr>
<td>R-2-3</td>
<td>2Φ16 1Φ32</td>
<td>150</td>
<td>252</td>
<td>-</td>
</tr>
<tr>
<td>R-2-4</td>
<td>2Φ16 1Φ32</td>
<td>150</td>
<td>257</td>
<td>-</td>
</tr>
<tr>
<td>R-2-5</td>
<td>2Φ16 1Φ32</td>
<td>100</td>
<td>279</td>
<td>338.5</td>
</tr>
<tr>
<td>R-2-6</td>
<td>2Φ16 1Φ32</td>
<td>100</td>
<td>266</td>
<td>322.5</td>
</tr>
</tbody>
</table>

CONCLUSION
According to obtained results and their analysis, following conclusions can be deduced.
1. The equation given by Concrete Society does not predict the core results accurately.
2. Compressive strength of core with cut rebar is always less than corresponding strength of core without rebar.
3. It appears that strength reduction due to existence of rebar in cores is between 25 to 60 percent. This reduction in cast cylindrical samples is about 16 to 24 percent.
4. It seems that rebar cutting leads to weakness in cores compressive strength due to crack formation between concrete and rebars.
5. There is not reliable relationship for correcting compressive strength results of cores with rebar. But in limited cases, correction of Concrete Society equation can give acceptable results.
6. Strength reduction of cast cylindrical samples with rebar is usually less than that of concrete cores.

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REFERENCES

STUDY OF NEW GROUTING TECHNIQUES IN REPAIR AND STRENGTHENING OF SOFT SOILS

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ABSTRACT

Demands for soft land strengthening and contaminated soil solidification have been increased due to the explosion population, development of technology, industry, and transportation systems.

Deep mixing methods (DMMs) are conventional methods for strengthening the soft soils. Cement (or binder) with soil have been mixed in this method. Injection of cement and additives with a high pressure and displacing the gases and liquids from within soil grains in combination with DMMs is a new wedding technique for increasing the efficiency of soil stabilization.

In this paper, different kinds of deep mixing methods (conventional and new techniques) were studied. By considering the type and alternative layers of soil, load size, the situation, cost and type of project selecting the right stabilizing method for repair and strengthening of soft soils recommended and it is believed that it could prompt engineers to resolve soil strengthening difficulties from the geotechnical view point.

Keywords: grouting, stabilizing, injection, mixing, soft soil, cement and binder

1. INTRODUCTION

1.1. Deep Mixing Methods (DMMs)

The DMMs refers to the rotary methods for the penetration of soil or rock and mixing of soil with suitable binder and at times with pneumatically dry or wet end for enough depth. This technique has been used for construction of tunnels by injecting binder in the rock too.

Based on design requirements, site conditions, soil layers, restraints, and economics, the use of DMMs is increasingly spreading. The demand for improving and stabilizing land for different purposes is expected to increase in the future and the best way to fulfill it is by using DMMs. The main advantage of these methods is long term increase in strength, especially for some of the binders used. It has been mentioned that DMMs is the best way to improve soils and rocks. The following are the characteristics of the improved soils and rocks when these methods are employed [1]:

(i) Reduction of settlements, (ii) increase of stability, (iii) increase of bearing capacity, (iv) prevention of sliding failure, (v) reduction of vibration, (vi) liquefaction mitigation, and (vii) remediation of contaminated ground.

The Federal Highway Administration has suggested that these techniques can be
classified based on [2]:
1) Method of additive injection (i.e. wet or dry injection).
2) Method by which additive is mixed (i.e. rotary, mechanical energy or by high pressure jet).
3) The location of the mixing tool (i.e. near the end of the drilling rods or along a portion of the drilling rods).

Majority of the companies which are working in these fields agree that the DMMs can be divided into three common techniques [9]:
1. Shallow soil mixing (SSM), which uses a single mechanical mixing auger located at the end of the drilling tool (Kelly bar).
2. Deep soil mixing (DSM), which utilizes a series of overlapping augers and mechanical mixing shafts.
3. Jet grouting which can be considered a type of soil mixing. In order to inject a liquid into voids within a structure, it is necessary to displace the gases and liquids from within these. This utilizes high velocity ranges from 28 to 42 Mpa backpressure and jets to hydraulically shear the soil and blends a cement grout or suitable binder to form a soil-cement column or column with soil and special binder.

Table 1 clearly names the methods which are utilized in each part. The explanation and comparison of each part will be made subsequently.

<table>
<thead>
<tr>
<th>Shallow soil mixing (SSM)</th>
<th>Deep soil mixing (DSM)</th>
<th>Jet grouting systems</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Single phase</td>
<td>Dual phase</td>
</tr>
<tr>
<td></td>
<td>I. Maxperm grouting system</td>
<td>Dry jet mixing system (DIM)</td>
</tr>
<tr>
<td></td>
<td>II. Navigational drilling system</td>
<td></td>
</tr>
<tr>
<td></td>
<td>III. Vacuum grouting injection</td>
<td></td>
</tr>
</tbody>
</table>

The researchers have come up to four basic jet grouting systems to be used, which are [4]:
(i) Single phase (grout injection only), (ii) dual phase (grout + air injection), (iii) triple phase (water+air injection and followed by grout injection), and (iv) super jet grouting (air injection+drilling fluid by grout injection). Figure 1 shows the sketches of the jet grouting systems.

**Figure 1. The systems in jet grouting.**
2. SHALLOW SOIL MIXING (SSM)

2.1. Ras-Columns

Ras-Columns are one of the most common soils mixing in DDMs which are based on mechanical soil mixing technology. This method has been used for improving shallow soils and seldom in deep mixing.

The mixing head is combined with blades which can rotate inversely. In other words, in bottom auger the mixing blades rotate clockwise and in the upper auger, the mixing blades rotate anti clockwise [3].

This technique causes the cement to mix with soil homogenously, and thus produces higher quality soil-cement columns. The first step of this method is Rig positioning. This is followed by penetration whereby after passing dry excavation zone, injecting slurry without any jetting should start. The third step is churning or moving the head up and down to mix homogenous thoroughly. Finally is the completion step, where the head is withdrawn and soil column cement is completed (Figure 2).

![Figure 2. The steps in ras-columns system [3].](image)

2.2. Advantages and Main Points

1. Ability to produce soil-cement column of 1.4 to 2.5 m diameter.
2. Applicable to a wide range of soils hence providing excellent quality improvement.
3. Uniform mixing and homogenous product quality due to counter-rotation mechanism (in comparison with traditional equipment).
4. Low noise and low vibration system.
5. Computer-based control and monitoring system ensures quality improvement (in some latest ones).

3. DEEP SOIL MIXING (DSM)

3.1. Cement Deep Mixing System (CDM)

The second method which is related to deep soil mixing and is one of the DMMs is CDM. As mentioned before, in this method a series of overlapping augers and mechanical mixing shafts are applied. Figures 3(a) and 3(b) show CDM machines with 2 and 3 augers respectively.

CDM is normally utilized in soft soil that contains mineral soils such as clay or sand. In some conditions where mineral soils are absent, sand should be added before mixing in cement slurry. CDM is a soil stabilization method which mixes cement slurry with soft soil in situ to attain a required strength. Soft soil is
stabilized by the 2-phase chemical reaction. A hydration reaction occurs and an ettringite of capillary crystals is generated when the cement mixes with water. Then a pozzolanic reaction follows, as the age grows, where the hydration product reacts with the clay minerals in the soil [3].

Figure 3: (a) The CDM machine with 2 augers (b) The CDM machine with 3 augers

3.2. Advantages and Main Points
1. CDM is a drilling and mixing operation with low noise and low vibration, and does not generate dust.
2. CDM method mixes soft soil in situ with cement slurry without any jetting. The soil should have mineral soils like sand and clay for hydration product with cement.
3. Because of a series of overlapping augers, it saves soil mixing time and labor while maintaining efficiency in comparison with previous method.
4. Computer-based control and monitoring system ensures quality improvement (in some latest ones).

4. JET GROUTING SYSTEMS
Jet grouting systems, which is the third part of DMMs, have some similarities with the previous methods. Apart from having the same mixing tools, this method also applies the same process whereby the in-situ soil will be cut and broken by high pressure jet of slurry and produce homogenously improved zone around the mechanically mixed core.

In addition, for underwater applications, it is desirable to have highly flowable grout that can resist water dilution and segregation, and spread readily into place. The slump of concrete or grout is a good measure of the consistency and flow characteristics of a concrete or grout mixture. This equates (to) a mid range slump. A very high slump grout gives maximum water dilution. A very low slump grout results in little or no flow characteristics. For underwater grout, the slump flow is influenced (in order of influence) by the anti-washout admixture concentration and the binder content, the water-cementitious material ratio, and the water reducer concentration [10, 11].
4.1. Single Phase
4.1.1. Maxperm Grouting System

It has been mentioned that the jet grouting is divided into 4 phases. The first part which is single phase has 4 common methods. One of the newest methods in jet grouting system is Maxperm grouting system. This method is commonly utilized around the world by several names such as dual-tube double-packer grouting system.

In this method, the contractor can inject several materials in the soil which has several layers with different characteristics. On the other hand, the ground is made up of alternative layers, consisting of different particle sizes, and permeability can be stabilized by this method. The pre-defined region is improved and its sketches are shown in Figure 4.

![Figure 4. Pre-defined region can be improved by maxperm grouting system.](image)

The following are the steps that take place in this method which are also shown subsequently in Figure 5[3]:

1. Apply casing drilling (ϕ=100 mm to a pre-defined depth).
2. Install the grout pipe with special sleeve packers and strainers then withdraw the casing.
3. Install the dual-wall inner tube equipped with double packer which inflates the
4.1.2. Advantages and Main Points
1. Cost-effective and labor saving alternative, because it can be separated and cause large borehole spacing.
2. Special layers, regions, any pre-defined point, a narrow area and ground with underground obstacles in soil can be improved by this method.
3. Large size improvement.
4. A pre-defined zone is homogenously stabilized by injecting grout from discrete injection points.
5. The system allows repetitive grouting at the same Injection point with different grout materials even after the work is completed.
6. This method can be used as a remedial method for structures.

4.1.3. Navigational Drilling System
The second method in single phase jet grouting is navigational drilling system. This is a new method which is broadly used. By using 3D navigational drilling system, horizontal grout holes can be installed from the surface without excavating a shaft.

The bit locator system monitors exact location of the drill bit with special locator sensors free from magnetic disturbance. The system gives the operator in real time such information as direction and inclination of drill bit, tool face orientation, and deviation from preplanned alignment. Figure 6 shows the schematic representation of the flexible borehole alignment.

![Figure 6. The schematic representation of the flexible borehole alignment which can be made by navigational drilling system [3]](image)

Below are the steps that take place in this method:
1. Operator starts drilling while monitoring bit location (with the tool face oriented to the goal).
2. Withdraw the inner steel rod.
3. Install a grout pipe.
4. Withdraw the outer pipe.

Figure 8 clarifies the steps that take place in the navigational drilling system.
4.1.4. Advantages and Main Points
1. Drills of this method are the most flexible among injection and grouting tools (it can carve with various radiuses, e.g. 20)
2. Drills are able to drill at a long-distance.
3. Drills can solve the underground obstacle problems with special bit locator system.
4. Enable ground improvement and soil remediation under or behind existing structures without affecting operation or damaging underground structures
5. This method can be used as a remedial method for structures.

4.1.5. Vacuum Grouting Injection
It is worth pointing out that pressure injection may be less successful when the pressures needed to dispel gases and liquids from the voids are so high as to risk disrupting the structure. For instance, this may happen when the voids consist of many fine interstices and are not always interconnecting (which may result in the need for a very large number of injection points), when complete filling is very difficult to achieve, or when it is difficult to confine the grout to the area to be injected.

The third part of single phase of jet grouting is Vacuum Grouting Injection. In this technique a partial vacuum is first established in a portion of the structure (or the whole of the structure if it is small enough), drawing off gases and liquids from the voids and interstices. This vacuum holds the structure together, rather than exerting any potentially disruptive forces as in pressure injection. After achieving a stable vacuum, the injection liquid is introduced either through injection pipes set at appropriate intervals and depths or, over the surfaces and into the structure through cracks, fissures and porous areas [6]. In Figure 7 the proceeding of Vacuum Grouting Injection and its instrument are displayed.

4.1.6. Advantages and Main Points
1. It can be used for filling small, essentially air-tight voids through a single hole, where difficult access complicates the provision of vent holes (like Dam's concrete) [7].
2. It has been employed for filling small voids under steel liners, as well as defects in the original grouting of post-tensioning ducts. In other words, it can
fill off closely spaced fine defects in concrete or masonry.
3. It can be seen clearly which part of the voids are filled by grouting (in part 2). Whenever the plastic cover saturates the entire area under plastic sheeting, all air will be sucked in including any void space as well (Figure 8).
4. This method can be used as a remedial method for structures.
5. The mixing machine used is mobile and can be easily relocated to the next soil mixing location at/on site.
6. This method can be used as a remedial method for structures.

![Figure 8. The plastic cover is used for all air to be sucked into any void spaces as well [12]](image)

5. DUAL PHASE SYSTEM

5.1. Dry jet Mixing System (DJM)
The second part of jet grouting systems is dual phase system. The DJM is a highly effective ground treatment system used to improve the load performance characteristics of soft clays, peats and other weak soils. The concept of using dry binder for deep soil mixing was first presented in Scandinavia in 1967 by Mr. Kjeld Paus from Sweden. A period of thirty years has passed since then but the technique has been evolved considerably. The method is based on injecting dry binder carried by compressed air into soil [8]. The DJM uses mixing blades to mix dry reagents, such as cement or lime, with in-situ soils for remediation.

In this method, the process employs the effects of both hydration and the bonding of soil particles to increase the shear strength and reduce the compressibility of the soil mass [4].

**Advantages and Main Points**
1. The use of air instead of water to transport the binder in pipes and hoses is a big advantage where the temperature drops below the freezing point many months of the year or in the high ground conditions [8].
2. Additives to cement and lime can be used with particles of sizes less than 5 mm [3].
3. DJM does not need water or slurry preparation. Operation without water keeps the site clean.
4. Little dust is introduced into air and the operation is safe with minimum noise
and minimum ground vibration.
5. The mobility and automatic monitoring system (the latest one) of mixing machine records help in getting high quality performance and saves on labor.
6. Soil mix column with diameters of 600mm to 1000mm can be constructed to controlled height and depth. The amounts of binder agents commonly used are 80-100 kg/m³ in soft clay and 150-200 kg/m³ in peat [4].

5.2. Triple Phase
5.2.1. Jumbo Eco Pile System (JEP)
The next part of JEP is triple phase system. It can be said that the JEP is the most popular method in this system. Most frequently, the applications of JEP, which is also named soilcrete-jet grouting, are: underpinning, tunnel protection, foundation restoration and modification, shaft supports, deep foundation, earth pressure relief, panel walls, vault slabs, column walls, sealing cover, dam sealing, joint sealing, sealing slabs, and groundwater exits. Figure 9 shows the steps of this method.

![Figure 9. The steps of jumbo ECO pile system (JEP)](image)

This method is also recognized as cement soil stabilization. With the aid of high pressure cutting jets of water or cement suspensions having a nozzle exit velocity $\langle 100 \text{ m Sec}^{-1} \rangle$; eventually the air shrouding the soil around the borehole is eroded [4]. It has been suggested that when the cohesion of the ground exceeds 50 kNm$^{2}$, a separate study is required. A separate study is also required for the sand and gravel layer [3].

Advantages and Main Points
1. Large diameter column improvement from double jet monitors.
2. Double jet shortens construction time.
3. Tow jet crosses and cuts soil to smaller size, thus producing high quality product.
4. In comparison with the conventional method, this method is more cost-effective and time saving alternative.
5. The compressive strength of JEP system is from 2-25Mpa.
5.3. Super Jet Grouting

5.3.1. Ras-Jet System

Ras-Jet System is used in super jet grouting method. This method is the same as the Ras-Columns which have been elaborated in the first part of this paper. However, in Ras-Jet System, while the mixing blades are rotating; the same slurry is jetted simultaneously. With this system, the homogenous soil-cement column mass of a large diameter will be installed underground.

Grout slurry, air, and drilling fluid are pumped through separate chambers in the drill string. Upon reaching the designed drill depth, jet grouting is initiated with high velocity, coaxial air and grout slurry to erode and mix with the soil, while the pumping of drilling fluid is ceased. This system uses opposing nozzles and highly sophisticated jetting monitor specifically designed for focus of injection media [4]. Figure 10 shows the soil cement columns which are produced by Ras-Jet system.

Advantages and Main Points [3]
1. Applicable to a wide range of soils, providing excellent quality improvement.
2. Large diameter column improvement of diameter (1.6 - 2.0m, excluding the jet grouting part).
3. Uniform mixing and homogenous product quality due to counter-rotation mechanism.
5. Super high pressure jet of slurry cuts and breaks in-situ soil and produces homogenously improved zone around the mechanically mixed core.

6. CONCLUSIONS

1. The DMMs which are applied to stabilize and improve soils are spreading increasingly and have been accepted worldwide as a soil improvement method. DMMs are based on mixing binders such as cement, and/or lime and other additives with soil grains, using rotating mixing tools or jetting, simultaneously.
2. These methods have been suggested and applied for soil and rock stabilizing, slope stability, liquefaction mitigation, vibration reduction (along the railway),
roads and railroads bridge foundations, embankments, construction of excavation support systems or protection of structure close to excavation sites, solidification and stabilization of contaminated soils, and remedial grout injection of building.

3. Based on conditions such as the types of soil and rock layers, time table of project, location, importance of project, and the economic situation, the use of multiple-auger or single auger deep mixing methods, jet grouting methods, or a combination of several methods may be required.

REFERENCES
CD07
Special Concretes
THE STUDY OF VELOCITY AND FREQUENCY OF PASSING WATER THROUGH A MAGNETIC FIELD AND ITS EFFECT ON THE COMPRESSIVE AND TENSILE STRENGTH OF CONCRETE

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ABSTRACT
Concrete is a material which can have a range of different strengths with using different ratios of its composites. In this regard, every factor that can help us reach better mechanical properties and other characteristics of concrete is worth studying. In our study, regarding above information, we study the effects of water properties (considering magnetizing water) of concrete on some of the mechanical features of concrete containing microsilica such as compressive and tensile strength at different curing ages. The water in concrete has been passed through magnetic field with velocities such as Q, Q/2, Q/3, Q/6 and with 1, 3 and 6 passing times. The results indicate that increasing the number of times water is passed (from 1 to 6), improves the compressive and tensile strength of concrete, and decreasing the velocity of passing water through magnetic field in one cycle (from Q to Q/6) has a similar result. The improvement of decreasing the velocity in one cycle is more significant than the improvement caused by passing water through a number of times, and the compressive and tensile strength will be improved substantially. The compressive and tensile strength of concrete will be increased up to 15%. This method is considered very economical and does not need special equipments in industry.

Keywords: microsilica, magnetic water, superplasticizer, compressive

1. INTRODUCTION
Henrico Anton Lorenz (1920) introduced the effect of the magnetic field on water that got the Nobel Prize. He found that, under the effect of a magnetic field, polar molecules are arranged and separated, therefore, water becomes lighter. When the molecules pass through a magnetic field, a change in their arrangements appears. Since each water molecule is like a little magnet, as shown in Figure 1, by passing through the magnetic field, they arranged in an end - to - end way; This is called polarity. Water molecules are not separated; they are attached to each other as a group by a hydrogen bond. The lesser the number of molecules are in this group of molecules, their activity will be more. The magnetic field helps to this process (Figure 2).
The general principle of magnetic technology is based on the interactions between a magnetic field and a moving electrical charge, in form of an ion in this system. When the ions pass through a magnetic field, some force is given to each single ion. In case, the orientation and the charge of the ions are against each other, it causes a fluctuation in the ions which, as a result, leads to the formation of sediment. The magnetic fields lead to the induction of electrical charge through positive and negative ions. As a consequence, ions of different charges repel each other instead of attracting each other. Therefore, a negative and a positive ion of the same charge that should form sediment, cannot approach enough to each other. Magnetic processing of water system is categorized in two and three, forms of installation and operation, respectively. Magnetic system may be installed inside or outside the flow. The inner systems are those which all or part of them is on the way of the flow. The outer systems are completely out of the water flow, thus they can be installed on the pipe (Figure 3) [1].

In terms of the kind of the operation these systems are categorized in the following way:
1. Magnetic: Mostly a permanent magnet.
2. Electromagnetic: In which the magnetic field is provided by the electromagnetic.
3. Electrostatic: In which the electrical field is forced into the water flow, which leads to the production of a magnetic field and, as a result, it attracts or repels the ions. The electrostatic units are always inside, whereas the other two kinds can be either inside or outside (outer) [1].

The degree of magnetization of water depends on three factors:
- The amount of the liquid in contact with the magnet.
- The power of the used magnet.
- The period of time that the liquid is in contact with the magnet.

During the recent years, there have been few researches carried out about the effect of magnetized water on the properties of normal concrete. In these researches, normal water was passed through a magnetic field in a frequency and with a specified velocity, and then this water was used in the mix designing. The result of these researches was an increase in the strength and other mechanical properties of concrete.
If water passes through a magnetic field during one or more frequencies or with different velocities, the behavior of the strength properties of the concrete containing Microsilica is studied in this research.

![Figure 3. Different kinds of magnetic systems in terms of the place of installation](image)

2. EXPERIMENTAL PROCEDURES

2.1. Properties of the Used Materials

2.1.1. Cement

The cement used in making concrete specimens of normal Portland cement type II of KHASH cement factory is based on the ASTM standard [3]. The unite weight obtained is 3.157 gr/cm³. Tables 1 and 2 represent physical properties and results of the chemical analysis of the specimen, respectively.

2.1.2. Aggregates

Natural coarse (Gravel) and fine aggregates (Sand) are used to make the specimens. Different properties of the aggregates used, such as the unit weight (in terms of dry, saturated, and apparent), water absorption and their grading are obtained (Tables 3 and 4). Water absorption of the aggregates, represents the amount of the moisture of the aggregates in a saturated-dry surface (SSD).

| Table 1: Physical and mechanical properties of Portland cement type II [3] |
|----------------------------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|
| Compressive strength (Kg/cm²)   | Setting time    | Hydration heat  |                  |
| 28 days                         | 7 days          | 3 days          | final            | initial         | 7 days          | 28 days         |
| > 380                           | > 270           | > 170           | < 200            | > 70            | < 70            | < 80            |

| Table 2: Results of the chemical analysis of Portland cement type II [3] |
|-----------------|----------------|----------------|----------------|----------------|----------------|----------------|
| SO₃             | MgO            | CaO            | Fe₂O₃          | Al₂O₃          | SiO₂           | Chemical compound |
| <2.0            | <2.3           | 63.2-64.8      | 3.6-4.3        | 4.7-5.4        | 21.0-22.2      | percentage     |
| Na₂O+0.658K₂O   | F.Cao          | L.O.1          | Lns.Res        | Cl             |                 | Chemical compound |
| <1.0            | <1.3           | <1.0           | <0.65          | <0.05          |                 | percentage     |

Grading represents the amount of distribution and presence of different sizes of aggregates in a sand or gravel specimen. The size of an aggregate is the average diameter that can be considered for the aggregate. Therefore, dividing an aggregate
into grains of the same size, is the definition of grading [4]. This grading of fine aggregate (sand) and coarse aggregate (gravel), as shown in diagrams 1 and 2, is experimentally obtained.

<table>
<thead>
<tr>
<th>Aggregate</th>
<th>The actual dry unit weight</th>
<th>The actual saturated unit weight</th>
<th>The apparent unit weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravel</td>
<td>2646</td>
<td>2673</td>
<td>2718</td>
</tr>
<tr>
<td>Sand</td>
<td>2556</td>
<td>2576</td>
<td>2607</td>
</tr>
</tbody>
</table>

Table 4: The 24-hour water absorption aggregates based on the results of the test

<table>
<thead>
<tr>
<th>Percentage of water absorption</th>
<th>Aggregate</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.005</td>
<td>Gravel</td>
</tr>
<tr>
<td>0.76</td>
<td>Sand</td>
</tr>
</tbody>
</table>

2.1.3. Water

Drinking water is used in carrying out tests and making specimens. To magnetize water it was first poured in to a system (at the end of which a pipe was attached from outside and in some parts of the pipe the AQUA apparatus (Figure 5) was installed, and there were two valves before and after the apparatus (Figure 4)), and after the water went out of the vessel and passed through a magnetic field, in case the valve installed at the end of the AQUA was turned on, it would pass through the pipe into the vessel 2 and it would be used in making specimens, immediately. If the objective was to repass the water through the magnetic field, the water in vessel 2 would be re-poured into vessel 1, and this would be done as many times as required. If the objective was passing the water with a lower velocity less than that of the basis, the valve before the AQUA was turned on so that the water could pass through the magnetic field with the desired discharge. Flow meters were used to control of the velocity of the water.

2.1.4. Admixtures

In this study silica fume (S.F) was used with the average grain of 0.2 µm and an actual specific weight of about 2.2 gr/cm³. To study the extra effect of silica fume in concrete, specimens of 10% and 20% of silica fume were made as substitutes for cement and different results of the compressive and tensile strengths were obtained from magnetized water. Because of an increase in the viscosity and cohesiveness of the matrix and a decrease in the workability of concrete due to the use of silica fume, superplasticizer was used.

2.1.5. Superplasticizer

Superplasticizers are used in naphthalene formaldehyde sulphonate tests. This material is used as liquid in the form of solution in water. It is dark brown and has a specific weight of 1.2gr/cm³.
2.2. Properties of the Mix Designings
Properties of parts of the concrete mixture made with magnetized water (during different frequency or different velocities in a frequency) are calculated with absolute volume method and are shown in Table 5. CO₁ to CO₄ represent specimens made with normal water and are considered as the observer concrete. CN and CV represent specimens made with magnetized water in different frequencies and specimens made with magnetized water in different velocities, respectively.

Notes:
1. Q is the base discharge and is 105.8 cm³/s.
2. In all Tables, the quantity of Q is cm³/s.
3. In represents the number of magnetization of water cycles.
4. In this study, concrete specimens were made in which density of the cement as 400 kg/m³, the water to cement were 0.5 and 0.4 ratios and also 10% and 20% of Microsilica.
5. According to the fact that silica fume is a cement material, cement materials (B=C+S.F.) are used instead of cement, and water to cement material ratios
(W/B) instead of water to cement (W/C) in the calculations and Table of the mix designings.

7. The maximum amount of the superplasticizer is limited to 2% of the weight of the cement materials.

8. The amount of the air in these specimens is 2% of the volume of the concrete.

9. The average amount of slump is 60 mm.

10. The maximum size of the aggregates is 19 mm.

2.3. Details of the Specimens and the Preserving Method

Specimens were made to carry out the concrete compressive and tensile strength tests, which will be discussed later.

**Table 5: The amounts of the mix designing (Kg/m³)**

<table>
<thead>
<tr>
<th>Calculated specific weight</th>
<th>Amount of the basic passing discharge (d)</th>
<th>Number of water magnetic frequency (n)</th>
<th>Superplasticizer</th>
<th>B= C/S.F</th>
<th>Microsilica (S.F)</th>
<th>Cement (C)</th>
<th>Water (W)</th>
<th>Sand (S)</th>
<th>Gravel (G)</th>
<th>Percentage of microsilica</th>
<th>CO1</th>
<th>CO2</th>
<th>CO3</th>
<th>CO4</th>
</tr>
</thead>
<tbody>
<tr>
<td>۵۲٫۰۳۱۱</td>
<td>۵۲٫۰۳۱۱</td>
<td>۱۰٫۱۸</td>
<td>۹٫۲۵</td>
<td>۱۰٫۱۸</td>
<td>۱۰٫۱۸</td>
<td>۱۰٫۱۸</td>
<td>۱۰٫۱۸</td>
<td>۱۰٫۱۸</td>
<td>۱۰٫۱۸</td>
<td>۱۰٫۱۸</td>
<td>CO1</td>
<td>CO2</td>
<td>CO3</td>
<td>CO4</td>
</tr>
<tr>
<td>۵۲٫۰۳۱۱</td>
<td>۵۲٫۰۳۱۱</td>
<td>۱۰٫۱۸</td>
<td>۹٫۲۵</td>
<td>۱۰٫۱۸</td>
<td>۱۰٫۱۸</td>
<td>۱۰٫۱۸</td>
<td>۱۰٫۱۸</td>
<td>۱۰٫۱۸</td>
<td>۱۰٫۱۸</td>
<td>۱۰٫۱۸</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>۵۲٫۰۳۱۱</td>
<td>۵۲٫۰۳۱۱</td>
<td>۱۰٫۱۸</td>
<td>۹٫۲۵</td>
<td>۱۰٫۱۸</td>
<td>۱۰٫۱۸</td>
<td>۱۰٫۱۸</td>
<td>۱۰٫۱۸</td>
<td>۱۰٫۱۸</td>
<td>۱۰٫۱۸</td>
<td>۱۰٫۱۸</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Table 6: comparison of the specimens W/B=0.4 , S.F=20%**

<table>
<thead>
<tr>
<th>specimen</th>
<th>CO1</th>
<th>CN1</th>
<th>CN5</th>
<th>CN9</th>
<th>CV1</th>
<th>CV5</th>
<th>CV9</th>
</tr>
</thead>
<tbody>
<tr>
<td>n</td>
<td>۰</td>
<td>۰</td>
<td>۰</td>
<td>۰</td>
<td>۰</td>
<td>۰</td>
<td>۰</td>
</tr>
<tr>
<td>Q</td>
<td>۱۰٫۱۸</td>
<td>۱۰٫۱۸</td>
<td>۱۰٫۱۸</td>
<td>۱۰٫۱۸</td>
<td>۱۰٫۱۸</td>
<td>۱۰٫۱۸</td>
<td>۱۰٫۱۸</td>
</tr>
</tbody>
</table>

**Table 7: Comparison of the specimens W/B=0.4 , S.F=10%**

<table>
<thead>
<tr>
<th>specimen</th>
<th>CO2</th>
<th>CN2</th>
<th>CN6</th>
<th>CN10</th>
<th>CV2</th>
<th>CV6</th>
<th>CV10</th>
</tr>
</thead>
<tbody>
<tr>
<td>n</td>
<td>۰</td>
<td>۰</td>
<td>۰</td>
<td>۰</td>
<td>۰</td>
<td>۰</td>
<td>۰</td>
</tr>
<tr>
<td>Q</td>
<td>۱۰٫۱۸</td>
<td>۱۰٫۱۸</td>
<td>۱۰٫۱۸</td>
<td>۱۰٫۱۸</td>
<td>۱۰٫۱۸</td>
<td>۱۰٫۱۸</td>
<td>۱۰٫۱۸</td>
</tr>
</tbody>
</table>

**Table 8. comparison of the specimens W/B=0.5 , S.F=20%**

<table>
<thead>
<tr>
<th>specimen</th>
<th>CO3</th>
<th>CN3</th>
<th>CN7</th>
<th>CN11</th>
<th>CV3</th>
<th>CV7</th>
<th>CV11</th>
</tr>
</thead>
<tbody>
<tr>
<td>n</td>
<td>۰</td>
<td>۰</td>
<td>۰</td>
<td>۰</td>
<td>۰</td>
<td>۰</td>
<td>۰</td>
</tr>
<tr>
<td>Q</td>
<td>۱۰٫۱۸</td>
<td>۱۰٫۱۸</td>
<td>۱۰٫۱۸</td>
<td>۱۰٫۱۸</td>
<td>۱۰٫۱۸</td>
<td>۱۰٫۱۸</td>
<td>۱۰٫۱۸</td>
</tr>
</tbody>
</table>
Table 9. Comparison of the specimens W/B=0.5, S.F=10%

<table>
<thead>
<tr>
<th>specimen</th>
<th>CO4</th>
<th>CN4</th>
<th>CN8</th>
<th>CN12</th>
<th>CV4</th>
<th>CV8</th>
<th>CV12</th>
</tr>
</thead>
<tbody>
<tr>
<td>n</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Q</td>
<td>1.05,18</td>
<td>1.05,18</td>
<td>1.05,18</td>
<td>1.05,18</td>
<td>0.65</td>
<td>0.67</td>
<td>17.53</td>
</tr>
</tbody>
</table>

A) Compressive strength test
Standard 10*10*10 cm moulds were used in making compressive specimens. To specify the compressive strength three specimens were made to be tested at each age, meaning that for each design, three cubic specimens for the compressive strength test at the age of 7 days, three cubic specimens for the compressive strength test at the age of 14 days and three cubic specimens to be tested at the age of 28 were made and the average results of the test of the three specimens were placed into the previous results. On the whole 252 cubic 10*10*10 cm specimens were made to specify the strength terms of 7, 14 and 28 days. To convert the compressive strength resulted from the cubic specimens to the standard cylinder, the proposed factors of Iranians Concrete Regulations were used. The strengths obtained, are the average of the strengths of the three concrete specimens made in the laboratory, and are shown in the table. [5]

B) Tensile strength test
15*30 cylindrical specimens and the Brazilian test method were used to achieve the tensile strength. Three models were made for each specimen and the average of the result are shown in Tables and Figures. All the tensile specimens of the age of 7 and 28 days were tested. Totally, 168 cylindrical specimens (15*30) were made of the age mentioned.[5]

3. RESULTS
Two subjects are studied in this part of the research. The first section compares the effect of two techniques of magnetizing water (passing water through the magnetic field in different frequencies or passing it through this field with different velocities) on the compressive and tensile strength. The results of the two techniques are studied in the second part.

3.1. Compressive and Tensile Strength
The results are shown in Table 10.

3.2. Results Discussing
In this part, due to the variety of the existing designs and the results obtained in the previous parts, examining and comparing the concretes made with the maximum compressive and tensile strengths (the designs with magnetized water in six frequencies or magnetized water with water passing through a magnetic field as much as 1/6 of the base passing discharge) with the observer concrete are studied.
Table 10. Results of average compressive and tensile strength of specimens of different ages (MPa)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Average 7-days tensile strength</th>
<th>Average 28-days compressive strength</th>
<th>Average 14-days compressive strength</th>
<th>Average 7-days compressive strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>CO1</td>
<td>7.12</td>
<td>1.24</td>
<td>2.33</td>
<td>0.00</td>
</tr>
<tr>
<td>CO2</td>
<td>7.24</td>
<td>1.32</td>
<td>2.44</td>
<td>0.00</td>
</tr>
<tr>
<td>CO3</td>
<td>7.34</td>
<td>1.42</td>
<td>2.55</td>
<td>0.00</td>
</tr>
<tr>
<td>CO4</td>
<td>7.44</td>
<td>1.52</td>
<td>2.66</td>
<td>0.00</td>
</tr>
<tr>
<td>CO5</td>
<td>7.54</td>
<td>1.62</td>
<td>2.77</td>
<td>0.00</td>
</tr>
<tr>
<td>CO6</td>
<td>7.64</td>
<td>1.72</td>
<td>2.88</td>
<td>0.00</td>
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<tr>
<td>CO7</td>
<td>7.74</td>
<td>1.82</td>
<td>2.99</td>
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<tr>
<td>CO8</td>
<td>7.84</td>
<td>1.92</td>
<td>3.10</td>
<td>0.00</td>
</tr>
<tr>
<td>CO9</td>
<td>7.94</td>
<td>2.02</td>
<td>3.21</td>
<td>0.00</td>
</tr>
</tbody>
</table>

Diagram 3. Effect of magnetizing water on the compressive strength of concrete of different ages

Diagram 4. Effect of magnetizing water on the tensile strength of concrete of different ages
3.2.1. Concrete with 0.4 Water to Cement Ratio and Containing 20% of Microsilica

The results of the comparison between this mix designing in the concrete with normal water and with magnetized water (in six frequencies and with a passing velocity of 1/6) are shown in diagrams 3 and 4. As illustrated in these figures, the magnetization of water causes an increase in the compressive and tensile strength. Furthermore, the induction of the magnetic property into water, by passing it through a magnetic field with 1/6 of the base velocity, compared with the number of processing frequencies causes more increase in the strength.

3.2.2. Concrete with 0.4 Water to Cement Ratio and Containing 20% of Microsilica

A comparison between the results of the diagrams 5 and 6 shows that due to magnetizing water, mechanical properties (including the compressive and tensile strength) will improve. For example, by increasing the number of magnetic processing frequencies of water, the 28 days compressive and tensile strengths of the concrete, in comparison with the observer specimen, will increase by 17.3 and 15.1, respectively.

Diagram 5. Effect of magnetizing water on the compressive strength of concrete of different ages

Diagram 6. Effect of magnetizing water on the tensile strength of concrete of different ages

Induction of magnetic property by decreasing the velocity of water passing through the field, causes an increase of 29.6% and 20.8% in these strengths. Therefore, magnetizing water via decreasing its velocity of passing through the magnetic field causes more increase in the compressive and tensile strengths of the concrete.

3.2.3. Concrete with 0.5 Water to Cement Ratio and Containing 20% of Microsilica

Results obtained from this comparison are shown in diagrams 7 and 8. According to these results, the concrete containing magnetized water, will have its maximum of tensile and compressive strength via decreasing the velocity of water passing through the magnetic field. For example, via this method of magnetizing water, the tensile and compressive strength will, in comparison with the observer concrete, increase by 24.6% and 32.9% respectively, whereas by increasing the number of magnetic processing, these two quantities will, in comparison with the normal
specimens of water, increase by 13.5% and 17.5%, respectively.

Diagram 7. Effect of magnetizing water on the compressive strength of concrete of different ages

Diagram 8. Effect of magnetizing water on the tensile strength of concrete of different ages

4. CONCLUSION
Looking of the fact that, optimized strengths are obtained by magnetized water with six frequencies or 1/6 of the base discharge, the results of examining the laboratory results of this study and the final conclusion of the information there in are as follows:
- Generally, by once magnetizing water, the average changes in the compressive and tensile strength, will in comparison with normal water, be about 9% and 8.5%, respectively whereas the average amount of the increase in the compressive and tensile strength of all ages, by once to three times of magnetizing will, respectively be about 1.9% and 6.1% and with three to six times of magnetizing will be 2.1% and 1.8%. This is the case that magnetizing water via passing in through the magnetic field with the velocity of 1/2 of the discharge, the averages in the compressive and tensile strength, is respectively in comparison with normal water, about 17.6% and 20.4% whereas the average amount of the increase in the compressive and tensile strength from 1/2 of the discharge to 1/3 at all ages in respectively 6.5% and 5.6% and from 1/3 to 1/6 of the discharge is about 3.2% and 0.9%.

According to what has been mentioned:
1. Increasing the number of magnetizing water frequencies (from 1 to 6) and also decreasing the velocity of water passing through the magnetic field (from Q to Q/6), the average increase in the strength will face a decline.
2. six times of magnetizing water will change the average compressive and tensile strength, at all ages, to about 13% and 16.6% in comparison with the concrete made with normal water, where as by water passing with a velocity of 1/6 of the base passing discharge the average compressive and tensile strengths, at all ages, will respectively change about 27.3% and 25.1% in comparison with the
concrete made with normal water. Therefore,

\[
\begin{array}{c|c|c|c|c}
\text{Tensile Strength (MPa)} & \text{CO4} & \text{CN12} & \text{CV12} \\
\hline
0 & 1.25 & 1.64 & 1.92 \\
5 & 2.44 & 2.89 & 3.19 \\
10 & 3.50 & 3.50 & 3.50 \\
15 & 4.50 & 4.50 & 4.50 \\
20 & 5.50 & 5.50 & 5.50 \\
25 & 6.50 & 6.50 & 6.50 \\
30 & 7.50 & 7.50 & 7.50 \\
35 & 8.50 & 8.50 & 8.50 \\
\end{array}
\]

\[
\begin{array}{c|c|c|c|c}
\text{Compressive Strength (MPa)} & \text{CO4} & \text{CN12} & \text{CV12} \\
\hline
0 & 20 & 24.8 & 34.1 \\
5 & 26.7 & 38.9 & 46.2 \\
10 & 32.5 & 41.6 & 50.8 \\
15 & 38.3 & 49.4 & 58.6 \\
20 & 44.1 & 58.7 & 67.9 \\
25 & 50.0 & 67.8 & 77.1 \\
30 & 55.9 & 77.0 & 86.3 \\
35 & 61.7 & 86.4 & 95.6 \\
\end{array}
\]

Diagram 9. Effect of magnetizing water on the compressive strength of concrete of different ages

Diagram 10. Effect of magnetizing water on the tensile strength of concrete of different ages

A) Magnetizing water with decreasing the velocity of water passing through the magnetic field, in comparison with the other technique of magnetizing water (by increasing the number of magnetic processing frequencies) is the most efficient technique, in a way that former technique causes an increase, in comparison with the later technique, of 14.3% and 8.5% in the compressive and tensile strength.

B) according to the results above, in the subject of tensile strength, water passing through the magnetic field with the velocity of 1/6 of the base discharge is not so cost-saving and caused a decline in the strength.

- The concrete containing 10% of Microsilica and a 0.5 water to cement ratio, shows the best response to magnetizing and the compressive and tensile strength, due to this change in the property of water (during six frequencies), the amount of the increase in the compressive strength will be about 16.9% while the concrete containing 10% of Microsilica and a 0.4 water to cement ratio has the maximum increase (about 22.5%), due to six times of increasing the number of the magnetic frequency, in the tensile strength. In case of magnetizing water (with the velocity of 1/6 of the discharge), the concrete containing 10% of Microsilica and a 0.5 water to cement ratio, shows the best response to the magnetizing in the compressive and tensile strength and due to this change in the property of water (during water passing will 1/6 of the base passing discharge), the amount of the increase in the compressive and tensile strength will be 26% and 37.9% respectively. Therefore, with a decrease in the percentage of Microsilica and an increase in the water to cement ratio, the concrete will have the maximum increase in the mechanical properties.
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4. A. Same, “Quality and mix designing of concrete, Technical University of Isfahan, 2006”, 44-54;
MIX DESIGN OF STRUCTURAL SELF-COMPACTING CONCRETE USING VOID-BULK DENSITY METHOD

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ABSTRACT
Self Compacting Concrete (SCC) was firstly developed in Japan in 1987. (Alternative: Pioneering works on Self Compacting Concrete returns to 1980s in Japan) In recent years, much research has been conducted in other to achieve a reasonable and also suitable mix design method for controlling the compaction experiment and determination of the compliance particular trait of SCC. But, just a few researches have been done in to propose a mix design method that can have both of highly fluid state and good viscosity, simultaneously. SCC is a special kind of concrete that can flow through and fill the gaps of reinforcement and corners of molds without any need for vibration and compaction during the placement process. In this paper Void-Bulk Density mix design method for structural SCC is investigated. In this method, firstly, the relationship between the void volume (or density of combined aggregates) and coarse-to-total aggregate volume ratio is established by packing different amounts of coarse and fine aggregates following ASTM C 29/C 29M, using the void volume of the dry binary aggregate (fine and coarse), is determined. Then, based on the optimum ratio that results from minimum void of aggregate and minimum bulk density, mix design is accomplished and finally in order to increase the flowability of the concrete have been added, some excess paste via reducing ratio of volume aggregate in unit volume concrete. Obtained results of the experiments on fresh concrete (Slump flow, L-box, V-funnel) and hardened concrete (compressive strength, tensile strength, elastic moduli and durability) show that this method is appropriate for SCC. In this study nine different SCC mixtures having the volume of paste and the ratio between sand and gravel as variables were compared with eight different mixtures of conventionally vibrated concrete (CVC).

Keywords: Self-compacting concrete, compressive Strength, Durability, elastic moduli, Mix design

1. INTRODUCTION
The development of Self-Compacting Concrete (SCC), also referred to as “Self-Consolidating Concrete” and “High-Performance Concrete”, has recently been one of the most important developments in the building industry. It is a kind of concrete that can flow through and fill gaps of reinforcement and corners of
Self-consolidating concrete (SCC) is a highly flowable, yet stable, concrete that can spread readily into place, fill the formwork, and encapsulate the reinforcement, if present, without any mechanical consolidation and without undergoing any significant separation of material constituents. The introduction of the modern SCC is associated with the drive towards better quality of concrete pursued in Japan in late 1980s, where the lack of uniform and complete compaction had been identified as the primary factor responsible for poor performance of concrete structures. SCC has many advantages over conventional concrete such as:

- Eliminating the need for vibration;
- Decreasing the construction time and labor cost;
2. MIXTURE DESIGN PROCEDURES FOR SCC

Several design procedures based on scientific theories or empirical experiences have been proposed for SCC [7-8]. In general, these procedures fall into the following two categories: 1) combination of high-range water-reducing admixture and high content of mineral powders, and 2) combination of high-range water-reducing admixture and viscosity-modifying admixture (VMA) with or without defoaming agent. Figure 2 illustrates the general principles for the design of SCC, as considered from the excess paste theory. The conventional concrete design method begins with the determination of the amounts of water and cement, and ends with the calculation of the amount of aggregates. Because aggregates are much less expensive and more stable than cement pastes, a quality concrete should contain as much aggregate and less cement paste as possible. Thus, the most reasonable approach to determine the amounts of cement pastes for the concrete should be based on the characteristics of the aggregates used and of the concrete designed. In this paper, a procedure has been developed to design SCC using a combination of the least void volume for a binary aggregate mixture, excess paste theory [9-10] and ACI 211.2,“Standard Practice for Selecting Proportions for Structural Concrete [11].

Figure 2(a) shows compacted aggregate particles. In order to obtain a concrete mixture with proper workability, it is necessary to have not only sufficient amount of cement paste to fill the voids among aggregate particles, but also enough paste to form a thin layer of coating on the surface of aggregates to overcome some frictions between aggregate particles, as shown in Figure 2(b). Without a film of cement paste around aggregates as a lubricant, the movement between aggregates would be difficult. To further increase the workability of the concrete mixture to become a self-consolidating concrete, it is necessary to increase the volume of excess paste or the distance between aggregate particles, as shown in Figure 2(c). The required volume of excess paste is dependent on gradation, shape, and surface texture of the aggregates used, and can be determined through laboratory experiments for concrete mixtures with desired properties.

To determine the volume of filled paste and excess paste, the void volume of the dry binary aggregate (fine and coarse) mixtures should be determined first. The relationship between void volume or density of combined aggregates and coarse-to-fine aggregate volume ratio can be established by packing different amounts of coarse and fine aggregates following ASTM C 29/C 29M,19 as shown in Figure 3. It can be seen from Figure 3 that the lowest void volume for the combined coarse and fine aggregates used in this project is around 280 L/m3 when the coarse-to-fine aggregate volume ratio is 0.4.

The target compressive strength $f'_c$ of the designed SLC was 28 MPa (4000 psi) at 28 days using ASTM Type I Portland cement. Because no statistical strength data are available for this concrete, ACI 318 “Building Code Requirements for Structural Concrete” requires that an average strength of the tested concrete at 28 days be $f'_c + 8$ MPa (1200 psi), or 36 MPa (5200 psi). ACI 211.2 provides guidelines on relationships between compressive strength and cement content, and relationship between compressive strength and water cement ratio ($w/c$). Based on
the strength requirement and ACI 211.2, a cement content of 420 kg/m³ and a w/c of 0.48 were used in this study. The volume of excess paste was determined by experiments.

Figure 2. Scheme of compacted aggregate and concrete mixtures.

Figure 3. Effect of coarse-to-total aggregate volume ratio on bulk density and void volume of binary aggregate mixture consisting of coarse lightweight aggregate and fine natural siliceous sand.
Different volumes of combined aggregates were replaced by cement paste with the same property. It was found that a replacement of 20% aggregate (by volume) by excess paste would give the concrete the required flowability and segregation resistance. The workability of the concrete mixture was adjusted by using a high-range water-reducing admixture. During the mixture proportioning, the cement content was fixed at 420 kg/m³; the rest of the paste was made from powders, such as Limestone and Silica fume.

Bulk Density (“Unit Weight”) and Voids in Aggregate

Bulk Density-calculate the bulk density for the rodding, jigging, or shoveling procedure as follows:

\[ M = \frac{(G - T)}{V} \]

Or

\[ M = \frac{(G - T) \times F}{V} \]

Where:

- \( M \) = bulk density of the aggregate, \( \text{kg/m}^3 \),
- \( G \) = mass of the aggregate plus the measure, kg,
- \( T \) = mass of the measure, kg,
- \( V \) = volume of the measure, \( \text{m}^3 \), and
- \( F \) = factor for measure, \( \text{m}^3 \).

Figure 4. Cylindrical metal measure with Tamping Rod and piece of plate glass

Void Content-Calculate the void content in the aggregate using the bulk density determined by either the rodding, jigging, or shoveling procedure, as follows:

\[ \%\text{Void} = \frac{100 \times (S \times W) - M}{S \times W} \]

Where:

- \( M \) = bulk density of the aggregate, \( \text{kg/m}^3 \),
- \( S \) = bulk specific gravity (dry basis) as determined in accordance with Test Method C 127 or Test Method C 128, and
- \( W \) = density of water, 998 \( \text{kg/m}^3 \).
Relative Density (Specific Gravity) (OD)—Calculate the relative density (specific gravity) on the basis of oven-dry aggregate as follows:

\[ \text{Relative density (specific gravity) (OD)} = \frac{A}{(B - C)} \]

Where:
- \(A\) = mass of oven-dry test sample in air, g,
- \(B\) = mass of saturated-surface-dry test sample in air, g, and
- \(C\) = apparent mass of saturated test sample in water, g.

Powder Volume—Calculate the powder volume as follows:

\[ V_p = V_w + V_c - V_{\text{EXP}} - \text{Void} \]

Where:
- \(V_p\) = Powder Volume (lit)
- \(V_w\) = Water Volume (lit)
- \(V_c\) = Cement Volume (lit)
- \(V_{\text{EXP}}\) = Excess Paste Volume (lit)
- \(\text{Void}\) = Void Volume (lit)

3. MIX DESIGN OF STRUCTURAL SCC USING VOID-BULK DENSITY METHOD

Nine batches of concrete were designed using the same mixture proportions, as shown in Table 1.

Concrete mixtures were mixed in a high-speed shear mixer. The properties of freshly mixed concretes were determined as described in the following. For each batch, two 100x200mm cylinders were cast for splitting strength testing and six 100x100x100mm cube were cast for compressive and elastic moduli testing. The specimens were cast in one layer without any compaction or vibration. After casting, all the molded specimens were taken to a fog room at 23±2°C. The curing and testing of these specimens for measurement of different properties are described in the following.

Table 1: Mixture proportions of SCC

<table>
<thead>
<tr>
<th>Mixture No.</th>
<th>Coarse Aggregate</th>
<th>Sand</th>
<th>Water</th>
<th>Cement</th>
<th>Silica Fume</th>
<th>Limestone</th>
<th>sp</th>
</tr>
</thead>
<tbody>
<tr>
<td>SCC1</td>
<td>684</td>
<td>884</td>
<td>191</td>
<td>458</td>
<td>46</td>
<td>149</td>
<td>6.8</td>
</tr>
<tr>
<td>SCC2</td>
<td>686</td>
<td>886</td>
<td>190</td>
<td>459</td>
<td>48</td>
<td>149</td>
<td>7.15</td>
</tr>
<tr>
<td>SCC3</td>
<td>699</td>
<td>907</td>
<td>169</td>
<td>429</td>
<td>36</td>
<td>165</td>
<td>7.01</td>
</tr>
<tr>
<td>SCC4</td>
<td>709</td>
<td>918</td>
<td>165</td>
<td>443</td>
<td>71</td>
<td>83</td>
<td>10.10</td>
</tr>
<tr>
<td>SCC5</td>
<td>711</td>
<td>924</td>
<td>180</td>
<td>444</td>
<td>63</td>
<td>107</td>
<td>9.8</td>
</tr>
<tr>
<td>SCC6</td>
<td>712</td>
<td>925</td>
<td>188</td>
<td>455</td>
<td>134</td>
<td>0</td>
<td>10.40</td>
</tr>
<tr>
<td>SCC7</td>
<td>705</td>
<td>917</td>
<td>186</td>
<td>451</td>
<td>87</td>
<td>61</td>
<td>9.47</td>
</tr>
<tr>
<td>SCC8</td>
<td>683</td>
<td>917</td>
<td>190</td>
<td>397</td>
<td>92</td>
<td>110</td>
<td>9.38</td>
</tr>
<tr>
<td>SCC9</td>
<td>692</td>
<td>900</td>
<td>205</td>
<td>439</td>
<td>116</td>
<td>33</td>
<td>10.10</td>
</tr>
</tbody>
</table>
3.1. Slump Flow Test
The slump flow test measures the horizontal free flow of SCC by using a regular slump cone. It was first developed in Japan for use in assessment of flowability of underwater concrete. This is a simple, rapid test procedure and is suitable for construction site use. The slump cone was filled with concrete mixtures without rodding, and then lifted up vertically. The diameters of spread mixtures in four directions after unconfined lateral spread were measured, and the average of the four measurements was used as the flowability of the concrete mixture. The slump flow of the mixtures was measured at 30, 60, and 90 min after the addition of mixing water to examine how the flowability of SCLC mixtures changed with time. Between measurements, the SCC mixtures were stored in a bucket covered with a damp cloth to avoid moisture loss.

3.2. V-funnel test
A V-funnel, as shown in Figure 5, was used to determine the flowability of the concrete. The funnel was filled with a concrete mixture without rodding or tamping, then the trap door at the bottom was opened to allow concrete to flow out under gravity. The time from opening the trap door until complete discharge of the concrete mixture was recorded as an indication of the flowability of the concrete.

![Figure 5. Schematic illustration of V-funnel](image)

3.3. L-box test
L-box tests assess the filling and passing ability of SCC. Serious lack of stability (segregation) can also be observed easily during the testing. The testing apparatus is shown in Figure 6. The vertical section was filled with a concrete sample without rodding or tamping, and then the sliding door was lifted. The time for concrete mixture to flow to the end of the horizontal section was recorded, and the distance H1 and H2 were measured. The flow time can give an indication of flowability. The ratio H2/H1 is called the blocking ratio. Obvious blocking of coarse aggregates behind the reinforcing bars can be visually observed easily.
Table 2: Properties of freshly mixed SCLC mixtures

<table>
<thead>
<tr>
<th>Mixture no</th>
<th>Slump flow (mm)</th>
<th>Slump flow (s)</th>
<th>V-funnel (s)</th>
<th>L-Box H2/H1,%</th>
<th>L-Box flow t1 (s)</th>
<th>Segregation resistance</th>
<th>Density kg/m³</th>
</tr>
</thead>
<tbody>
<tr>
<td>SCC1</td>
<td>615.0</td>
<td>2.75</td>
<td>7.10</td>
<td>0.900</td>
<td>0.73</td>
<td>GOOD</td>
<td>2416.67</td>
</tr>
<tr>
<td>SCC2</td>
<td>625.0</td>
<td>2.65</td>
<td>6.70</td>
<td>0.910</td>
<td>0.54</td>
<td>GOOD</td>
<td>2425.00</td>
</tr>
<tr>
<td>SCC3</td>
<td>610.0</td>
<td>3.31</td>
<td>12.42</td>
<td>0.785</td>
<td>1.01</td>
<td>GOOD</td>
<td>2412.50</td>
</tr>
<tr>
<td>SCC4</td>
<td>655.0</td>
<td>4.05</td>
<td>9.25</td>
<td>0.875</td>
<td>0.68</td>
<td>GOOD</td>
<td>2400.00</td>
</tr>
<tr>
<td>SCC5</td>
<td>710.0</td>
<td>1.18</td>
<td>6.12</td>
<td>0.895</td>
<td>0.51</td>
<td>GOOD</td>
<td>2439.38</td>
</tr>
<tr>
<td>SCC6</td>
<td>585.0</td>
<td>2.25</td>
<td>6.45</td>
<td>0.805</td>
<td>0.74</td>
<td>GOOD</td>
<td>2424.38</td>
</tr>
<tr>
<td>SCC7</td>
<td>690.0</td>
<td>1.29</td>
<td>6.02</td>
<td>0.835</td>
<td>0.46</td>
<td>GOOD</td>
<td>2416.67</td>
</tr>
<tr>
<td>SCC8</td>
<td>670.0</td>
<td>1.28</td>
<td>5.88</td>
<td>0.885</td>
<td>0.43</td>
<td>GOOD</td>
<td>2397.78</td>
</tr>
<tr>
<td>SCC9</td>
<td>600.0</td>
<td>1.19</td>
<td>5.94</td>
<td>0.825</td>
<td>0.45</td>
<td>GOOD</td>
<td>2400.00</td>
</tr>
</tbody>
</table>

Table 3: Mixture proportions of CVC

<table>
<thead>
<tr>
<th>Mixture no</th>
<th>Coarse aggregate</th>
<th>Sand</th>
<th>Water</th>
<th>Cement</th>
<th>Silicafume</th>
<th>Limestone</th>
<th>sp</th>
</tr>
</thead>
<tbody>
<tr>
<td>CVC1</td>
<td>691</td>
<td>895</td>
<td>176</td>
<td>393</td>
<td>81</td>
<td>180</td>
<td>5.86</td>
</tr>
<tr>
<td>CVC2</td>
<td>1062</td>
<td>806</td>
<td>159</td>
<td>413</td>
<td>36</td>
<td>0</td>
<td>6.88</td>
</tr>
<tr>
<td>CVC3</td>
<td>1042</td>
<td>877</td>
<td>148</td>
<td>385</td>
<td>38</td>
<td>0</td>
<td>7.16</td>
</tr>
<tr>
<td>CVC4</td>
<td>1041</td>
<td>914</td>
<td>138</td>
<td>352</td>
<td>39</td>
<td>0</td>
<td>7.45</td>
</tr>
<tr>
<td>CVC5</td>
<td>1001</td>
<td>849</td>
<td>165</td>
<td>447</td>
<td>23</td>
<td>0</td>
<td>7.9</td>
</tr>
<tr>
<td>CVC6</td>
<td>1038</td>
<td>886</td>
<td>146</td>
<td>426</td>
<td>22</td>
<td>0</td>
<td>8.80</td>
</tr>
<tr>
<td>CVC7</td>
<td>1040</td>
<td>823</td>
<td>167</td>
<td>437</td>
<td>22</td>
<td>0</td>
<td>7.05</td>
</tr>
<tr>
<td>CVC8</td>
<td>1071</td>
<td>884</td>
<td>156</td>
<td>372</td>
<td>20</td>
<td>0</td>
<td>7.16</td>
</tr>
</tbody>
</table>
ine different SCC mixtures, using the volume of paste and the relative amount of sand and gravel as variables, and four different mixtures of CVC were made (Tables 1 and 3). Natural sand and gravel with a high percentage of well rounded particles was used with a maximum grain diameter of 12.5 mm for SCC and 19.5 mm for CVC.

4. RESULT
The compressive strengths of SCC and CVC showed similar values for an identical w/b ratio (Figure 7).

![Figure 7. Compressive Strength at 28 days versus w/b ratio](image)

The average E-modulus of SCC was about 8% lower than that of CVC for an identical compressive strength (Figure 8).

![Figure 8. E-modulus versus Compressive Strength, both at 28 days](image)
There was no significant difference in the relation between compressive and splitting tensile strength of SCC in comparison with CVC although the values for SCC showed a relatively high standard deviation (Figure 9).

Figure 9. Splitting tensile strength versus Compressive Strength at 28 days

5. CONCLUSIONS
Based on the results of this study, the following conclusions can be drawn:
1. SCLC can be designed using a combination of the least void volume for a binary aggregate mixture, excessive paste theory, and ACI standard practice for selecting proportions for structural concrete. Both ground Limestone powder and Silicafume can be used satisfactorily as powder for making up the excessive paste for SCC.
2. Differences in the properties of SCC and CVC used in this study were mainly caused by their relative volume of paste:
3. The E-modulus of SCC was about 8% smaller than that of CVC for an identical compressive strength.
4. At the age of 28 days SCC and CVC displayed the same compressive and splitting tensile strength with a constant w/b ratio.

REFERENCES


ENGINEERING PROPERTIES OF GEOPOLYMER CONCRETE BASED ON ALKALI ACTIVATED NATURAL POZZOLAN

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ABSTRACT
The development of alkali activated binders with superior engineering properties and longer durability have emerged as an alternative to OPC. It is possible to use alkali-activated natural pozzolans to synthesize environmental friendly sound geopolymeric cementitious construction materials. The main benefit of geopolymeric cement is the reduction in environmental impact in harmony with the concept of sustainable development. This paper presents a comprehensive summary of the extensive studies conducted on the results of experimental investigations on the engineering properties of geopolymer concrete using activated Iranian natural pozzolan namely, Taftan. Experimental work was conducted to determine mechanical strength; modulus of elasticity; ultrasonic pulse velocity and shrinkage of different concrete mixtures. Test data are used to identify the effects of salient factors such as water to binder ratios and curing conditions that influence the properties of the geopolymer concrete. The results show that mortar and concrete made with alkali activated natural pozzolan develop moderate to high mechanical strength and modulus of elasticity and shrink much less than ordinary Portland cement (OPC) concrete.

Keywords: alkali-activated binder; geopolymeric cement; natural pozzolan

1. INTRODUCTION
Portland cement concrete is a major construction material worldwide use is said to be second only to water. Unfortunate, the production of Portland cement releases large amount of CO₂ in to the atmosphere. This gas is a major contribution to the greenhouse effect and the global warming of the planet. To reduce greenhouse gas emissions, efforts are needed to develop environmentally friendly construction materials. Unlike with regular concrete the chemical reactions that form geopolymer concrete alternative do not give off carbon dioxide or require high temperatures, which also lead to CO₂ emissions.
In geopolymer concrete, the geopolymer paste serves to bind the coarse and fine aggregates, and any un-reacted material. Geopolymer concrete can be utilized to manufacture pre-cast concrete, structural and non-structural elements, to make
concrete pavements, immobilize toxic waste, and produce concrete products that are resistant to heat and aggressive environments [5, 6]. This paper presents the technology of making geopolymer concrete using natural pozzolan as its source material and presents the results of laboratory tests conducted on this material. The research data presented in this paper are useful to understand the engineering properties of geopolymer concrete.

2. PREVIOUS RESEARCH ON GEOPOLYMER MATERIAL
The term “geopolymer” describing a family of mineral binders those have a polymeric silicon-oxygen-aluminum framework structure. The mechanism of geopolymerisation may consist of dissolution, transportation or orientation, and polycondensation [15], and takes place through an exothermic process [3, 8]. Different pathways for preparation of a synthetic geopolymer, including the order of addition of the raw materials, show different evolutions of compressive strength of the materials. The best method is to prepare an alkaline solution (mixing MOH and water and stirring for 2 minutes), adding pozzolan to alkaline solution for 15 minutes in a mixer, followed by sodium silicate, and mixing for 15 minutes [10]. The geopolymer paste serves to bind the coarse and fine aggregates and any un-reacted material to prepare geopolymer concrete [5, 6]. The nature of the fresh geopolymer concrete is stiff paste with high viscosity hence it tends to have low workability [5, 6]. A geopolymer mix can be timed to set either fast or slow, by adjusting the mixture components. Depending on the synthesis conditions, structural integrity and reasonable strength were attained in a short time, sometimes in as little as sixty minutes [14]. With the use of granulated blast furnace slag as the source material with the addition of metakaolinite, Cheng and Chiu (2003) found that the setting time of geopolymer paste was affected by curing temperature, type of alkaline activator and the composition of source material. They stated that the setting time of above geopolymer paste was between 15 to 45 minutes at 60°C. The laboratory experience by Hardjito, Wallah, Sumajouw and Rangan (2004) showed that the fresh geopolymer concrete could be handled for at least 120 minutes after mixing, without any sign of setting and degradation in compressive strength. These have mostly depended on the compounds in the source material, for instance the higher the content of CaO, the faster the setting. The presence of compounds other than Al₂O₃ and SiO₂ in the source material may also delay the setting. The pfa based geopolymers show faster initial setting time at higher temperatures and the final setting of these mortars occur from 15 to 25 minutes after the initial setting [7]. There are many different views as to which main parameters affect the compressive strength and other mechanical properties of geopolymer concrete. Palomo et al. (1999) stated that the significant factors affecting the compressive strength are the type of alkaline activator, the curing temperature and the curing time [5, 6]. Other researchers have reported that the important parameters for satisfactory polymerization are the relative amounts of Si, Al, K, Na, molar ratio of Si to Al in solution, the ratio of alumina silicate mineral to kaolinite, type of alkaline activator, water content, and curing temperature [1, 12, and 15]. The presence of silicate ions
in the aqueous substantially improves the mechanical strength and modulus of elasticity values but has a slightly adverse effect on the otherwise very strong matrix/aggregate and matrix/steel bond [4]. Experimental results show that the \( \text{H}_2\text{O}/\text{M}_2\text{O} \) molar ratio in the mixture composition is a significant parameter affecting the compressive strength of fly ash based geopolymer concrete, whereas the influence of the \( \text{Na}_2\text{O}/\text{SiO}_2 \) molar ratio is less significant. An increase of the \( \text{H}_2\text{O}/\text{M}_2\text{O} \) molar ratio and water to geopolymer solids ratio decreases the compressive strength of geopolymer. In addition, Van Jaarsveld et al. (2002) found that curing at elevated temperatures for long periods of time may weaken the structure of hardened material. The research on fly ash-based geopolymer binder, Palomo, Grutzeck, and Blanco (1999) have confirmed that curing temperature and curing time significantly influenced the compressive strength but seems not to be same for different alumina silicate. Longer curing time and higher curing temperature increased the compressive strength in fly ash based geopolymer concrete, although the increase in strength may not be significant for curing at more than 60°C and for periods longer than 48 hours [5, 6]. In most cases 70% of the final compressive strength is developed in the first 4 hours of setting. Because the chemical reaction of the geopolymer paste is a fast polymerization process, the compressive strength does not vary with the age of concrete, after it has been cured for 24h. This observation is in contrast to the well-known behaviour of OPC concrete, where the hydration process extends over a long period and hence strength increases over time [5, 6]. Another Kinetic difference between the Portland and alkaline systems is the existence of a relatively low threshold temperature in the former, above which thermal curing can have an adverse effect on the mechanical development and even on material durability. In an activated ash, on the contrary, a suitable choice of reaction time and curing temperature can yield different reaction product without detracting from material durability, because according to Fernandez, Palomo, and Hombradoz (2006) increases in the curing temperature go hand-in-hand with decreases in the amount of Al incorporated into the final product and a concomitant improvement in mechanical properties. Such improvements parallel the formation of a homogeneous alumina-silicate matrix [3].

When alkali-activated slag cement concrete is cured in water, compressive strength of the concrete keeps increasing until 365 days. However, if the concrete is cured in sealed condition the strength stopped increasing at about 90 days. This may be attributed to the lack of moisture available for the hydration of slag inside the concrete. The concrete exposed to air exhibit the lowest strength all the time and strength retrogression at ages greater than 28 days. The strength reaches a maximum after 14 to 28 days of hydration, and then starts to decrease [13]. Puertas et al. (2003) reported that the elastic modulus of OPC mortars was 5679 MPa, also higher than the values for activated PFA mortars (4441 MPa). Fernandez-Jimenez et al. (2006) found that the addition of soluble silicates in the alkali solution improves the modulus of elasticity. However, this improvement was not sufficient and the alkali activated PFA concrete showed a much lower static modulus of elasticity than expected. The values presented for OPC concrete ranged
from 30.3 to 32.3 GPa while for geopolymeric concrete ranged from 10.7 (without silicate) to 18.4 GPa (with silicate). Hardjito et al. (2004) observed better elastic modulus results for a concrete samples made in similar conditions: 22.95 to 30.84 GPa.

Geopolymers also attain shrink much less on setting (for 7 days 0.2% & for 28 days 0.5% of OPC) [14]. The explanation for this behaviour is to be found in the micro structural characteristic of the new binder and the main reaction product of the alkali activation of fly ash which causes a zeolite-type phase. Zeolite properties and microstructure are widely known to be unaffected by the loss of the water incorporated during their synthesis because not only water loss is reversible in most zeolites but also they are able to absorb water from the humidity in atmosphere [4].

3. EXPERIMENTAL WORK
3.1. Material and Mixing Procedure

In this research, Taftan andesite was selected as the most reactive natural pozzolan in Iran, used to produce Portland pozzolan cement by Khash Cement Factory. The chemical composition was analysed by XRF and presented on Table 1. Potassium hydroxide was used as pellets to produce the alkaline solution for geopolymeric concrete production. It was a 98% pure KOH supplied by MERK International Ltd. Sodium silicate was also provided by Iran Silicate Industrial Company in the form of granules, powder and solution (water glass). The chemical composition of the solution provided by the manufacture was: 8.5% of sodium oxide (Na₂O), 26.5% of silicon oxide (SiO₂) and 65% of water. Aggregate used in this study was obtained from deposits of Karaj River in northwest of Iran comprising 14mm and 4.75mm coarse aggregates and fine sand. The fineness modulus of combined aggregates was 2.08. The proportioning of the concrete mixture was based on the BRE method targeting a 40 MPa (28 days) compressive strength and a slump of 70 mm. Then, the amount of cement was substituted with the same quantity of natural pozzolan and the amount of water was ignored since there was already water in the alkali solution. The BRE method was used only to decide what is a common ratio of binder, sand and coarse aggregates, it was not expected that the actual 28 days compressive strength or the slump would be in accordance with the values designed. The details of the different mixes are presented in Table 2 and the notation for the mixes is as follows:

CM1: PC control mix with w/c=0.45
CM2: PC control mix with w/c=0.55
ATAF1: Activated Taftan pozzolan with w/c=0.45
ATAF2: Activated Taftan pozzolan with w/c=0.55

The mixing of the geopolymeric concrete was carried out in two different mixers sequential. The paste was prepared in a Hobart mixer (2 litre capacity) and added to a horizontal pan mixer (20 litre capacity) which contains the aggregates. Taftan specimens were de-moulded 24 hours after casting. Then they were cured in two curing regimes and at three different temperatures:
1. Sealed curing: Three series of specimens were sealed wrapped in a special plastic covering which was tested to be impermeable and stored in a controlled room kept at three different temperature equal to 20±2, 40±2 and 60±2°C.

Table 1: Chemical composition (oxide percent) of the materials used in this investigation reported by Kansaran Binaloud X-ray laboratory in Tehran, Iran (2005-2006)

<table>
<thead>
<tr>
<th>Material</th>
<th>LOI</th>
<th>SiO₂</th>
<th>Al₂O₃</th>
<th>Fe₂O₃</th>
<th>CaO</th>
<th>MgO</th>
<th>TiO₂</th>
<th>K₂O</th>
<th>Na₂O</th>
</tr>
</thead>
<tbody>
<tr>
<td>Taftan andesite</td>
<td>1.85</td>
<td>61.67</td>
<td>15.90</td>
<td>4.32</td>
<td>7.99</td>
<td>2.04</td>
<td>0.438</td>
<td>2.12</td>
<td>3.21</td>
</tr>
</tbody>
</table>

2. Steam curing: Three series of the specimens were put in the steam curing chamber set at three different temperatures equal to 20±2, 40±2 and 60±2°C for measuring compressive and splitting tensile strength and one series was put at 40±2°C for other measurements.

3.2. Testing Procedures

In order to determine the compressive strength of geopolymeric concrete each of the subsequent mixtures were prepared in 100x100x100 mm cubes and the compressive strength for these samples were tested according to BS EN 12390-3:2000. Details of casting and curing are described in section 3.1. Three samples of each condition were tested for 1, 7, 14, 28, 90, 180 and 365 days, and the average compressive strength values reported as the results.

PUNDIT was used to measure the ultrasonic pulse velocity in accordance with BS 1881: part 203: 1986. The measurement was conducted on the 100mmx100mm end face of prism with a length of 500mm. Duplicate sets of samples were tested at 28, 90, 180 days. For measuring ultrasonic pulse velocity, a pulse of longitudinal vibration is produced by means of 54 kHz an electro acoustical transducer of 50mm diameter and picked out by another transducer after travelling a known path length. The splitting tensile strength of all mixes was measured using 100mm Φ x 200mm length cylinders. The samples were prepared and splitting tensile tests performed as described in BS EN 12390-6:2000. The specimens were tested in duplicate sets at 7, 14, 28, 90, 180 and the average results are reported.

The static modulus of elasticity is determined according to BS1881-121:1983
standard by subjecting a 100mmØx200mm cylinder specimen to uni-axial compression and measuring the deformation by means of dial gauges fixed between certain gauge lengths. Dial gauge reading divided by gauge length will give the strain while load applied divided by area of cross section will give the stress. A series of readings are taken and the stress-strain relationship is established. The modulus of elasticity so found out from actual loading is called static modulus of elasticity.

In the present work the changes in length of 75x75x280mm concrete prisms were measured by commonly used mechanical equipment. Predrilled metal studs were fixed to either end of the concrete specimen with the adhesive at a preset spacing with the aid of a standard calibration bar. The distance between each two pins located into the stud holes was measured by an accuracy of about 0.0025 mm at certain times. One end of the reference rod was designed as the top and it was kept uppermost during all measurements. The prisms were placed in the apparatus with the marked end uppermost and were rotated slowly around the contact surfaces to measure the changes in length. For each mix two specimens were cast and cured for 3 and 7 days. The prisms were then left in a room controlled at 20°C and 70% humidity room and chemical shrinkage was measured using the length comparator in accordance with BS 812: Part 120: 1989.

4. RESULTS AND DISCUSSION

4.1. Compressive Strength

The results of the compressive strength tests on geopolymeric concrete using activated natural pozzolan and control Portland cement concrete mixes are presented in Figures 1.

In all cases, the strength of the concretes increased with age. The rate of strength gain is high at early ages and gradually decreases at longer ages. Geopolymeric concrete mixes mostly showed lower strengths than OPC control mixes at early ages, but they reached the same and even higher strengths than OPC mixes after long-term aging. ATAF1 have the highest compressive strength equal to 43.5 MPa. While ATAF2 mix has resulted compressive strength equal to 39.1 MPa after 365 days which is close to the amount resulted for OPC control mix.

It is well known that the lack of curing greatly affect the strength development. Figure 1 clearly shows the effect of different curing temperatures in two conditions of curing: sealed and steam curing. It generally sealed condition gives the best results in long term the same as OPC control concrete although the difference between the two conditions is not significant.

The results suggest that the optimum temperature for curing alkali-activated Taftan pozzolan is 60°C at early ages but curing at 40°C under sealed conditions gave the highest strength results in the long-term.

An increase of the water to binder ratio decreases the compressive strength of geopolymer concrete significantly (Figure 2).
Long term compressive strength of activated Taftan Pozzolan (W/C=0.4)

Long term compressive strength of activated Taftan Pozzolan (W/C=0.45)

Long term compressive strength of activated Taftan Pozzolan (W/C=0.5)

Long term compressive strength of alkali activated Taftan Pozzolan (W/C=0.55)

Figure 1. Effect of different curing condition and curing temperature on compressive strength development for activated Taftan pozzolan with different water to binder ratio

Figure 2. Compressive strength at 28 days versus water to binder ratio (W/B) for alkali activated Taftan pozzolan at different curing temperature
4.2. Ultrasonic Pulse Velocity

Figure 3 shows the results for the ultrasonic pulse velocity test for all mixes. The figure shows that ATAF1 achieved the highest values followed by ATAF2. Tabular form of the results is shown in Table 3. Comparing the results with Table 4, which gives the pulse velocity rating as suggested by Central Water and Power Research Station, Khadakwasla (India), presents lower velocity corresponding to the same compressive strength. It seems that in geo-polymeric concrete due to its lower density the velocity of pulses are lower than OPC concrete.

![Figure 3. Ultrasonic Pulse Velocity for different mixes](image)

<table>
<thead>
<tr>
<th>Mixes</th>
<th>Age(days)</th>
<th>Velocity(km/sec)</th>
<th>Compressive strength(MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CM1</td>
<td>28</td>
<td>3.95</td>
<td>29.8</td>
</tr>
<tr>
<td></td>
<td>90</td>
<td>5.0</td>
<td>39.23</td>
</tr>
<tr>
<td></td>
<td>180</td>
<td>4.95</td>
<td>39</td>
</tr>
<tr>
<td>CM2</td>
<td>28</td>
<td>3.5</td>
<td>24.78</td>
</tr>
<tr>
<td></td>
<td>90</td>
<td>4.7</td>
<td>36.78</td>
</tr>
<tr>
<td></td>
<td>180</td>
<td>4.6</td>
<td>36</td>
</tr>
<tr>
<td>ATAF1</td>
<td>28</td>
<td>3.39</td>
<td>39.7</td>
</tr>
<tr>
<td></td>
<td>90</td>
<td>4.0</td>
<td>35.6</td>
</tr>
<tr>
<td></td>
<td>180</td>
<td>4.5</td>
<td>40.97</td>
</tr>
<tr>
<td>ATAF2</td>
<td>28</td>
<td>3.03</td>
<td>21.36</td>
</tr>
<tr>
<td></td>
<td>90</td>
<td>4.2</td>
<td>38.72</td>
</tr>
<tr>
<td></td>
<td>180</td>
<td>4.2</td>
<td>38.06</td>
</tr>
</tbody>
</table>

Table 4: Quality Criteria Suggested by Central Water and Power Research Station Khadakwasla (India)

<table>
<thead>
<tr>
<th>Velocity (km/sec)</th>
<th>Classification (Quality)</th>
<th>Compressive strength(Kg/cm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.0 and above</td>
<td>Very good</td>
<td>300 to 350</td>
</tr>
<tr>
<td>3.5 to 4.0</td>
<td>Good</td>
<td>250 to 300</td>
</tr>
<tr>
<td>3.0 to 3.5</td>
<td>Medium</td>
<td>200 to 250</td>
</tr>
<tr>
<td>3.0 and below</td>
<td>Poor</td>
<td>150 to 200</td>
</tr>
</tbody>
</table>
4.3. Indirect Tensile Strength

The results of the indirect tensile strength tests up to 180 days are shown in Figure 4. The trend in tensile strength is similar to that obtained for compressive strength. Tensile strength increases as time proceeds. Figure 6 illustrates a difference development in tensile strength of different mixes. As far as the geopolymeric concrete mixes based on activated natural pozzolan are concerned, higher strength was observed at longer ages in comparison with control mixes. At early age, ATAF2 shows lower tensile strength results than OPC control mix. Although, the tensile strength of ATAF1 geopolymeric concrete mix is 3.57 MPa after 28 days and higher than the OPC control mix. The results of long term tensile strength show that Taftan geopolymeric concrete mixes have higher tensile strength than OPC control mix equal to 3.69 and 3.0 MPa after 365 days.

The tensile strength of this type of concrete as compared to its compressive strength, is more sensitive to improper curing, the same as OPC concrete. Figure 4 illustrates the effect of curing conditions and temperatures on tensile strength of concrete based on activated natural pozzlans. The optimum temperature of curing is 40°C, the same as that found for compressive strength.

Figure 4 shows that the higher water to binder ratio results in lower tensile strength, same as OPC mixes.
4.4. Static Modulus of Elasticity

Results of the static modulus of elasticity are shown in Figure 5. Similar to the compressive strength results of the mixes, static modulus of elasticity increases with age. This improvement is fast in the first 28 days as the most of the modulus value is generally achieved in this period. During the first 14 days the mixes made with activated natural pozzolans have mostly shown lower values of static modulus of elasticity than OPC concrete mixes, except ATAF1 mix. The static modulus of elasticity for ATAF1, ATAF2 mixes after 14 days is 33.96, 14.033GPa, respectively with that for the CM1 mix is 26.55GPa. Long term results show that the static elastic modulus of some of alkali activated natural pozzolans such as ATAF1 are around 5% to 20% more than OPC mixes. The long term static modulus of elasticity of ATAF1, ATAF2 mixes were resulted at 32.664, 26.805GPa in compare with OPC concrete mixes which resulted 29GPa.

The elastic modulus was affected by the curing temperatures and conditions. Early age static modulus of elasticity increases with increasing the curing temperature to a limit which seems to be related to the water to binder ratio. For ATAF1 with water to binder ratio equal to 0.45, the elastic modulus increases with increasing curing temperature up to 40°C and then decreases when the curing temperature rises up to 60°C. This optimum temperature to achieve higher static modulus of elasticity raises to 60°C for ATAF2 mixes with water to binder ratio equal to 0.55. However, the long term results drop to the same amount resulted for the mix cured at 40°C.

4.5. Drying Shrinkage

The shrinkage time curves are shown in Figures 6. From this investigation the following observations are made:

Figure 6. Effect of length, temperature and condition of curing on drying shrinkage development with age for ATAF1 and ATAF2 mixes
1) The graphs show that the magnitude of shrinkage increases with time and the rate of shrinkage decreases rapidly with time. The rate of shrinkage in Taftan pozzolan mixture was similar but not as rapid as the rate of development of strength and, seems to be constant after 60 days.

2) In OPC concrete one of the important factor which influences the magnitude of shrinkage is water to cement ratio of concrete and the values of shrinkage increases with the increasing of this ratio. The results indicate that the total water to binder ratio has a significant effect on the shrinkage properties of geopolymer concretes as well and seems to be contrary to the behaviour of OPC concrete, where the higher the water to binder ratio lowers the amount of drying shrinkage resulted. The maximum amount of final drying shrinkage for ATAF2 mix observed is 43% of that for ATAF1. The shrinkage of ATAF1 and ATAF2 mixes at 180 days at same curing conditions were $1185 \times 10^{-6}$ and $514 \times 10^{-6}$, respectively.

3) The results show that at a given water to binder ratio, the drying shrinkage at all ages varied with different curing regimes and temperatures. In concrete based on alkali activated natural pozzolan, the higher the curing temperature the lower the amount of drying shrinkage resulted. The lowest amount of drying shrinkage for different curing temperatures correspond to ATAF1 and ATAF2 mixes was $239 \times 10^{-6}$ and $161 \times 10^{-6}$, respectively and achieved for mixes cured at 60ºC. It can be observed that steam curing shows higher amount of drying shrinkage. This phenomenon may be related to the pozzolan nature minerals and because of swelling properties of its minerals during absorbing moisture. When the samples are subjected to wetting condition, they start swelling. Swelling is due to the adsorption of water by the natural minerals in pozzolan gel. The water molecules act against the cohesive force and tend to force the gel particles further apart as a result of which swelling takes place. In addition, the ingress of water decreases the surface tension of the gel. The property of swelling when placed in wet condition, and shrinking when placed in drying condition. While in OPC concrete the magnitude of shrinkage is less sensitive to moisture movement in concrete.

4) The length of curing affects the amount of drying shrinkage as well. The specimens cured for a period of three days seem to absorb environmental humidity similar to zeolites but after 7days the property of water absorption reduces. Thus, the former show lower amount of shrinkage than the latter. In calcined Shahindej, this phenomenon is not observed.

5. CONCLUSION

The main conclusions drawn from the investigation of engineering properties of geopolymeric concrete based on activating natural pozzolans (i.e. alkali activated natural pozzolan or AANP) are summarized as follows:

1) Geopolymeric concrete mixes based on activated natural pozzolans have mostly shown lower strength than OPC mixes at early ages, but they have reached the same and even higher strength than OPC mixes after long-term.

2) It seems that the ultrasonic pulse velocity in the geopolymeric concrete due to
its inherent property caused lower density is lower than in OPC concrete having the same compressive strength.

3) During the first 14 days the mixes made with activated natural pozzolans have mostly shown lower values of static modulus of elasticity than OPC concrete mixes. However, long term results show that the static elastic modulus of alkali activated natural pozzolans concrete is generally around 5 to 20% more than OPC mixes.

4) The elastic modulus of AANP concrete was affected by the curing temperatures. Early age static modulus of elasticity increases with increasing the curing temperature to a limit which seems to be related to the water to binder ratio. That means if there be the lack of water due to its evaporation at higher temperature before the full strength is gained the static modulus of elasticity decreases at higher temperatures.

5) The AANP concrete mixes may exhibit lower drying shrinkage in comparison with the OPC mixes at the same water to binder and cement to aggregate ratio.

6) The results indicate that the total water to binder ratio have a significant effect on the shrinkage properties of geopolymer concretes and seems to behave differently to OPC concrete in this respect. Where, higher the water to binder ratio for geopolymer concrete the lower is the amount of drying shrinkage.

7) In concrete made with alkali activated natural pozzolan, the higher the curing temperature, the lower the amount of drying shrinkage resulted.

8) It can be observed that steam curing shows higher amount of drying shrinkage. This phenomenon may be related to the pozzolan natural minerals and because of swelling properties of its minerals during absorbing moisture.

REFERENCES


USING SIMPLE PASTE AND MORTAR TESTS RESULTS TO OPTIMIZE SELF CONSOLIDATING CONCRETE

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ABSTRACT
Self-consolidating concrete (SCC) is usually proportioned with several mineral and chemical admixtures. A key factor for a successful formulation is a clear understanding of the role of the various constituents in the mix and their effects on the fresh and hardened concrete. In this research a three phase mix design procedure starting from designing the paste and then mortar leading to the design of a stable SCC was adopted. Selection of type and amount of paste ingredients was based on the result of a previous study presented elsewhere. A modified version of Marsh cone test, called FlowCyl, was utilized to quantify the viscosity related property of the paste. Also, using a miniature slump cone the spread diameter of the paste, which is related to its yield stress, was measured. Then, using a reduced size V-funnel and a mini slump cone, the influence of volume of selected paste on the flow behavior of mortar was studied and the mortar suitable for SCC was chosen. In this method if segregation or bleeding occurs in paste or mortar, the mix proportion would be adjusted at the corresponding design phase. Finally, by adding various quantities (volume) of the chosen mortar to the coarse aggregate, the effect of mortar volume on the flow behavior of SCC was studied and an optimized SCC with satisfactory workability was achieved.

1. INTRODUCTION
Over the last decade, SCC has been the most important subject in concrete research since it has unique properties and numerous advantages in practical applications. Self-consolidating concrete is attractive for several reasons including the reduction of: manpower, construction time, noise disturbance on the job site, defects on the concrete surface, and the overall project cost [1-4].

The main factor in the mix design of SCC is the increase in the powder content, to increase the separation of the aggregate particles. As the use of cement for the entire powder fraction would be too expensive and detrimental for durability of the structure, as well as green environment, fine powdered mineral admixtures such as limestone powder, micronized quartzite, fly ash, silica fume etc. are usually substituted for cement. SCC also relies on the use of superplasticizers (SP) for achieving its fluidity and, often, viscosity modifying agents (VMA) to prevent segregation and to achieve robustness [1-4]. Due to the complexity of the mix design, with several chemical and mineral admixtures, the proportion design and
compatibility of these admixtures should be optimized via testing the paste and mortar [5- 7]. The properties of paste and mortar can be established by using only a few liters of the material, while providing the designer with useful information for understanding the effects of various constituents on their fresh and hardened properties as well as predicting the behavior of fresh and hardened concrete. Fresh SCC is a multiphase material but, from a practical viewpoint can be regarded as a two-phase material namely: a solute being suspended in a solution [8]. In the model the solute can be considered as either coarse aggregates, or fine and coarse aggregates in which case the solution would be either highly flowable mortar or paste, respectively. The flow properties and segregation resistance (stability) of SCC are consequently controlled via proper adjustment of the rheology of the mortar as well as that of paste and adequate selection of the content of aggregates [8-10]. Numerous researchers have successfully used the two point test, introduced by Tattersall [11], to measure the rheological properties of concentrated suspensions. In this model, using Bingham equation, two parameters define the flow of paste, mortar and concrete: yield stress and plastic viscosity [12-14]. The measurement of yield stress and plastic viscosity requires a rheometer and where it is not available, alternative simpler one point tests can be used to assess the workability. The slump spread-flow and flow time or rate of discharge from a funnel is used by many researchers to identify the yield stress and viscosity of paste, mortar and concrete [15, 16]. Obviously the size and shape of the apparatus used for testing concrete is bigger than that of mortar; and for testing the paste, mini-sized apparatus is used. In fact extensive concrete testing requires a large amount of materials and labor, which is expensive and is not always practical. While, testing paste and mortar is easier, cheaper, and needs much less material than concrete testing. There is, therefore, a need to engineer the mix design proportion of SCC starting from paste design and then mortar leading to a SCC with satisfactory desired properties. The objective of this study was to use a three phase mix design procedure starting from designing the paste and then mortar leading to the design of a stable SCC. A modified version of Marsh cone test called FlowCyl, introduced by Mortsell [17] and widely used in European countries [18], was utilized to quantify the viscosity, called flow resistant ratio $\lambda_Q$, of the paste. Also, using a miniature slump cone (see Figure 1-a), the spread diameter of the paste, which is related to its yield stress, was measured. Then, the influence of paste volume on the flow behavior of mortar was studied via a reduced size V-funnel and mini slump cone suggested by EFNARC [1]. The self compactability of the mortar was examined by a small U-box suggested by Saak et al [2]. And the mortar suitable for SCC design was chosen. In this method if segregation or bleeding occurs in paste or mortar, the mix proportion would be adjusted at the corresponding design phase. Finally, the flow behavior of SCC, using slump flow and V-funnel flow time, was studied by adding various quantities (volume) of the chosen mortar to the coarse aggregate and optimized SCC with satisfactory flow behavior was achieved. Passing ability of SCC was verified by L-box test. A mix design program based on volume proportioning and the objectives of this study was prepared using spread sheet software (EXCEL). The slump cones and funnels used in this study are shown in Figure 1.
2. SIGNIFICANCE OF THE RESEARCH
For many years concrete was composed of only three materials (aggregate, cement, and water) and the simple mix design methods such as ACI-211 was used to proportion the ingredients. But in the recent years, the concrete mixture-proportioning problem has become more and more complicated. With the appearance of new components like organic admixtures and supplementary cementitious materials, SCC with several numbers of ingredients and high performance expectation has been emerged. Its design must be engineered starting first from the paste and then mortar, leading to an optimized SCC mixture with its required fresh and hardened properties fulfilled. It is shown that following this approach and using the results of very simple paste and mortar tests, not only simplifies the SCC mix design process, but also optimizes the mix proportion. The collection of test results utilizing this approach would make a valuable data base for the design engineer and ready concrete mix supplier.

3. EXPERIMENTAL PROGRAM
3.1. Materials and Test Variables
Type I-425 Portland cement with density=3.15 and silica fume with density=1.9 was used. Micronized quartzite with density of 2.65 was used as filler. The chemical properties of cement are given in Table 1. The density and absorption capacity of fine and coarse aggregates are given in Table 2 and their grain size distribution are given in Figures 2 and 3.

<table>
<thead>
<tr>
<th>Table 1. Properties of Portland cement</th>
</tr>
</thead>
<tbody>
<tr>
<td>SiO₂%</td>
</tr>
<tr>
<td>Al₂O₃%</td>
</tr>
<tr>
<td>Fe₂O₃%</td>
</tr>
<tr>
<td>MgO%</td>
</tr>
<tr>
<td>CaO%</td>
</tr>
<tr>
<td>SO₃</td>
</tr>
<tr>
<td>LOI%</td>
</tr>
<tr>
<td>C₃S%</td>
</tr>
<tr>
<td>C₃A%</td>
</tr>
<tr>
<td>(Na₂O+0.658K₂O)%</td>
</tr>
</tbody>
</table>
The superplastisizer used was based on carboxylic ether polymer with long lateral chains (Gelenium 110) and 40% solid content. It has density of 1.07. Two water powder ratios (W/P) of 0.25 and 0.30 were considered and utilizing the result of previous study presented elsewhere type and amount of powder and admixture was selected. The paste was designed and tested. Then mortar with paste volume of 50, 55 and 60 percent was made and its flow behavior was investigated; any necessary adjustments were made and the optimized mortar was selected. Finally SCC with mortar volume of 67.5, 70 and 72.5 percent was made from the selected mortar and the effect of mortar volume on the flow behavior of SCC was studied. At this phase, based on the design requirements, an optimized SCC would be selected. A data base of the collected mix design along with the result of performed tests could be used by design engineer and ready concrete mix supplier to obtain a reliable model for fast and smart mix design.

3.2. Mix Proportions and Measurements
A mix design program based on volume proportioning was prepared using spread sheet software (EXCEL). The experiments were carried out in three phases described as follows.

3.2.1. Flow Measurement and Properties of Paste
The mix proportion design of pastes is given in Table 3. The following mixing procedure was adopted, using a 3 speed Hobart 5 liter mixer equipped with a standard whisk: all dry constituents were mixed for 30 seconds at low speed. Then the mix water and HRWRA were added while the mixer was running at low speed. After wet mixing for 120 seconds, the mixer was stopped and undispersed cement paste was detached from the walls of the bowl during 30 seconds. In the next 2 minutes, the mixing was done at medium speed. The mixer was then stopped for 5 minutes, covered by a plastic sheet and finally it was allowed to run at medium speed for 60 more seconds. After mixing was completed, the paste was poured into

Table 2. Properties of the aggregates

<table>
<thead>
<tr>
<th>Aggregate Type</th>
<th>SSD Density</th>
<th>Dry Density</th>
<th>Absorption Capacity %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fine</td>
<td>2.59</td>
<td>2.51</td>
<td>3.24</td>
</tr>
<tr>
<td>coarse</td>
<td>2.58</td>
<td>2.53</td>
<td>1.97</td>
</tr>
</tbody>
</table>

The grading curves of fine and coarse aggregate are shown in Figures 2 and 3.
a 3 liter plastic flask equipped with a tight cap for intermediate storage. Before the FlowCyl test was performed, the slump flow spread, temperature and density of paste was determined.

<table>
<thead>
<tr>
<th>Paste ID</th>
<th>W/P (%)</th>
<th>A/P (%)</th>
<th>C/P (%)</th>
<th>Sf/P (%)</th>
<th>Fi/P (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>P1</td>
<td>0.25</td>
<td>0.6%</td>
<td>80%</td>
<td>10%</td>
<td>10%</td>
</tr>
<tr>
<td>P2</td>
<td>0.30</td>
<td>0.6%</td>
<td>80%</td>
<td>10%</td>
<td>10%</td>
</tr>
</tbody>
</table>

### 3.3. The FlowCyl Test

The FlowCyl apparatus is a modification of the Marsh Cone test apparatus originally developed to characterize oil well cements. The original cone of the Marsh Cone test has been replaced by a cylinder ending in a cone with a narrow outlet. This modification has been done in order to simplify the time - flow rate relation. The inner diameter of the cylinder is 80 mm and the outlet 8mm. The total length is 400 mm of which the cylindrical part is 300 mm. The cylinder is placed vertically in a rack. The FlowCyl test started 10 minutes after contact of water and powder. The flask containing the paste was shaken rigorously, and the paste was filled into the FlowCyl apparatus. During filling, the bottom outlet of the FlowCyl was closed by finger tip. When the FlowCyl cylinder was filled to the top level mark, the outlet was opened and a steel bowl placed on an electronic balance connected to a computer collected the paste flowing out of the cylinder and the weight was recorded by computer in 2 second intervals. Using a spread sheet program, the viscosity related parameter called “flow resistance ratio \( \lambda_Q \)” is calculated. \( \lambda_Q \) value can vary between 0 and 1. The larger value corresponds to more viscose paste. The corresponding \( \lambda_Q \) was 0.90 and 0.72 for P1 and P2 respectively.

### 3.2.1. Flow Measurement and Properties of Mortar

After selection of past with adequate viscosity, 6 mortar mixtures with paste volume of 50, 55 and 60 percent were made. The mixture proportion is given in Table 4.

<table>
<thead>
<tr>
<th>Table 4: Mortar mix proportion design kg/m³</th>
</tr>
</thead>
<tbody>
<tr>
<td>M1</td>
</tr>
<tr>
<td>----</td>
</tr>
<tr>
<td>V_p (%)</td>
</tr>
<tr>
<td>W/P</td>
</tr>
<tr>
<td>C</td>
</tr>
<tr>
<td>Sf</td>
</tr>
<tr>
<td>Fi1</td>
</tr>
<tr>
<td>Fi2</td>
</tr>
<tr>
<td>W</td>
</tr>
<tr>
<td>Sp</td>
</tr>
<tr>
<td>FA</td>
</tr>
</tbody>
</table>

C=Cement, Sf=Silica fume, Fi1=Sand particles < 0.15mm, Fi2 = Micronized quartzite, W=Water, Sp = Superplastisizer, FA = Fine Aggregate, \( V_p \)= paste Volume
Mortar mixtures were prepared in 5 litter batches using an open pan forced action mixer. The mixing sequence consisted of homogenizing the dry materials (sand and powder) for 30 s, and then the water and super plasticizer were gradually added while the mixture was mixed for 180 seconds. After 300 s of rest, the mortar received another 120 seconds of mixing. The temperature and density of each mixture were measured.

The flow property of each mortar was evaluated using slump flow spread and V-funnel flow time tests. Both tests are based on the same principle as those for concrete but with a reduced scale version of the equipments. In the slump flow spread test the mini cone, placed on a smooth leveled surface, was filled with mortar and lifted. The final diameter of the mortar after self-weight flow on the smooth plate was measured in two perpendicular directions and the average was noted. In the V-funnel test, after filling the funnel the trap door was opened and the flow time was measured until the first sighting of daylight when looking down through the funnel. The passing ability and self compactability of mortar was verified using a small U-box with three steel rods spaced approximately 25 mm apart to act as obstacles, hindering the cohesive flow of the material. After filling one side of the box with mortar, the door separating the two chambers was lifted and the equilibrium height was measured. With regards to the results of these tests a suitable mortar was selected. Figure 1-B shows the apparatus used.

3.2.2. Flow Measurement and Properties of SCC

After selection of mortar with adequate flowability, 6 concrete mixtures with mortar volume of 67.5, 70, and 72.5 percent were made. The mixture proportion is given in Table 5.

<table>
<thead>
<tr>
<th>Table 5: SCC mix proportion design kg/m³</th>
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</thead>
<tbody>
<tr>
<td>C1</td>
</tr>
<tr>
<td>---</td>
</tr>
<tr>
<td>Vₘ %</td>
</tr>
<tr>
<td>W/P</td>
</tr>
<tr>
<td>C</td>
</tr>
<tr>
<td>Sf</td>
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<tr>
<td>Fi1</td>
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<tr>
<td>Fi2</td>
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<tr>
<td>W</td>
</tr>
<tr>
<td>Sp</td>
</tr>
<tr>
<td>FA</td>
</tr>
<tr>
<td>CA</td>
</tr>
</tbody>
</table>

Vₘ= Volume of mortar, C=Cement, W=Water, P=Powder, Sf=Silica fume, Fi1=Sand particles<0.15mm, Fi2=Micronized quartzite, Sp=Superplastisizer, FA=Fine Aggregate, CA=Coarse Aggregate, Paste volume in mortar=55%

Preparation of concrete and its mixing sequence was similar to that of mortar described in the previous section. The air content, temperature, and density of each mixture along with its flow properties including Slump flow spread, T₅₀₀, V-funnel flow time, V₅₀₀°F, and L-box blocking ratio were measured. These tests were
done according to “The European Guidelines for Self-Compacting Concrete” [18]. The slump flow and T$_{500}$ time is a test to assess the flowability and the flow rate of self-compacting concrete in the absence of obstructions. It is based on the slump test described in ASTM C-143 or EN 12350-2. The result is an indication of the filling ability of self-compacting concrete. The T$_{500}$ time is also a measure of the speed of flow and hence the viscosity of the self-consolidating concrete. The fresh concrete was poured into a cone as used for the slump test. When the cone was withdrawn upwards, the time from commencing upward movement of the cone to when the concrete flowed to a diameter of 500 mm was measured and recorded as the T$_{500}$ time. The largest diameter of the flow spread of the concrete and the diameter of the spread at right angles to it were then measured and the mean was recorded as the slump-flow.

The V-funnel test was used to assess the viscosity, filling ability, and passing ability of self-consolidating concrete. The funnel was filled with fresh concrete and the time taken for the concrete to flow out of the funnel is measured and recorded as the V-funnel flow time. To assess the static segregation tendency of the mix, this test was repeated but a 5 minute rest period was enforced after filling the funnel; and then the trap door was opened and the time of SCC discharge was noted as V$_{FT@5min}$.

The L-box test, with three bars, was used to assess the passing ability of self-consolidating concrete while flowing through tight spaces between reinforcing bars without segregation and blocking. The L-box, made from transparent plexiglass, was supported on a level horizontal base and the gate between the vertical and horizontal sections was closed. The concrete was poured into the vertical section of the L-box and after a minute the gate was raised so that the concrete could flow through the obstacle into the horizontal section of the box. When movement was ceased, the heights of concrete at the end of the horizontal section of the L-box (h$_1$) and just behind the steel bars (h$_2$) were measured and the blocking ratio h$_1$/h$_2$ was recorded.

4. RESULTS AND DISCUSSION

4.1. Appropriate Volume Percentage of Paste in Mortar

Mortar mixtures with the selected pastes consisting of 50, 55, and 60 percent paste were made to study the effect of amount of paste on the flowability of mortar. The test results are shown in figures 4 and 5. As shown in the figures increasing paste volume has a greater effect on the flowability of lower W/P mixture (higher λ$_Q$). Domone [19] has suggested that a mortar mini-slump spread range of 28 to 34 cm and mortar V-funnel flow time of 1 to 7.5 seconds would lead to SCC with satisfactory fresh properties recommended by European guidelines for SCC. With regards to this recommendation, as well as optimization purpose, it was decided to select the mortar with 55% paste volume for designing SCC.
4.3. Appropriate Volume Percentage of Mortar in SCC

Designing SCC using the results of paste and mortar tests was the last phase of this study. Mortar with 55% paste volume was selected and SCC with mortar volume of 67.5, 70, and 72.5 percent was made. W/P of 0.25 and 0.30 were considered and the fresh SCC was tested to study the effect of mortar volume on its flow properties. The test results are summarized in Table 6.

<table>
<thead>
<tr>
<th>Table 6. Fresh SCC test results</th>
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</thead>
<tbody>
<tr>
<td>SF mm</td>
</tr>
<tr>
<td>C1</td>
</tr>
<tr>
<td>C2</td>
</tr>
<tr>
<td>C3</td>
</tr>
<tr>
<td>C4</td>
</tr>
<tr>
<td>C5</td>
</tr>
<tr>
<td>C6</td>
</tr>
</tbody>
</table>

The values of slump flow and v-funnel flow time vs. mortar volume of SCC are plotted in Figures 6 and 7 respectively. As shown the flowability is increased with increasing amount of mortar. Even though the slump flow of mixes C1, C2, C4 and C5 satisfies the requirement of SCC but the corresponding values of L-Box blocking ratio and V-funnel flow time at 5 minutes was not satisfactory and had tendency to segregate. Based on these results mixes C3 and C6 with mortar volume of 72.5% had excellent flowability and passing ability with no sign of segregation or bleeding.
Figure 6 shows that the more viscose the paste is the more mortar volume is needed in order to obtain the same slump flow and the difference in mortar volume gets smaller as the volume percentage of mortar increases beyond 70%. Also at this level the passing ability of SCC starts to improve significantly.

5. CONCLUSION
Investigation of new materials and evaluation of different combination of ingredients for selection of their most effective combination for making SCC can be done utilizing the single point tests mentioned in this paper. A three phase mix proportion design procedure starting from paste and then mortar leading to desired SCC was adopted and a spread sheet (EXCEL) program was prepared to fulfill this purpose. The effect of paste viscosity and its volume percentage on rheological properties of mortar was studied. Mortar with 55% paste volume was selected and the effect of mortar volume on fresh properties of SCC was investigated. As mortar volume increased beyond 70%, filling and passing ability of SCC was improved. A data base may be defined to collect the result of tests performed for each design phase which could be used by design engineer and ready concrete mix supplier to obtain a reliable model for fast and smart mix design.

ACKNOWLEDGEMENTS
The work presented here has been performed within a research project financed by the Building and Housing Research Center of Iran. Their support is gratefully acknowledged.

REFERENCES
8. 8- Yahiaa, A., Tanimura M., Shimoyama Y., “Rheological properties of highly flowable mortar containing limestone filler-effect of powder content and W/C
AN INVESTIGATION ON EFFECT OF USING PP FIBERS AND DIFFERENT CEMENTITIOUS MATERIALS ON MECHANICAL PROPERTIES OF EPS CONCRETE

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1Assistant professor, Dept of Civil Engg, Faculty of Engg, University of Guilan, Rasht, Iran
2,3M.S. Student, Dept of Civil Engg, Faculty of Engg, University of Guilan, Rasht, Iran
4Professor, Dept of Textile Engg, Faculty of Engg, University of Guilan, Rasht, Iran

ABSTRACT
The use of lightweight concrete in many applications of modern construction is increasing, owing to the advantages that lower density results in decreasing the magnitude of dead load of the structure which lead to smaller cross sections for load bearing elements. Expanded polystyrene (EPS) beads are a type of artificial lightweight nonabsorbent aggregates which can be used to produce low density concretes by replacing with normal aggregates, either partially or fully, depending upon the requirements of density and strength. Also plastic shrinkage is the dimensional change that occurs in all fresh cement based materials within the first few hours after it has been placed which is not unacceptable in itself, but it is some times accompanied by development of cracks that are unsightly and objectionable. Polypropylene and other synthetic fibers are added to concrete as secondary reinforcement in order to control this plastic shrinkage. On the other hand, the addition of fibers affects on the properties of hardened concrete like compressive and tensile strength, elastic modulus and toughness. The present study covers the use of polypropylene fibers at contents equal to 0.1%, 0.3%, 0.5% and 1% by volume of EPS concrete in order to study about the effects of its addition into the EPS concrete matrix on mechanical properties. Also the effects of using Silica fume and Rice husk as two supplementary cementitious materials were investigated.

Keywords: EPS concrete, PP fibers, silica fume, rice husk, mechanical properties

1. INTRODUCTION
Lightweight concretes can be produced by replacing the normal aggregates in concrete either partially or fully, depending upon the requirements of density and strength [1]. Historically, lightweight concrete is used for both structural and non-structural applications. use of lightweight concretes in construction of high rise buildings, offshore structures and long span bridges due to the advantage of its low density, results in a significant benefit in terms of load bearing elements of smaller cross section and a corresponding reduction in the size of the foundation [2]. Lightweight aggregates are broadly classified in to two types, natural (pumice, diatomite, volcanic cinders, etc.) and artificial (perlite, expanded shale, clay, slate, sintered PFA, etc.). One of the main problems associated with the use of
conventional lightweight aggregates produced from clay, slate and shale in concrete is that these porous aggregates absorb a very large quantity of the mixing water. This is known to affect the performance of the concrete, apart from the fact that it is difficult to maintain specific water content during the casting. Also, this absorption of water by the aggregate will mean that additional water will be required to maintain the slump at acceptable levels. These increased water contents necessitate higher cement contents, even without the benefit of higher strength [3]. Expanded polystyrene is a kind of stable foam with low density, nonabsorbent, closed cell nature aggregates consisting of discrete air voids in a polymer matrix. As a type of artificial ultra-lightweight aggregate, the polystyrene beads can easily be incorporated in mortar or concrete to produce lightweight concrete, with a wide range of densities, required for building applications like cladding panels and load-bearing concrete blocks. Also, they can be used as a construction material for floating marine structures, as an energy-absorbing material for the protection of buried military structures and as fenders in offshore oil platforms [4]. Also, it was reported that it can be used for other specialized applications like the sub-base material for pavement and railway track bed, as construction material for floating marine structures, sea beds, and sea fences, as an energy-absorbing material for the protection of buried military structures, and as fenders in offshore oil platforms [5,6]. Polypropylene fibers have been widely used for the reinforcement of cementitious materials to improve the toughness and energy absorption capability of matrix [7]. They were found to be extremely effective in reducing free plastic shrinkage, in retarding first crack appearance and in controlling crack development [8]. Although effectiveness of PP fibers in shrinkage cracking, impact resistance and ductility of cement matrices has been proved by many researchers, effect of PP fibers on compressive and flexural strength is not quit clear [9]. Presently, a comprehensive investigation on the mechanical behavior of the EPS concretes containing polypropylene fibers is not available. In this study, concretes with different EPS contents, were reinforced with polypropylene fibers and the effects of using fibers on mechanical properties were evaluated.

2. EXPERIMENTAL INVESTIGATION
2.1. Materials and Mix Proportions
Cement: The cement used in all mortar mixes was ordinary Portland cement which corresponds to ASTM type 1. The chemical analysis of Portland cement is shown in Table 1.
Silica fume: Silica fume has been used as supplementary cementing materials to partially replacement for many years. It has been also used for producing high performance concrete or achieving other desired properties. The silica fume used in this study contained 91.1% of SiO₂ with average size of 7.38 µm.
Rice husk ash: Rice husk ash has been used in many countries as a low cost concrete admixture because of its role as filler and a pozzolan [12]. It has been also used. For producing high performance concrete (HPC) or achieving other desire properties. RHA used in this experiment contained 91.62% of SiO₂ with average size of 15.83 µm.
Table 1: Chemical composition of Cement and Silica fume and Rice husk ash

<table>
<thead>
<tr>
<th>Oxide</th>
<th>Portland cement</th>
<th>Silica fume</th>
<th>Rice husk ash</th>
</tr>
</thead>
<tbody>
<tr>
<td>SiO₂</td>
<td>21/00</td>
<td>91/10</td>
<td>91/62</td>
</tr>
<tr>
<td>Al₂O₃</td>
<td>4/60</td>
<td>1/55</td>
<td>0/49</td>
</tr>
<tr>
<td>Fe₂O₃</td>
<td>3/20</td>
<td>2/00</td>
<td>0/73</td>
</tr>
<tr>
<td>CaO</td>
<td>64/50</td>
<td>2/42</td>
<td>2/51</td>
</tr>
<tr>
<td>MgO</td>
<td>2/00</td>
<td>0/06</td>
<td>0/88</td>
</tr>
<tr>
<td>SO₃</td>
<td>2/90</td>
<td>0/45</td>
<td>-</td>
</tr>
<tr>
<td>Na₂O + K₂O</td>
<td>1/00</td>
<td>-</td>
<td>2/39</td>
</tr>
<tr>
<td>LOI</td>
<td>1/50</td>
<td>2/10</td>
<td>-</td>
</tr>
</tbody>
</table>

Superplastisizer: Superplastisizers are now widely used as additives in concrete with high rheological requirements. The use of superplastisizers allows reducing the water to cement ratio (w/c) of mortar and concrete without significantly changing their flow properties. Sodium salts of formaldehyde condensates disperse the cement particles by electrostatic repulsion which results from the adsorption on cement surfaces [13, 14]. Due to high specific surface of silica fume and rice husk ash which need more water for complete hydration, workability of concrete will be affected. In order to achieve desire fluidity, polycarboxylate ether was incorporated in to all mixes. The content of superplastisizer was adjusted for each mixture to keep constant the workability of concrete.

Aggregates: Natural river sand was used with specific gravity of 2.51 gr/cm³ and absorption capacity equal to 3.4%. Natural River gravel was used as coarse aggregate with specific gravity of 2.54 gr/cm³ and absorption capacity equal to 2.57%.

Figure 3. Sieve Analysis of used Sand and Gravel based on ASTM standard

EPS: The grading shows that used EPS has mostly (85%) 3.5 mm size beads. The density of used expanded polystyrene was evaluated to be 0.0257 gr/cm³.

Polypropylene fiber: polypropylene fibers which is used is waste carpet fibers,
has been cut by length of 6mm.

| Table 2: Characteristics of the polypropylene fibers |
|-------------------------------|-----------------|
| properties                     | description     |
| Morphology                     | Fibrillated or mono filament |
| Specific weight [gr/cm³]       | 0.95            |
| Diameter [μm]                  | 20 – 200        |
| Modulus of elasticity [GPa]    | 5 – 10          |
| Tensile strength [MPa]         | 500 - 750       |
| Ultimate strain [%]            | 5 - 15          |
| Elongation at fracture [%]     | Approx. 20      |
| Melting point [°C]             | 160             |
| Bonding with cement            | Good            |
| Stability in cement            | Good            |

Mix proportions: Three percentages of using EPS of 15%, 25% and 40% by volume were listed. In order to investigate the effect of polypropylene fibers on mechanical properties of EPS concrete, it was used in mixes by four percentages of 0.1%, 0.3%, 0.5% and 1% by volume, silica fume and rice husk ash replacement were 10% and 20% by weight in the cementitious material, respectively. The complete details of the concrete mixes are presented in Table 3.

<table>
<thead>
<tr>
<th>Table 3: Mix proportion of the specimens</th>
</tr>
</thead>
<tbody>
<tr>
<td>mix No.</td>
</tr>
<tr>
<td>---------</td>
</tr>
<tr>
<td>1</td>
</tr>
<tr>
<td>2</td>
</tr>
<tr>
<td>3</td>
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<tr>
<td>4</td>
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<tr>
<td>5</td>
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<td>11</td>
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<td>12</td>
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</tbody>
</table>

Production of EPS concrete: EPS beads were wetted initially with a part of the mixing water and superplasticizer before adding the remaining materials. Mixing was continued until a uniform and flowing mixture was obtained. The fresh concrete densities and slump values were measured immediately after the mixing which showed a variation between 50 and 70cm. The specimens were cured under
wet gunny bags initially and, after demolding, were stored in water [11].

**Specimens:** Cube specimens of 100 mm were tested for the uniaxial compressive strength. Cylinders with a diameter of 150 mm and a height of 300 mm were tested for the splitting tensile strength and modulus of elasticity. The 50×50×200 mm beam specimens were tested in three point bending with the span of 180 mm with a cross head movement of 1 mm/min. all the tests were done by using a testing machine with a maximum load of 3000kN.

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2.2. Result and Discussion

**Fresh concrete:** The main parameter, which is often used to determine the workability of fresh concrete, is the slump test. The slump value depends mainly on the water absorption and porosity of the aggregates, water content in the mixture, amount of the aggregate and fine material in the mixture, shape of the aggregates and surface characteristics of the constituents in the mixture. The slump values decreased significantly with the addition of polypropylene. With the use of sufficient compaction, the fresh concrete would flow satisfactorily again and the polypropylene fibers would be uniformly dispersed in the mixture. Furthermore, with the constant water-cement ratio, the slump values of the concrete mixtures containing polypropylene fibers were not significantly affected by the aggregate types. This was primarily caused by the good adhesion in the fresh concrete, which was created by the polypropylene fibers. The mixes having the higher percentage of silica fume and rice husk ash show higher flow values. All the concretes were flexible and easy to work with, and could be easily compacted using just hand compaction.

**Compressive strength:** With or without using of EPS in mixes, the addition of polypropylene fibers in the concrete did not significantly affect the compressive strength of concretes. Test results showed increase in value of compressive strength in some specimens and on the other hand, some other showed decrease on mentioned parameter. Also from the results, it is clear that the rate of strength gain in early ages increased using silica fume as a replacement of ordinary cement. But rice husk ash, as a replacement of ordinary cement needs more time to show its
benefits as a pozzolanic material. In this case, an improvement in mechanical properties can be observed at the age of 90 days and upper. The desired density of EPS concrete can be determined by varying the EPS volume in the mix. The variations of compressive strength with the plastic density of concrete were observed to be linear. Moreover, the failure mode of the concrete specimens containing EPS aggregates under compressive loading observed to be gradual and the specimens were capable of retaining the load after failure without full disintegration. By adding polypropylene in EPS concrete matrix, the failure mode observed to be more gradual. This clearly shows the high energy absorption capacity of these concretes.

![Graph showing compressive strength of concrete at different contents of EPS and PP fibers](image)

**Figure 2. Compressive strength of concrete at different contents of EPS and PP fibers**

![Graph showing variation of compressive strength with age and cementitious replacement](image)

**Figure 3. Variation of compressive strength with age and cementitious replacement**

**Split tensile strength:** The variation of tensile strength with the EPS and polypropylene content of admixture is given in Figure 4. From this, it can be seen that the tensile strength increased with decreasing EPS content of concrete.
Figure 4. Variation of tensile strength with PP and EPS content of concrete

The splitting failure mode of the concrete specimens containing EPS aggregates was also observed to be gradual. Adding polypropylene fibers showed an increase in the value of tensile strength, as well as adding polypropylene in concrete matrix, the failure mode observed to be more gradual and specimens did not separate in two parts as shown in Figure 5. Effect of using silica fume and rice husk ash as a replacement of ordinary cement on split tensile is similar to the effects observed during compressive strength tests.

Figure 5. Effect of using polypropylene in concrete on failure mode

Modulus of elasticity: Static modulus of elasticity tests were carried out on the 150×300 mm EPS concrete cylinders. The results of these tests showed that this mechanical material property has a linear variation with the used volume of EPS in admixture, but vice versa. It means that an increase in the volume of EPS, used in concrete lead to decrease in the magnitude of the parameter modulus of elasticity.
which is expected. From the results, the addition of polypropylene fibers in the concrete did not significantly affect the modulus of elasticity of mixes. It means that the use of polypropylene fibers by the mentioned percentages in EPS concrete resulted in the low influence on modulus of elasticity of concrete rather than the influences contributed by the other constituents of concrete.

![Figure 6. variation of Modulus of Elasticity with PP and EPS content of concrete](image1)

The results also indicated that adding silica fume to admixture caused an increase in modulus of elasticity in all mixes, however using rice husk ash as a supplementary cementitious material caused a decrease in value of the modulus of elasticity at 28 days.

**Flexural behavior:** from the results, the flexural capacity decreased with an increase in the volume of EPS in mixes. Also no effect on the flexural behavior was observed. However, by increasing used polypropylene’s volume, flexural capacity was observed to be improved.

![Figure 7. variation of Flexural strength with PP and EPS content of concrete](image2)
3. CONCLUSION
The mechanical strength of EPS concrete showed a linear increase with an increase in concrete density. The failure was observed to be gradual (compressible), and the specimens were capable of retaining the load after failure, without full disintegration. The strength of EPS concretes appears to increase linearly with an increase in concrete density, or with a decrease in the EPS volume. With or without using of EPS in mixes, the addition of polypropylene fibers in the concrete did not significantly affect the compressive strength of concretes. The rate of strength gain was increasing with using silica fume as a replacement of ordinary cement. But rice husk ash, as a replacement of ordinary cement needs more time to show its benefits as a pozzolanic material. In this case, an improvement in mechanical properties can be observed at the age of 90 days and upper. Be seen that the tensile strength increased with an increase in compressive strength. Modulus of Elasticity of concretes decreased with the incorporation of EPS. High amounts of EPS contents, decreased the elastic module more. Effect of polypropylene fibers on module of elasticity was not clear. Adding silica fume to admixture increased the modulus of elasticity however using rice husk ash had not positive effect in early ages. Results of flexural behavior test showed that, the flexural strength decreased with an increase in the volume of EPS in mixes. Results showed that application of PP fibers improved the flexural strength of concrete.

REFERENCES
2. ACI Committee 213R-0.3, Guide for structural lightweight aggregate concrete, American Concrete Institute, Farmington Hills, MI, 2003.
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AN INVESTIGATION ON EFFECT OF USING PP FIBERS AND DIFFERENT CEMENTITIOUS MATERIALS ON MECHANICAL PROPERTIES OF EPS CONCRETE

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ABSTRACT
The use of lightweight concrete in many applications of modern construction is increasing, owing to the advantages that lower density results in decreasing the magnitude of dead load of the structure which lead to smaller cross sections for load bearing elements. Expanded polystyrene (EPS) beads are a type of artificial lightweight nonabsorbent aggregates which can be used to produce low density concretes by replacing with normal aggregates, either partially or fully, depending upon the requirements of density and strength. Also plastic shrinkage is the dimensional change that occurs in all fresh cement based materials within the first few hours after it has been placed which is not unacceptable in itself, but it is sometimes accompanied by development of cracks that are unsightly and objectionable. Polypropylene and other synthetic fibers are added to concrete as secondary reinforcement in order to control this plastic shrinkage. On the other hand, the addition of fibers affects on the properties of hardened concrete like compressive and tensile strength, elastic modulus and toughness. The present study covers the use of polypropylene fibers at contents equal to 0.1%, 0.3%, 0.5% and 1% by volume of EPS concrete in order to study about the effects of its addition into the EPS concrete matrix on mechanical properties. Also the effects of using Silica fume and Rice husk as two supplementary cementitious materials were investigated.

Keywords: EPS concrete, PP fibers, silica fume, rice husk, mechanical properties

1. INTRODUCTION
Lightweight concretes can be produced by replacing the normal aggregates in concrete either partially or fully, depending upon the requirements of density and strength [1]. Historically, lightweight concrete is used for both structural and non-structural applications. Use of lightweight concretes in construction of high rise buildings, offshore structures and long span bridges due to the advantage of its low density, results in a significant benefit in terms of load bearing elements of smaller cross section and a corresponding reduction in the size of the foundation [2]. Lightweight aggregates are broadly classified in to two types, natural (pumice, diatomite, volcanic cinders, etc.) and artificial (perlite, expanded shale, clay, slate, sintered PFA, etc.). One of the main problems associated with the use of
conventional lightweight aggregates produced from clay, slate and shale in concrete is that these porous aggregates absorb a very large quantity of the mixing water. This is known to affect the performance of the concrete, apart from the fact that it is difficult to maintain specific water content during the casting. Also, this absorption of water by the aggregate will mean that additional water will be required to maintain the slump at acceptable levels. These increased water contents necessitate higher cement contents, even without the benefit of higher strength [3]. Expanded polystyrene is a kind of stable foam with low density, nonabsorbent, closed cell nature aggregates consisting of discrete air voids in a polymer matrix. As a type of artificial ultra-lightweight aggregate, the polystyrene beads can easily be incorporated in mortar or concrete to produce lightweight concrete, with a wide range of densities, required for building applications like cladding panels and load-bearing concrete blocks. Also, they can be used as a construction material for floating marine structures, as an energy-absorbing material for the protection of buried military structures and as fenders in offshore oil platforms [4]. Also, it was reported that it can be used for other specialized applications like the sub-base material for pavement and railway track bed, as construction material for floating marine structures, sea beds, and sea fences, as an energy-absorbing material for the protection of buried military structures, and as fenders in offshore oil platforms [5,6]. Polypropylene fibers have been widely used for the reinforcement of cementitious materials to improve the toughness and energy absorption capability of matrix [7]. They were found to be extremely effective in reducing free plastic shrinkage, in retarding first crack appearance and in controlling crack development [8]. Although effectiveness of PP fibers in shrinkage cracking, impact resistance and ductility of cement matrices has been proved by many researchers, effect of PP fibers on compressive and flexural strength is not quiet clear [9]. Presently, a comprehensive investigation on the mechanical behavior of the EPS concretes containing polypropylene fibers is not available. In this study, concretes with different EPS contents, were reinforced with polypropylene fibers and the effects of using fibers on mechanical properties were evaluated.

2. EXPERIMENTAL INVESTIGATION
2.1. Materials and Mix Proportions
Cement: The cement used in all mortar mixes was ordinary Portland cement which corresponds to ASTM type 1. The chemical analysis of Portland cement is shown in Table 1.
Silica fume: Silica fume has been used as supplementary cementing materials to partially replacement for many years. It has been also used for producing high performance concrete or achieving other desired properties. The silica fume used in this study contained 91.1% of SiO₂ with average size of 7.38 µm.
Rice husk ash: Rice husk ash has been used in many countries as a low cost concrete admixture because of its role as filler and a pozzolan [12]. It has been also used. For producing high performance concrete (HPC) or achieving other desir properties. RHA used in this experiment contained 91.62% of SiO₂ with average size of 15.83 µm.
### Table 1: Chemical composition of Cement and Silica fume and Rice husk ash

<table>
<thead>
<tr>
<th>Oxide</th>
<th>Portland cement</th>
<th>Silica fume</th>
<th>Rice husk ash</th>
</tr>
</thead>
<tbody>
<tr>
<td>SiO$_2$</td>
<td>21/00</td>
<td>91/10</td>
<td>91/62</td>
</tr>
<tr>
<td>Al$_2$O$_3$</td>
<td>4/60</td>
<td>1/55</td>
<td>0/49</td>
</tr>
<tr>
<td>Fe$_2$O$_3$</td>
<td>3/20</td>
<td>2/00</td>
<td>0/73</td>
</tr>
<tr>
<td>CaO</td>
<td>64/50</td>
<td>2/42</td>
<td>2/51</td>
</tr>
<tr>
<td>MgO</td>
<td>2/00</td>
<td>0/06</td>
<td>0/88</td>
</tr>
<tr>
<td>SO$_3$</td>
<td>2/90</td>
<td>0/45</td>
<td>-</td>
</tr>
<tr>
<td>Na$_2$O + K$_2$O</td>
<td>1/00</td>
<td>-</td>
<td>2/39</td>
</tr>
<tr>
<td>LOI</td>
<td>1/50</td>
<td>2/10</td>
<td>-</td>
</tr>
</tbody>
</table>

**Superplastisizer:** Superplastisizers are now widely used as additives in concrete with high rheological requirements. The use of superplastisizers allows reducing the water to cement ratio (w/c) of mortar and concrete without significantly changing their flow properties. Sodium salts of formaldehyde condensates disperse the cement particles by electrostatic repulsion which results from the adsorption on cement surfaces [13, 14]. Due to high specific surface of silica fume and rice husk ash which need more water for complete hydration, workability of concrete will be affected. In order to achieve desire fluidity, polycarboxylate ether was incorporated in to all mixes. The content of superplastisizer was adjusted for each mixture to keep constant the workability of concrete.

**Aggregates:** Natural river sand was used with specific gravity of 2.51 gr/cm$^3$ and absorption capacity equal to 3.4%. Natural River gravel w used as coarse aggregate with specific gravity of 2.54 gr/cm$^3$ and absorption capacity equal to 2.57%.

**EPS:** The grading shows that used EPS has mostly (85%) 3.5 mm size beads. The density of used expanded polystyrene was evaluated to be 0.0257 gr/cm$^3$.

**Polypropylene fiber:** polypropylene fibers which is used is waste carpet fibers,
Table 2: Characteristics of the polypropylene fibers

<table>
<thead>
<tr>
<th>properties</th>
<th>description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Morphology</td>
<td>Fibrillated or mono filament</td>
</tr>
<tr>
<td>Specific weight [gr/cm³]</td>
<td>0.95</td>
</tr>
<tr>
<td>Diameter [μm]</td>
<td>20 – 200</td>
</tr>
<tr>
<td>Modulus of elasticity [GPa]</td>
<td>5 – 10</td>
</tr>
<tr>
<td>Tensile strength [MPa]</td>
<td>500 - 750</td>
</tr>
<tr>
<td>Ultimate strain [%]</td>
<td>5 - 15</td>
</tr>
<tr>
<td>Elongation at fracture [%]</td>
<td>Approx. 20</td>
</tr>
<tr>
<td>Melting point [°C]</td>
<td>160</td>
</tr>
<tr>
<td>Bonding with cement</td>
<td>Good</td>
</tr>
<tr>
<td>Stability in cement</td>
<td>Good</td>
</tr>
</tbody>
</table>

Mix proportions: Three percentages of using EPS of 15%, 25% and 40% by volume were listed. In order to investigate the effect of polypropylene fibers on mechanical properties of EPS concrete, it was used in mixes by four percentages of 0.1%, 0.3%, 0.5% and 1% by volume, silica fume and rice husk ash replacement were 10% and 20% by weight in the cementitious material, respectively. The complete details of the concrete mixes are presented in Table 3.

Table 3: Mix proportion of the specimens

<table>
<thead>
<tr>
<th>mix No.</th>
<th>Cement (kg/m³)</th>
<th>S.F %</th>
<th>R.H %</th>
<th>Water (kg/m³)</th>
<th>w/(c+s)</th>
<th>0-3 (kg/m³)</th>
<th>3-6 (kg/m³)</th>
<th>6-12 (kg/m³)</th>
<th>EPS %</th>
<th>PP %</th>
<th>Fresh Density (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>400</td>
<td>-</td>
<td>-</td>
<td>180</td>
<td>0.45</td>
<td>666</td>
<td>118</td>
<td>957</td>
<td>-</td>
<td>2400</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>400</td>
<td>-</td>
<td>-</td>
<td>170</td>
<td>0.43</td>
<td>540</td>
<td>95</td>
<td>777</td>
<td>15%</td>
<td>1900</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>400</td>
<td>-</td>
<td>-</td>
<td>165</td>
<td>0.41</td>
<td>431</td>
<td>76</td>
<td>620</td>
<td>25%</td>
<td>1700</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>400</td>
<td>-</td>
<td>-</td>
<td>160</td>
<td>0.4</td>
<td>294</td>
<td>52</td>
<td>423</td>
<td>40%</td>
<td>1350</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>360 10%</td>
<td>-</td>
<td>-</td>
<td>190</td>
<td>0.48</td>
<td>652</td>
<td>115</td>
<td>940</td>
<td>-</td>
<td>2300</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>360 10%</td>
<td>-</td>
<td>-</td>
<td>175</td>
<td>0.44</td>
<td>524</td>
<td>93</td>
<td>755</td>
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<td>7</td>
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<td>422</td>
<td>75</td>
<td>607</td>
<td>25%</td>
<td>1650</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>360 10%</td>
<td>-</td>
<td>-</td>
<td>170</td>
<td>0.43</td>
<td>282</td>
<td>50</td>
<td>406</td>
<td>40%</td>
<td>1300</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>320 20%</td>
<td>-</td>
<td>20%</td>
<td>210</td>
<td>0.52</td>
<td>620</td>
<td>110</td>
<td>895</td>
<td>-</td>
<td>2250</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>320 20%</td>
<td>-</td>
<td>20%</td>
<td>205</td>
<td>0.51</td>
<td>470</td>
<td>80</td>
<td>670</td>
<td>15%</td>
<td>1850</td>
<td></td>
</tr>
<tr>
<td>11</td>
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<td>-</td>
<td>20%</td>
<td>205</td>
<td>0.51</td>
<td>385</td>
<td>68</td>
<td>555</td>
<td>25%</td>
<td>1600</td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>320 20%</td>
<td>-</td>
<td>20%</td>
<td>200</td>
<td>0.5</td>
<td>245</td>
<td>43</td>
<td>352</td>
<td>40%</td>
<td>1250</td>
<td></td>
</tr>
</tbody>
</table>

Production of EPS concrete: EPS beads were wetted initially with a part of the mixing water and superplasticizer before adding the remaining materials. Mixing was continued until a uniform and flowing mixture was obtained. The fresh concrete densities and slump values were measured immediately after the mixing which showed a variation between 50 and 70cm. The specimens were cured under...
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![Figure 2. Compressive strength of concrete at different contents of EPS and PP fibers](image)

**Split tensile strength:** The variation of tensile strength with the EPS and polypropylene content of admixture is given in Figure 4. From this, it can be seen that the tensile strength increased with decreasing EPS content of concrete.

![Figure 3. variation of compressive strength with age and cementitious replacement](image)
The splitting failure mode of the concrete specimens containing EPS aggregates also observed to be gradual. Adding polypropylene fibers showed an increase in the value of tensile strength, as well by adding polypropylene in concrete matrix, the failure mode observed to be more gradual and specimens did not separate in two parts as shown in Figure 5. Effect of using silica fume and rice husk ash as a replacement of ordinary cement on split tensile is similar to the effects that observed during compressive strength tests.

**Figure 4. Variation of Tensile strength with PP and EPS content of concrete**

**Figure 5. Effect of using polypropylene in concrete on failure mode**

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![Graph](image1.png)

**Figure 6. variation of Modulus of Elasticity with PP and EPS content of concrete**

The results also indicated that adding silica fume to admixture caused an increase in modulus of elasticity in all mixes, however using rice husk ash as a supplementary cementitious material caused a decrease in value of the modulus of elasticity at 28 days.

**Flexural behavior:** from the results, the flexural capacity decreased with an increase in the volume of EPS in mixes. Also no effect on the flexural behavior was observed. However, by increasing used polypropylene’s volume, flexural capacity was observed to be improved.

![Graph](image2.png)

**Figure 7. variation of Flexural strength with PP and EPS content of concrete**
3. CONCLUSION
The mechanical strength of EPS concrete showed a linear increase with an increase in concrete density. The failure was observed to be gradual (compressible), and the specimens were capable of retaining the load after failure, without full disintegration. The strength of EPS concretes appears to increase linearly with an increase in concrete density, or with a decrease in the EPS volume. With or without using of EPS in mixes, the addition of polypropylene fibers in the concrete did not significantly affect the compressive strength of concretes. The rate of strength gain was increasing with using silica fume as a replacement of ordinary cement. But rice husk ash, as a replacement of ordinary cement needs more time to show its benefits as a pozzolanic material. In this case, an improvement in mechanical properties can be observed at the age of 90 days and upper. Be seen that the tensile strength increased with an increase in compressive strength. Modulus of Elasticity of concretes decreased with the incorporation of EPS. High amounts of EPS contents, decreased the elastic module more. Effect of polypropylene fibers on module of elasticity was not clear. Adding silica fume to admixture increased the modulus of elasticity however using rice husk ash had not positive effect in early ages. Results of flexural behavior test showed that, the flexural strength decreased with an increase in the volume of EPS in mixes. Results showed that application of PP fibers improved the flexural strength of concrete.

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DAMPER CONCRETE FOR SEISMIC WAVES

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ABSTRACT
When an earthquake happens, different kinds of compression and shear waves are produced. Structures in direction of compression waves P (sense's weight of structure) have good enforcement and main loss of structures is in sense of shear (sense of propagation S waves). By reducing amplitude of seismic waves, we can reduce damages to the building. Nowadays, one of the most important difficulties in front of engineering structure is finding ways for reducing side movement of structure and vibrations caused by the device measuring earthquakes in buildings especially high structures for adding to their factor of safety. Seismic waves are among mechanic waves and for propagation need material environment and their reduction has opposite environment's density. So, reducing density of propagation environment is one of ways for reducing amplitude of waves. On the other side, liquids do not have a stiffness shear and cannot effect and propagation S waves. So, in this plan, we have tried using this specifications and installing sphere full of fluid in the sample environment of ordinary concrete, make new concrete and evaluate its behavior so that has considerable reduce against shear waves.

For considering the made sample behavior and comparing it with ordinary concrete, cubic samples with sizes 10x20x20 and 10x20x60cm are made and were put under impulse loads. Comparing test results, it was observed that amplitude of registered wave on the concrete sample with fluid compare to sample of ordinary concrete has reduced on average %50. So, by using this damping concrete, we can reduce the seismic energy and acceleration effected to the structure and considering amount of this reduction, the structure weight is also reduced. Reduction of structure weight caused reduction of secondary seismic force.

Keywords: damper, shear wave, fluid

1. INTRODUCTION
Buildings are always subject to great dynamic loads which are caused by different environment factors, and the seismic load caused by earthquake is one of them. So, today one of the most important problems in front of structure engineers is finding ways for reducing side movement of structure and produced vibrations in buildings especially high structures for increasing the structure's factor of safety, ease and tranquility of inhabitants. Nowadays, in some buildings of advanced countries such as Japan and USA, passive dampers are used and they have different kind so bracing rod damper and pillar damper. You can see some pictures of the most
useful ones in picture No. 1. Also, other kinds of dampers such as Viscous Damper (VEDs) and the one which can be adjusted for its weight (TMDs)… exist and by changing ductility of structure (R), it causes reduction of vibrations in the time unit or earthquake's rock. Using dampers is the base of reducing the seismic energy after entrance to structure. So, in this method, all changes are mechanic and do not have effect on nature of entrance waves. And, the seismic acceleration is exactly effected to the structure and is subject to dampers which constitute part of structure, are a mortised. Although reducing R has effect in ductility, but another effective parameter is the structure weight which is independent of R. using the present dampers, for every structure, considering its weight, damping should be calculated separately. But, in this plan, we tried considering the nature of seismic waves, as much as effective acceleration on the structure is reduced, we can say definitely that we will have as much as force reduction for structure that has made a lighter structure design and this again will lead to reducing final shear force.

2. SEISMIC WAVES
When an earthquake happens, different types of body waves are produced which are mainly mechanic and are propagation only in material environments. These waves are produced in the center of earthquake and are distributed in all directions.
They are two main groups: compression waves (P) and shear waves (S). In picture No. 2, way of production and propagation of these waves is observed. The first group which is primary, compression and length waves cause compression and connection of materials which pass through them. Second group is secondary; shear or width waves cause vibration of environment particles in vertical line on distributing waves and perform shear waves on the distribution environment. Velocity of waves is subject to material and hardness of composed materials in the distributed environment and the steady fluid which does not have stiffness shear can not effected and distribute shear waves.

3. DAMPER CONCRETE
This concrete considering specifications of the propagation environment of shear waves and in order to amortize more these waves is made. Since sense of propagation compression waves is in line with the structure gravity and structures for effect this force in sense vertical are very rigidity designs, so that compression waves of the earthquake often are not serious damage. But, shear waves while happening of earthquake cause the most damages. But, if we can make shear waves before entrance to damped structure, we can reduce favorably the caused damages by earthquake. In this case, a lighter structure will have stability against earthquake. This in return causes reduction of the force to structure. If this cycle continues, we can achieve the below diagram is obtained. Some sentences of it are economical from engineering viewpoint. It means that we can do this several times and this cycle is effective depending on the structure weight.

\[ F_n = a Q^n - 1 \]

Fn: Reduced earthquake force
A: Primary seismic force
Q: Amount of reduced seismic force in percent
N: No. of sentences

In making the sample concrete, sphere full of water have low shear resistance are available easily and are used as vertically sheets. Picture No. 3 shows a sample of these spheres. Shear waves of earthquake while passing these spheres because of their fluid are reduced. Amount of this reduction is subject to specifications and the amount of fluid inside the concrete and also the way of putting spheres in the concrete.

4. PERFORMING TESTS
Considering hypotheses and theories presented in the preface, we expect that waves and especially shear waves by passing this concrete are reduced. In this concrete, quite equal spheres by passage of time do not exit these spheres so that fluids are preserved for longer period in the environment. For primary considerations, effectiveness of the innovative concrete, at first a sample 20 x 20 x 10 cm is made and necessary considerations are made.
Since the primary sample was cubic, one side of the cubic is chosen and a pounding device for impact shown in picture No. 4 is used for making waves.

Since our aim is comparing reduction of wave's amplitude in the made concrete sample and also waves amplitude in addition to distribution environment is the size is impact, the pounding device system is so made that it has equal conditions in all impact. Besides, by repeating the test and taking average, we tried to reduce the fault of these tests. Also, another parameter which is determining is time's impact. Impact should be in a way which is equal in time. So, in every test, equal impact with equal time intervals are used and in processes of data equal time window are considered. In these considerations on cubic samples, we observed that these spheres as expected by theory can cause amortization of shear waves and amplitude of vibrations is reduced. These changes are observed in picture No. 5-A and 5-B. Since the primary sample was small, a second sample with sizes 60 x 20 x 10 were made, and results of the test were repeated.
During tests, two samples were used so that a suitable comparison is made in produced waves. I mean one sample of ordinary concrete and a sample of damping concrete in which fluid sphere existed. Difference of these samples is the spheres. And from other viewpoints such as: mixing design, amount of cement, type, grading of gravel and sand est. are considered the same. Before performing tests, it was necessary to get insured of producing shear waves by the pounding system. So, the test was done according to the picture No. 6. In this test, two horizontal single element sensors in two vertical lines were installed. And, from the side, we impact the samples. We expected that considering direction of the impact, the produced waves are shear; also the arrived polarity is changed. Results of this test shown in picture No. 7 confirm this hypothesis.
Continuing the tests, as observed in picture No. 8, hits are produced from side and instead of using geophone, horizontal device of systems measuring acceleration are very sensitive. 20 impacts are equally impact on the sample. The reason for using 20 impacts is getting average and reducing fault of impact size.

Figure 8. Testing in different environment.

In processing registered data, seismosignal is used and for omitting it, first it should pass through a filter in frequency limit of 1 to 25 Hertz. Considering acceleration response spectrum in damping %5, %10, and % 20 evaluation of data pseudo acceleration are received. Previous amount of them for both samples of concrete are measuring and reducing amplitude waves is got. And, by getting this ratio, we can say that the sample of produced concrete how much has role in reducing the amplitude and as result energy of seismic waves. Table No. 1 shows this ration in 10 samples tested for ordinary concrete and damping concrete.

<table>
<thead>
<tr>
<th>Test number</th>
<th>Awc (g)</th>
<th>Acc (g)</th>
<th>Awc/Acc ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Max acceleration in damping concrete</td>
<td>Max acceleration in typical concrete</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>0.9</td>
<td>2.6</td>
<td>0.35</td>
</tr>
<tr>
<td>2</td>
<td>1.3</td>
<td>2.4</td>
<td>0.54</td>
</tr>
<tr>
<td>3</td>
<td>0.55</td>
<td>2.3</td>
<td>0.24</td>
</tr>
<tr>
<td>4</td>
<td>0.55</td>
<td>2.3</td>
<td>0.24</td>
</tr>
<tr>
<td>5</td>
<td>1.25</td>
<td>2.8</td>
<td>0.45</td>
</tr>
<tr>
<td>6</td>
<td>1.15</td>
<td>2.3</td>
<td>0.50</td>
</tr>
<tr>
<td>7</td>
<td>0.95</td>
<td>2.6</td>
<td>0.37</td>
</tr>
<tr>
<td>8</td>
<td>1.25</td>
<td>2.4</td>
<td>0.52</td>
</tr>
<tr>
<td>9</td>
<td>1.2</td>
<td>1.8</td>
<td>0.67</td>
</tr>
<tr>
<td>10</td>
<td>1.05</td>
<td>1.8</td>
<td>0.58</td>
</tr>
</tbody>
</table>
Figure 9. Acceleration domain, speed and changes of recorded movement resulting from impacts, respectively A) before and B) after filter

Figure 10 is the response spectrum of amplitude figure of shear wave acceleration in the ordinary and damper concrete in 1.5 second compared together. In one test, amount of acceleration in damped concrete is less than ordinary concrete. And, in the figure of picture No. 11 the reduction ration of acceleration for 10 tests is shown.

Figure 10. The comparison of acceleration response spectrums for section wave in damper and normal concrete- blue for normal concrete and balk & white for damper concrete
As observed in pictures 10 and 11, using this damper concrete, we can seismic energy can be affected to structure. And, considering this amount of structure reduction is reduced. Also, reduction of structure weight causes secondary reduction of seismic force. This concrete reduces the wave acceleration directly and causes reduction of seismic force and reduces the structure weight indirectly. Later on, we intend to consider other parameters such as: dimensions, fluid volume, kind of fluid, arrangement of sphere and entrance of unit energy to the sample so that we achieve a reduction factor.

5. CONCLUSIONS
Since effect of spheres in small dimensions is considerable, we hope that by considering more, we can reduce seismic waves and consequently earthquake danger considerably. And, by reducing human losses, consumption of building materials, structure weight and seismic force are reduced. This is a useful method for reducing the seismic waves' intensity before entrance to structures.

ACKNOWLEDGEMENT
At the end, we would like to express our gratitude to Building and Housing Research Center BHRC presidency for providing the test facilities and engineer Mirzaei, management of center of controlling strong motion network of (BHRC) for his kind supports.
APPLICATION OF HYBRID FIBER REINFORCEMENT AND HIGH VOLUME COARSE FLY ASH IN SELF COMPACTING CONCRETE

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ABSTRACT
Self compacting concrete is termed as a concrete with high flow ability and cohesiveness which can fill its mold without the need of any extra vibration effort. Fiber inclusion to concrete enhances the mechanical properties, while making the concrete less workable. This article presents a study on the fresh and mechanical properties of a fiber reinforced self compacting concrete incorporating high-volume fly ash that does not meet the fineness requirements of ASTM C 618. A poly carboxylic based superplasticizer was used in combination with a viscosity modifying admixture. In mixes containing fly ash, 50% of cement by weight was replaced with fly ash. Two different types of steel fibers were used in combination, keeping the total fiber content constant at 60 kg/m³. Slump flow time and diameter, V funnel, and air content were performed to assess the fresh properties of the concrete. Compressive strength, split tensile strength, and ultrasonic pulse velocity of the concrete were determined for the hardened properties. It can be concluded that high-volume coarse fly ash could successfully be used in producing fiber reinforced SCC. Even though there is some reduction in the concrete strength, because of the use of high-volume coarse fly ash, it is possible to achieve self compaction with considerable fiber inclusion.

Keywords: self compacting concrete, fiber reinforcement, high volume coarse fly ash, fresh properties, ultrasonic test

1. INTRODUCTION
Use of self compacting concrete (SCC) in the construction industry has been increasing [1] because of its technical advantages such as flowing through the reinforcement and filling every corner of its mold without any need for vibration and compaction during its placement. Generally, SCC is achieved using new generation superplasticizers to reduce the water–binder ratio. In addition, supplementary cementitious or inert materials such as limestone powder, natural pozzolans, and fly ash is also used to increase the viscosity and reduce the cost of SCC. Among these materials, fly ash, a by-product of thermal power plants, has been reported to improve the mechanical properties and durability of concrete when used as a cement replacement material [2]. Concretes having large amounts
of fly ash are termed as high-volume fly ash (HVFA) concrete. HVFA concrete was initially developed for mass concrete applications to reduce the heat of hydration, but with its sufficient mechanical and excellent durability properties it has been used in structural and pavement applications [3]. Fly ash is usually separated at the power plants and high quality (fine) fly ash meeting the fineness requirement of ASTM C 618 can be used in producing blended cements or added as a separate ingredient at the ready mixed concrete batching plants. In addition to this fine fly ash, there are vast amounts of substandard (coarse) fly ash that can be utilized in the concrete industry. A successful application of the coarse fly ash in producing blended Portland cements was published by the researchers at CANMET [4]. Fly ash has also been increasingly used in the Turkish concrete industry. Recently, to increase the use of fly ash, investigations on HVFA in producing SCC are being performed [5]. In this article another application of this type of coarse fly ash will be presented on SCCs incorporating hybrid fiber reinforcement.

The term fiber reinforced concrete (FRC) is defined by ACI 116R, Cement and Concrete Terminology, as a concrete containing dispersed randomly oriented fibers. Inherently, concrete is brittle under tensile loading and mechanical properties of concrete may be improved by randomly oriented short discrete fibers which prevent or control initiation, propagation, or coalescence of cracks [6]. The character and performance of FRC changes depend on the properties of concrete and the fibers. The properties of fibers that are usually of interest are fiber concentration, fiber geometry, fiber orientation, and fiber distribution. Using a single type of fiber may improve the properties of FRC to a limited level. However, the concept of hybridization, adding two or more types of fiber into concrete, can offer more attractive engineering properties as the presence of one fiber enables more efficient utilization of the potential properties of other fibers [7-8]. Previous investigations showed that the use of steel fibers in SCC is feasible [9-10]. In these mixes, steel fibers can decrease workability of SCC as the fiber amount and slenderness ratio (length/diameter) increase. However, in case of well-proportioned SCC the workability is not influenced by the steel fibers [10]. The incorporation of fibers in concrete improves mechanical properties of concrete such as ductility, toughness, tensile strength, impact resistance and fatigue.

The objective of this study is to assess the effects of HVFA replacement on the fresh and hardened properties of SCCs incorporating different types of steel fibers. Moreover, the fly ash used in this study was a coarse fly ash that does not meet the fineness requirements of ASTM C 618. Even though, the suitability of using such a substandard fly ash needs much detailed investigations, this study covers the fresh and some hardened properties of such mixes. In addition to the fly ash, two different sizes of steel fibers were used at different proportions in making the concrete. Total mass of cementitious materials is 500 kg/m³, in which 50% of cement is replaced by the coarse grained fly ash. For comparison, a control SCC mix without any fly ash was also produced. The commercially available chemical admixtures used in this study included a viscosity modifying admixture (VMA) and a polycarboxylic based superplasticizer (SP).
2. MATERIALS
2.1. Portland Cement
The cement used in all mixes was a commercially available Portland cement (PC), which corresponds to ASTM Type I cement. It had a specific gravity of 3.09 and Blaine fineness of 3030 cm²/g. Chemical composition of the PC is given in Table 1.

2.2. Limestone Powder
Limestone powder (LP) was used as a mineral viscosity enhancing admixture. LP was a by product of marble extraction with a CaCO₃ content of 98% and a specific gravity of 2.70. The chemical composition of the limestone powder is also presented in Table 1.

2.3. Fly Ash
A fly ash (FA) from Çayırhan, Turkey was used in this study. Its chemical composition is given in Table 1. The FA had a relatively low specific gravity and Blaine fineness of 2.01 and 2420 cm²/g respectively. The percentage of fly ash retained when wet sieved on a 45-μm sieve was 46. Therefore, this FA failed to meet the fineness requirements of ASTM C 618. To confirm the fineness of the FA, the particle size distribution of the FA was also determined. Figure 1 shows the particle size distribution of the FA, as well as the LP, and PC used in this study. As can be seen from that plot, FA was much coarser compared to both PC and LP.

<table>
<thead>
<tr>
<th>Chemical analyses (%)</th>
<th>Portland cement</th>
<th>Fly ash</th>
<th>Limestone powder</th>
</tr>
</thead>
<tbody>
<tr>
<td>CaO</td>
<td>61.94</td>
<td>11.31</td>
<td>54.97</td>
</tr>
<tr>
<td>SiO₂</td>
<td>18.08</td>
<td>49.55</td>
<td>0.01</td>
</tr>
<tr>
<td>Al₂O₃</td>
<td>5.58</td>
<td>13.34</td>
<td>0.17</td>
</tr>
<tr>
<td>Fe₂O₃</td>
<td>2.43</td>
<td>8.51</td>
<td>0.05</td>
</tr>
<tr>
<td>MgO</td>
<td>2.43</td>
<td>4.10</td>
<td>0.64</td>
</tr>
<tr>
<td>SO₃</td>
<td>2.54</td>
<td>1.70</td>
<td>0.00</td>
</tr>
<tr>
<td>K₂O</td>
<td>0.99</td>
<td>1.99</td>
<td>0.00</td>
</tr>
<tr>
<td>Na₂O</td>
<td>0.18</td>
<td>3.08</td>
<td>0.00</td>
</tr>
<tr>
<td>LOI</td>
<td>4.40</td>
<td>2.74</td>
<td>43.66</td>
</tr>
</tbody>
</table>

2.4. Fiber
Two cylindrical steel fiber types, one with hooked ends (SF1) and one straight (SF2) were used. Their specific gravities were 7.85 and 7.17 respectively. The length and aspect ratio of the SF1 was 30 mm and 55, respectively, compared to 6 mm and 37.5 of SF2. The SF2 fiber was made of high strength steel with a brass coating, which provides it a relatively smooth surface. The total fiber content was kept constant at 60 kg/m³ for all the mixes.

2.5. Aggregates
As for the aggregates, crushed limestone and crushed sand from the same local
source were used. As can be seen from the gradation of the aggregates presented in Table 2, the maximum aggregate size was 19 mm. Both the coarse and fine aggregate had a specific gravity of 2.70, and water absorptions of 0.5 % and 1.2 % respectively.

![Figure 1. Particle size distribution of PC, FA and LP](image)

**Table 2: Aggregate grading**

<table>
<thead>
<tr>
<th>% passing</th>
<th>Sieve size (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coarse</td>
<td>Fine</td>
</tr>
<tr>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>58.6</td>
<td>100</td>
</tr>
<tr>
<td>35.8</td>
<td>100</td>
</tr>
<tr>
<td>0</td>
<td>96.9</td>
</tr>
<tr>
<td>-</td>
<td>85.5</td>
</tr>
<tr>
<td>-</td>
<td>68.3</td>
</tr>
<tr>
<td>-</td>
<td>42.3</td>
</tr>
<tr>
<td>-</td>
<td>17.4</td>
</tr>
<tr>
<td>-</td>
<td>3.7</td>
</tr>
</tbody>
</table>

3. CHEMICAL ADMIXTURES

A polycarboxylic type superplasticizer (SP) was used in all concrete mixes. In addition to the SP a viscosity modifying admixture (VMA) was also used. The properties of both admixtures, as provided by their manufacturers, are shown in Table 3.
Table 3: Properties of chemical admixtures

<table>
<thead>
<tr>
<th>Chemical Admixture</th>
<th>Specific gravity</th>
<th>pH</th>
<th>Solid content (%)</th>
<th>Main component</th>
</tr>
</thead>
<tbody>
<tr>
<td>SP</td>
<td>1.08</td>
<td>5-7</td>
<td>40</td>
<td>polycarboxylic</td>
</tr>
<tr>
<td>VMA</td>
<td>1.00</td>
<td>-</td>
<td>20</td>
<td>dispersed carbohydrate</td>
</tr>
</tbody>
</table>

4. EXPERIMENTAL PROCEDURES

4.1. Mix Proportions

The mix proportions of the mixes are summarized in Table 4. As seen in that table, five concrete mixes are prepared. The two control mixes did not contain any steel fibers. As a binder, one of the control mixes included PC (Control_PC) and the other one had FA replacing 50% by weight of PC (Control_FA). All of the remaining mixes had the same amount of FA as in Control_FA. These were named as FA_SF1, FA_SF1&SF2, and FA_SF2 indicating the type of steel fiber incorporated in the mix. For all the mixes, the total amount of binder (PC + FA), the amount of chemical admixtures, and the amount of LP were all kept constant. Water was added to the mix until the SCC characteristics were observed; therefore, the water/powder ratio was not kept constant and change was observed between 0.35 and 0.44.

Table 4: Mix proportions

<table>
<thead>
<tr>
<th>Mix ID</th>
<th>Water</th>
<th>PC</th>
<th>FA</th>
<th>LP</th>
<th>Aggregate Fine</th>
<th>Coarse</th>
<th>Steel fiber SF1</th>
<th>SF2</th>
<th>SP</th>
<th>VMA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Control_PC</td>
<td>250</td>
<td>500</td>
<td>0</td>
<td>70</td>
<td>905</td>
<td>539</td>
<td>0</td>
<td>0</td>
<td>5.5</td>
<td>6.25</td>
</tr>
<tr>
<td>Control_FA</td>
<td>230</td>
<td>250</td>
<td>250</td>
<td>70</td>
<td>888</td>
<td>529</td>
<td>0</td>
<td>0</td>
<td>5.5</td>
<td>6.25</td>
</tr>
<tr>
<td>FA_SF1</td>
<td>226</td>
<td>250</td>
<td>250</td>
<td>70</td>
<td>889</td>
<td>530</td>
<td>60</td>
<td>0</td>
<td>5.5</td>
<td>6.25</td>
</tr>
<tr>
<td>FA_SF1&amp;SF2</td>
<td>205</td>
<td>250</td>
<td>250</td>
<td>70</td>
<td>925</td>
<td>550</td>
<td>30</td>
<td>30</td>
<td>5.5</td>
<td>6.25</td>
</tr>
<tr>
<td>FA_SF2</td>
<td>205</td>
<td>250</td>
<td>250</td>
<td>70</td>
<td>924</td>
<td>549</td>
<td>0</td>
<td>60</td>
<td>5.5</td>
<td>6.25</td>
</tr>
</tbody>
</table>

4.2. Preparation and Casting of Test Specimens

The mixes were prepared at about 5 min. with a 70-liter rotating planetary mixer. The sand, coarse aggregate and fibers were first dry-mixed followed by the addition of fine materials and 1/3 of water. Finally, water and chemical admixtures were pre-mixed and added to the mix. After the mixing procedure was completed, tests were conducted on the fresh concrete to determine slump flow time and diameter, V-funnel flow time, and air content. Segregation and bleeding were visually checked during the slump flow test and was not observed in any of the mixes. From each concrete mix, six 150-mm cubes and six 100*200-mm cylinders were cast. All specimens were cast in one layer without any compaction. The cubes were used for the compressive strength and ultrasonic pulse velocity tests and the cylinders were used for the splitting tensile strength tests. After demolding, all specimens were stored in a curing room at 21±2 OC, and 95±5% relative humidity until testing.
4.3. Tests on Fresh Concrete

Deformability and viscosity of fresh concrete is evaluated through the measurement of slump flow time and diameter, and V-funnel flow time (Figure 2). The slump flow is used to assess the horizontal free flow (deformability) of SCC in the absence of obstructions. The procedure for the slump flow test and the commonly used slump test are almost identical. In the slump test, the change in height between the cone and the spread concrete is measured, whereas in the slump flow test the diameter of the spread concrete is determined as the slump flow diameter (D). According to Nagataki and Fujiwara, a slump flow diameter ranging from 500 to 700 mm is considered as the slump required for a concrete classified as SCC [11].

According to Specification and Guidelines for SCC prepared by EFNARC (European Federation of National Trade Associations), a slump flow diameter ranging from 650-800 mm can be accepted for SCC [12]. In the slump flow test concrete’s ability to flow and its segregation resistance can also be measured. To measure these properties, the time (t50) it takes for the concrete to reach a 500 mm spread circle and any segregation border between the aggregates and mortar around the edge of spread are recorded. EFNARC suggests t50 of 2 to 7 sec. for SCC. In addition to the slump flow test, V-funnel test is also performed to assess the flowability and stability of the SCC. The funnel is filled completely with concrete and the bottom outlet is opened, allowing the concrete to flow. The V-funnel flow time is the elapsed time (tV-f) in seconds between the opening of the bottom outlet and the time when the light becomes visible from the bottom, when observed from the top. Good flowable and stable concrete would consume short time to flow out. According to Khayat, a tV-f which is less than 6 sec. is recommended for a concrete to qualify as a SCC [13]. According to EFNARC, tV-f ranging from 6 to 12 sec. is considered adequate for a SCC [12].

![Figure 2. Workability tests on the HVFA-SCC](image)

4.4. Tests on Hardened Concrete

Tests performed on cured concrete specimens consist of the specimen compressive strength, the splitting tensile strength, and the ultrasonic pulse velocity. For each mix, cubic specimens were loaded under compressive load to failure (ultimate
load) at 28, and 56 days. The compressive strength was computed from the average of three specimens. The ultrasonic pulse velocities (UPV) of all six cubic specimens were measured on the two smooth sides of the specimen at 7, 14, 28, and 56 days. The UPV test was conducted with direct transducer arrangement using a pair of narrowband 54 kHz transducers using a commercially available PUNDIT system.

5. DISCUSSION OF TEST RESULTS
5.1. Fresh Concrete Properties
Table 5 lists the test results performed on fresh concrete. Included in that table are the w/p ratio of the mix, slump flow diameter (D) and time (t50), V-funnel flow time and air content. As seen in that table, the slump flow diameters of all mixes were in the range of 560 to 700 mm, slump flow times are less than 2.9 sec., and the V-funnel flow times (tV-f) were in the range of 2.4 to 4.3 sec. Therefore, all concrete mixes could be considered as SCC. In all of the SCC mixes, there was no segregation of aggregate near the edges of the spread-out concrete as observed from the slump flow test.

<table>
<thead>
<tr>
<th>Mix ID</th>
<th>w/p*</th>
<th>Fiber factor</th>
<th>Slump flow</th>
<th>V-Funnel flow time</th>
<th>Air Content</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>D (mm)</td>
<td>t50 (s)</td>
<td>tV-f (s)</td>
</tr>
<tr>
<td>Control_PC</td>
<td>0.44</td>
<td>0</td>
<td>560</td>
<td>2.9</td>
<td>4.3</td>
</tr>
<tr>
<td>Control_FA</td>
<td>0.40</td>
<td>0</td>
<td>690</td>
<td>&lt; 2.0</td>
<td>2.4</td>
</tr>
<tr>
<td>FA_SF1</td>
<td>0.40</td>
<td>42</td>
<td>660</td>
<td>&lt; 2.0</td>
<td>2.8</td>
</tr>
<tr>
<td>FA_SF1&amp;SF2</td>
<td>0.36</td>
<td>35</td>
<td>630</td>
<td>&lt; 2.0</td>
<td>4.0</td>
</tr>
<tr>
<td>FA_SF2</td>
<td>0.36</td>
<td>29</td>
<td>700</td>
<td>&lt; 2.0</td>
<td>2.7</td>
</tr>
</tbody>
</table>

Also observed in Table 5 is the change in w/p ratio for the same workability measure, i.e. the same D, t50, and tV-f. The Control_PC mix had the highest w/p ratio, but as part of the PC was replaced by FA the w/p ratio of all mixes decreased. This phenomenon is also observed by other researchers [4, 14]. In such studies, even though finer FAs were used, which is expected to increase the water requirement of a concrete mix, the smooth surface characteristics and spherical shape of the FA improved the workability characteristics of concrete mixes and the same workability was achieved by a smaller w/p ratio. Therefore, using a coarser FA with higher volumes is naturally going to decrease the water demand of a SCC mix for the same workability measure. The steel fibers also affected the fresh properties of the concrete mixes. The addition of SF1 type steel fibers did not affect the water requirement of the mix for the same workability. However, addition of SF2 type fibers which have smaller diameters and sizes reduced the amount of water. This could be explained by the geometry of the fibers as well as the surface characteristics of these fibers. SF2 fibers have smaller dimensions when
compared with SF1 fibers, thus have less potential to prevent the movement of aggregates. In addition, SF2 fibers are coated with brass and have very smooth surfaces, which reduce the energy loss during the movement of particles.

5.2. Hardened Concrete Properties

The results of hardened concrete tests are presented in Table 6. Included in that table are the 28 and 56 day compressive and splitting tensile strength tests and 7, 14, 28, and 56 day ultrasonic pulse velocity tests. Even though the w/p ratio of the mix was reduced, substitution of PC with a coarse FA resulted in lower strengths both at 28 and 56 days. This reduction was 43% at 28 days and 31% at 56 days. The low pozzolanic activity can be attributed to the coarseness of the FA used. Fiber inclusion did not significantly affect the measured mechanical properties; however, as seen in Figure 3 as the volume of the SF2 type fibers increased the compressive strength slightly increased. This is due to the relatively small dimensions of SF2 type fibers, which give these fibers the ability to delay the micro crack formation and to arrest and prevent their propagation afterwards up to a certain extent. Another explanation to the increase in the compressive strength could be the decrease in w/p ratio which decreased as the amount of SF2 type fibers increased. However, when the split tensile strengths are examined (Figure 4) it can be seen that there is a reduction in the split tensile strengths as the volume of SF2 type fibers are increased or the w/p decreased. The reduction in the split tensile strength is explained by the loss of the presence of longer SF1 type fibers which are responsible for the increase in tensile strengths.

**Table 6: Hardened properties**

<table>
<thead>
<tr>
<th>Mix ID</th>
<th>Compressive Strength (MPa)</th>
<th>Split Tensile Strength (MPa)</th>
<th>Ultrasonic Pulse Velocity (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>28 d* 56 d*</td>
<td>28 d* 56 d*</td>
<td>7 d† 14 d† 28 d† 56 d*</td>
</tr>
<tr>
<td>Control_PC</td>
<td>40.7 70 41.7 3.58 3.68</td>
<td>4565 4570 4578 4609</td>
<td></td>
</tr>
<tr>
<td></td>
<td>[0.5] [0.4] [0.1] [22]</td>
<td>[45] [35] [37]</td>
<td></td>
</tr>
<tr>
<td>Control_FA</td>
<td>23.3 28.6 2.8 3.34</td>
<td>4161 4200 4436 4564</td>
<td></td>
</tr>
<tr>
<td></td>
<td>[1.0] [1.2] [0.4] [0.5]</td>
<td>[31] [51] [74]</td>
<td></td>
</tr>
<tr>
<td>FA_SF1</td>
<td>19.6 24.5 3.10 3.69</td>
<td>3963 4007 4157 4317</td>
<td></td>
</tr>
<tr>
<td></td>
<td>[0.1] [0.6] [0.2] [0.6]</td>
<td>[10] [44] [45]</td>
<td></td>
</tr>
<tr>
<td>FA_SF1&amp;S_F2</td>
<td>22.8 26.1 3.40 3.82</td>
<td>3970 4100 4249 4383</td>
<td></td>
</tr>
<tr>
<td></td>
<td>[0.5] [2.0] [0.0] [0.3]</td>
<td>[42] [41] [40]</td>
<td></td>
</tr>
<tr>
<td>FA_SF2</td>
<td>22.5 31.8 3.08 3.23</td>
<td>4142 4224 4359 4506</td>
<td></td>
</tr>
<tr>
<td></td>
<td>[2.9] [0.8] [0.1] [0.2]</td>
<td>[109] [109] [82]</td>
<td></td>
</tr>
</tbody>
</table>

* Tests are performed on 3 specimens
† Tests are performed on 6 specimens
‡ Numbers in parenthesis are the standard deviations

Ultrasonic pulse velocity (UPV) is used to assess the hardening of the SCC mixes. As seen in Figure 5, as hydration continues the UPVs increased for all the SCC mixes. However, the slope of that curve is quite different for the PC and FA mixes. For the Control-PC mix the slope was much smaller as most of the hydration was
complete by 7 days. However, for the mixes with FA the hydration reactions continue after 7 days indicating a higher slope.

Figure 3. Effect of steel fibers on the compressive strength

Figure 4. Effect of steel fibers on the split tensile strength

Figure 5. Strength gain of SCC mixes
6. CONCLUSIONS
This paper discusses the part of the results of an experimental program carried out to investigate the effects of incorporation of HVFA, and steel fibers on the flow characteristics of SCC and mechanical properties in the hardened state.
It can be concluded that it is possible to achieve self compaction with considerable fiber inclusion. Incorporation of HVFA may reduce the water requirement of a SCC mix. In other words, using high volumes of coarse FA may increase the workability characteristics of SCC mixes. Therefore the amount of SP and VMA to achieve self compaction could be reduced with proper adjustments to the FA amount. However, it is also seen that using coarse FA may cause significant strength losses to the SCC mixes, since they are used in high volumes. However, the strength reduction due to low pozzolanic activity of the FA was partially offset by the use of smaller SF2 type steel fibers. It can also be concluded that the SF2 type steel fibers affect the properties of SCC mixes not only in the hardened state but also in the fresh state reducing the water requirement for the same workability measure.

ACKNOWLEDGEMENTS
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REFERENCES
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EFFECTS OF POLYPROPYLENE FIBERS ON PHYSICAL AND MECHANICAL PROPERTIES OF CONCRETES

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ABSTRACT
In this present study, the effects of adding polypropylene fibers on physical and mechanical properties of concretes are investigated. To this end, three concrete mixtures consisting of 6 mm, 12 mm and 19 mm polypropylene fibers are made and their physical/mechanical aspects are studied and compared with control concrete. The results manifest that adding polypropylene fibers increases the flexural strength slightly and decreases the cracks width. Besides, the compressive strength decreases slightly. These properties are improved with increase of fibers length.

Keywords: concrete, polypropylene fibers, crack bridge, flexural strength, impact resistance

1. INTRODUCTION
Nowadays concrete is one of the most applicable materials in construction of structures such as buildings, dams, bridges, tunnels, highway pavements, offshore structures, towers and so on. This material has received great attention because of its desirable performance in compression. Concrete is considered to be a relatively brittle material, so it is prone to cracking. Many investigations have been carried out in order to overcome this problem. The inclusion of adequate fibers improves tensile strength and provides ductility [1-3]. There are more investigations on the effects of different fibers on concrete properties [4-9].

Some of the important effects of fibers in concrete are: increasing the tensile strength, preventing the crack development and increasing the toughness of concrete. The fundamental advantage of adding fibers to concrete is known as crack bridging [9-14].

In recent years, concrete containing different fibers has been applied in large structures such as highway pavements and airports, huge foundations with large deformations and concrete cover of tunnels. Recently in order to prevent cracking in the covers of the pre-cast tunnels, un-reinforced concrete with the fibers has been used. On the other hand the investigations have shown the compressive strength reduction in fiber concretes. This reduction occurs because of the collection of Calcium-Hydroxide in the interface of hydrated cement and various types of fibers (such as Steel, Carbon, Dacron, Polypropylene fibers, and …) [15].

In recent decades the polypropylene fibers have been widely used in industries. Polypropylene fibers are relatively inexpensive, easy to split into finer sizes,
durable in the environment of cement matrix and they don’t rust. They have a relatively low modulus of elasticity, relatively poor bond and it is difficult to obtain uniform dispersion with Polypropylene fibers when a sufficiently large volume of fibers is used. In the present study, the effects of adding polypropylene fibers on physical and mechanical properties of concretes are investigated.

2. EXPERIMENTAL PROGRAM
2.1. Materials
2.1.1. Aggregates
Crushed coarse aggregates with maximum nominal size of 19mm and natural fine aggregates were selected. The Physical properties of coarse and fine aggregates are presented in Table 1. The aggregates grading curve is shown in Figures 1, 2.

<table>
<thead>
<tr>
<th>Aggregates Type</th>
<th>SSD Density* (gr/cm³)</th>
<th>Water absorption (%)</th>
<th>Passing from sieve #200 (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coarse Crushed</td>
<td>2.53</td>
<td>1.61</td>
<td>0.5</td>
</tr>
<tr>
<td>Fine Natural</td>
<td>2.56</td>
<td>2.46</td>
<td>1.1</td>
</tr>
</tbody>
</table>

SSD: Saturated Surface Dry

Figure 1. Particle size distribution of fine aggregates

Figure 2. Particle size distribution of coarse aggregates
2.1.2. Cement
Type II Portland cement (according to ASTM C595) produced by Tehran Cement manufactory, was used in this investigation. The chemical and physical properties of this cement are presented in Table 2.

2.1.3. Polypropylene Fibers
The polypropylene fibers in three sizes of 6, 12 and 19 mm were used. A sample of the fibers is shown in Figure 3.

<table>
<thead>
<tr>
<th>Table 2: Chemical and physical properties of cement</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Chemical analysis, %</strong></td>
</tr>
<tr>
<td>Calcium oxide (CaO)</td>
</tr>
<tr>
<td>Silica (SiO₂)</td>
</tr>
<tr>
<td>Alumina (Al₂O₃)</td>
</tr>
<tr>
<td>Iron oxide (Fe₂O₃)</td>
</tr>
<tr>
<td>Magnesia (MgO)</td>
</tr>
<tr>
<td>Sodium oxide (Na₂O)</td>
</tr>
<tr>
<td>Potassium oxide (K₂O)</td>
</tr>
<tr>
<td>Sulfur trioxide (SO₃)</td>
</tr>
<tr>
<td><strong>Bogue potential compound composition, %</strong></td>
</tr>
<tr>
<td>Tri-calcium silicate (C₃S)</td>
</tr>
<tr>
<td>Di-calcium silicate (C₂S)</td>
</tr>
<tr>
<td>Tri-calcium aluminate (C₃A)</td>
</tr>
<tr>
<td><strong>Other properties</strong></td>
</tr>
<tr>
<td>3 days compressive strength, kg/cm²</td>
</tr>
<tr>
<td>7 days compressive strength, kg/cm²</td>
</tr>
<tr>
<td>28 days compressive strength, kg/cm²</td>
</tr>
<tr>
<td>Initial setting time, min</td>
</tr>
<tr>
<td>Final setting time, min</td>
</tr>
<tr>
<td>Specific surface, cm²/gr</td>
</tr>
</tbody>
</table>

Figure 3. Sample of polypropylene fibers (6, 12 and 19 mm)

2.2. Mix Design
The mixtures were made on the basis of a series of experimental mix parameters
such as suitable slump, lack of segregation and bleeding. The mixture proportions are shown in Table 3. To prevent breaking the fibers, first concrete materials were mixed, and then the fibers were poured in the mixture by hand rapidly in a 1-2 minute period [16].

3. RESULTS AND DISCUSSIONS

3.1. Fresh Concrete
The fresh concrete specifications for each kind of the mixtures are shown in Table 4. The concretes containing polypropylene fibers had lower slump than the control concrete.

<table>
<thead>
<tr>
<th>Mixture identification</th>
<th>W/C</th>
<th>Water (kg)</th>
<th>Cement (kg)</th>
<th>Coarse (kg)</th>
<th>Fine (kg)</th>
<th>Polypropylene fibers (kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>6 mm 12 mm 19 mm</td>
</tr>
<tr>
<td>Control mix</td>
<td>0.5</td>
<td>175</td>
<td>350</td>
<td>860</td>
<td>980</td>
<td>-- -- --</td>
</tr>
<tr>
<td>Mix 1</td>
<td>0.5</td>
<td>175</td>
<td>350</td>
<td>860</td>
<td>980</td>
<td>2 -- --</td>
</tr>
<tr>
<td>Mix 2</td>
<td>0.5</td>
<td>175</td>
<td>350</td>
<td>860</td>
<td>980</td>
<td>-- 2 --</td>
</tr>
<tr>
<td>Mix 3</td>
<td>0.5</td>
<td>175</td>
<td>350</td>
<td>860</td>
<td>980</td>
<td>-- -- 2</td>
</tr>
</tbody>
</table>

Table 4: Fresh concrete specifications

<table>
<thead>
<tr>
<th>Mixture identification</th>
<th>Density (Kg/m³)</th>
<th>Slump (cm)</th>
<th>Air percentage</th>
<th>Observations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Control mix</td>
<td>2400</td>
<td>7</td>
<td>4.5</td>
<td>No bleeding-</td>
</tr>
<tr>
<td>Mix 1</td>
<td>2385</td>
<td>3.5</td>
<td>4</td>
<td>No segregation</td>
</tr>
<tr>
<td>Mix 2</td>
<td>2380</td>
<td>3</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>Mix 3</td>
<td>2380</td>
<td>3</td>
<td>4</td>
<td></td>
</tr>
</tbody>
</table>

3.2. Hardened Concrete
In order to determine the physical/mechanical properties of mixtures, tests of compressive strength, flexural strength, modulus of elasticity, abrasion resistance, impact resistance and shrinkage were performed.

3.2.1. Compressive Strength
The compressive strength of cube specimens was obtained according to the BS 1881 at the ages of 7, 28 and 56 days. The results are presented in Figure 4. It can be observed that the compressive strength of fiber concretes is less than the control concrete. This strength reduction can be induced by collection of Calcium-Hydroxide in the interface of fibers and hydrated cement. Besides, the compressive strength increased with increase of fibers length. The compressive strength of 19 mm fibers concretes is almost equal to the control concrete.

3.2.2 Flexural Strength
The flexural strength of the specimens was measured according to ASTM C293 at the ages of 7, 28 and 56 days. The results are shown in Figure 5.
As can be seen, the use of fibers increases the flexural strength of the concrete. This increasing trend may have occurred due to crack bridging of the fibers. Besides, the flexural strength increased with an increase in the length of the fibers. It can be concluded that longer fibers (with higher aspect ratio) can bridge the cracks better than other fibers.

![Compressive strength versus age](image1)

**Figure 4. Compressive strength versus age**

![Flexural strength versus age](image2)

**Figure 5. Flexural strength versus age**

### 3.2.3 Modulus Of Elasticity
The modulus of elasticity of the specimens was determined at the ages of 7, 28 and 56 days according to ASTM C469 and the results are shown in Figure 6. The use of fibers decreases the static modulus of elasticity of the concrete slightly. Besides, the increase in the length of the fibers causes a slight increase in the static modulus of elasticity. This increase may have occurred because of increase in the
3.2.4. Abrasion Resistance

The abrasion resistance of the specimens was measured at age of 28 days according to standards ASTM C779 and EN 1338. The results are shown in Table 5. It can be observed that abrasion resistance of the fiber concretes is better than the control concrete. Besides, the abrasion resistance of the concrete slightly increases with increase in the length of the fibers.

<table>
<thead>
<tr>
<th>Mixture Identification</th>
<th>Abrasion depth according to ASTM C779-89a</th>
<th>Abrasion according to EN 1338</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>After 30 min (mm)</td>
<td>After 60 min (mm)</td>
</tr>
<tr>
<td>Control mix</td>
<td>0.94</td>
<td>1.68</td>
</tr>
<tr>
<td>Mix 1</td>
<td>0.54</td>
<td>1.09</td>
</tr>
<tr>
<td>Mix 2</td>
<td>0.56</td>
<td>1.05</td>
</tr>
<tr>
<td>Mix 3</td>
<td>0.52</td>
<td>0.98</td>
</tr>
</tbody>
</table>

* Variation has been calculated with respect to after 60 min results

3.2.5. Impact Resistance

One of the important properties of the fiber concretes is the resistance against impact. The test of impact repeat with load drop is one of the valid tests for evaluation of impact resistance of concrete which has been suggested by ACI 544- part 2. In this test, the number of required impacts which causes to crack and rupture in concrete specimen, is determined and this number implies the qualitative estimation of the absorbed energy by the concrete specimen. In this test the standard hammer (with the weight of 4.5 kg and drop height of 457 mm) drops on the steel sphere of 63.5 mm diameter which has been fixed on the surface of the
concrete, and transfers the impact to the concrete specimen. The specimen shape is cylindrical with 152 mm diameter and 63.5 mm height. The results are shown in Table 6 at the age of 28 days.

Adding fibers to concrete decreases the required number of impacts due to first crack, while it increases the required number of the impacts due to complete rupture. The number of impacts until complete rupture is also increased with the increase of the length of the fibers.

<table>
<thead>
<tr>
<th>Mixture Identification</th>
<th>Required number of impacts due to first crack</th>
<th>Required number of the impacts due to complete rupture</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Minimum impacts</td>
<td>Maximum impacts</td>
</tr>
<tr>
<td>Control mix</td>
<td>21</td>
<td>25</td>
</tr>
<tr>
<td>Mix 1</td>
<td>19</td>
<td>21</td>
</tr>
<tr>
<td>Mix 2</td>
<td>16</td>
<td>18</td>
</tr>
<tr>
<td>Mix 3</td>
<td>16</td>
<td>18</td>
</tr>
</tbody>
</table>

3.2.6. Restrained Shrinkage Test

If the shrinkage of concrete occurs freely, the concrete section doesn’t crack, but if it is restrained, the tensile stresses will appear and the concrete becomes more prone to cracking. One of the effective methods of controlling restrained shrinkage cracking is the use of fibers in the concrete mixture. In this research, in order to evaluate the cracking potential caused by restrained shrinkage, the method of cracking in circular specimens which is suggested by ACI 544- part 2 is used. In this test, concrete is molded into a ring formwork with the thickness of approximately 30mm and of 300mm outer diameter. The specimen is subjected to wind blowing and low relative humidity and the procedure of crack development is monitored. A sample of this test is shown in Figure 7. In this test, width of the cracks and the pattern of cracks from the time of cracking were studied up to 90 days. The results of restrained shrinkage up to 90 days are shown in Table 7.

As can be observed, fiber concrete mixtures have less crack width in comparison with control concrete. The time of the first cracking in the fiber concrete has also increased. These results confirm results of the flexural strength test about fiber’s crack bridging.

Figure 7. Crack in restrained ring specimen drying from the top and bottom [14]
Table 7: Observed results of restrained shrinkage cracks up to 90 days

<table>
<thead>
<tr>
<th>Mixture identification</th>
<th>Time of the first cracking after 7 days curing (days)</th>
<th>Maximum crack number</th>
<th>Maximum crack width</th>
<th>Average crack width</th>
</tr>
</thead>
<tbody>
<tr>
<td>Control mix</td>
<td>3</td>
<td>3</td>
<td>0.3</td>
<td>0.27</td>
</tr>
<tr>
<td>Mix 1</td>
<td>4</td>
<td>3</td>
<td>0.26</td>
<td>0.21</td>
</tr>
<tr>
<td>Mix 2</td>
<td>6</td>
<td>2</td>
<td>0.2</td>
<td>0.18</td>
</tr>
<tr>
<td>Mix 3</td>
<td>6</td>
<td>2</td>
<td>0.2</td>
<td>0.17</td>
</tr>
</tbody>
</table>

4. CONCLUSION

From the results of this investigation, the following conclusions can be drawn:
- Adding polypropylene fibers to the concrete has led to a slight decrease in the compressive strength and modulus of elasticity of the specimens. The strength was decreased slightly (up to 10 percent for specimens made of 6 mm length fibers). Besides, the compressive strength has been increased with the increase in the length of the polypropylene fibers. The compressive strength of 19 mm fibers concrete is almost equal to the control concrete.
- According to obtained results of the flexural strength test, adding fibers to concrete has led to increase of flexural strength with respect to the control concrete. Flexural strength also increases with the increase in the length of the fibers. The concrete mixture with 19 mm fibers showed 10 percent higher flexural strength than the control concrete.
- The specimens which contained polypropylene fibers had better abrasion resistance than control concrete. Generally the fibers with different lengths showed equal abrasion resistance.
- Adding fibers to concrete decreased the required number of impacts for the first cracking, but increased it for complete rupture. The number of impacts for complete rupture increases with the increase of the length of the fibers. It can be concluded that adding fibers to concrete has an effective role in reduction of the crack width.
- According to obtained results, polypropylene fibers concretes are useful in control of shrinkage and fine cracks. It is recommended to apply these materials in construction of the concrete floors such as airports and industrial floors.

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CD07-010

OIL PALM SHELL, GROUND PALM OIL FUEL BASED LIGHTWEIGHT CONCRETE

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ABSTRACT
Compressive strength and water permeability of concretes containing ground palm oil fuel ash (GPOA) and ground rice husk–bark ash (GRBA) were investigated. From the tests, the replacement of Portland cement by both materials resulted in the higher water demand in concrete mixtures as compared to ordinary Portland cement (OPC) concrete with compatible workability. The compressive strengths of concretes containing 20% of GPOA and GRBA were as high as that of OPC concrete and were reduced as the increase in the replacement ratios. Although the compressive strengths of concrete with the replacement of GPOA or GRBA up to 40% were lower than OPC concrete, their water permeabilities were still lower than that of OPC concrete. These results indicate that both of GPOA and GRBA can be applied as new pozzolanic materials to concrete with an acceptable strength as well as permeability.

Keywords: oil palm shell, ground palm oil fuel ash, ground rice, lightweight concrete, permeability.

1. INTRODUCTION
Because of environmental problems and considering the rapid depletion of conventional aggregates, the use of aggregates from by-products and solid waste materials from different industries are highly desirable. One such alternative is oil palm shell (OPS), which is a form of agricultural solid waste. The bond behaviour of OPS is one such necessary investigation that has to be clearly established. The bond strength between the concrete matrix and the steel reinforcement is one of the most important aspects in structural reinforced concrete. The investigation involved pullout test on both plain and deformed steel bars under two types of curing conditions. In addition, the durability performance is also another important aspect that determines the viability of OPS concrete to be used in practical applications. The permeability of concrete has a high bearing on the concrete durability as it controls the penetration rate of moisture that may contain harmful or chemicals. The absorption characteristics of a concrete is another means of indicating its durability.

The addition of fillers and pozzolanic materials are introduced to improve the strength and other properties of concrete for necessary conditions. Palm oil fuel ash (POA) is produced from burning of fiber, shell, and empty fruit
bunch of palm oil tree as a fuel to heat the steam for electricity generation and palm oil extraction process. The palm oil fuel ash is so disposed in landfills that the amount of ashes increases every year and now becomes a burden. It is estimated that more than 100,000 tons of palm oil fuel ash has been produced every year and increases annually in Thailand. The study of palm oil fuel ash was started by Tay [1] who used it to replace Portland cement with 10-50%. He found that in the range of 20-50% of cement replacement, the decrease in the compressive strength of concrete at various ages was almost proportional to amount of the ash in the concrete mixtures, except when only 10% ash was used. Later, Awal and Hussin [2] reported that palm oil fuel ash had a good potential in suppressing expansion due to sulfate attack. In 2004, it was found that palm oil fuel ash, which contained a substantial amount of silica and was ground to a suitable fineness, could be used as a pozzolanic material to produce high strength concrete as high as 100 MPa at 90 days [3]. Rice husk-bark ash (RBA) is also a waste from electricity generation power plant. In the fluidized bed power plant, two parts of rice husk are used in conjunction with one part of eucalyptus tree bark by weight as fuel. The burning temperature of the materials is between 800 and 900°C. The disposal of rice husk–bark ash is also becoming a problem due to its quantity. It is estimated that more than 300,000 tons of rice husk–bark ash has been produced each year in Thailand [4]. Effort has, therefore, been made to utilize this ash. The study revealed that the ground rice husk–bark ash conforms to the Class N pozzolanic material [5] as prescribed by ASTM C 618 [6]. Moreover, the compressive strength of mortar containing very high fineness of rice husk–bark ash is equal or higher than the control mortar. Although some properties of concrete containing rice husk–bark ash have been reported [4], none of them deals with a relationship between the strength and the permeability. The aim of this research is to study the compressive strength and water permeability of concrete containing palm oil fuel ash and rice husk–bark ash. The results are compared to concrete containing fly ash, a well-known pozzolanic material, and also compared to the control concrete as well. The knowledge on the strength and permeability of concrete containing palm oil fuel ash and rice husk–bark ash is and could be beneficial on the utilization of these waste materials in concrete work, especially on the topic of durability.

2. EXPERIMENTAL PROGRAM

2.1. Materials

The properties of the sand and OPS used are illustrated in Table 1. It can be observed that the aggregate impact value (AIV) and aggregate crushing value (ACV) of OPS aggregates were much lower compared to the conventional crushed stone aggregates. The chemical composition of OPS was determined and is presented in Table 2. Before the OPS are used as aggregates, they were sieved and only aggregates passing the 12.5mm sieve were used for mixing. The sieve analysis of the river sand and OPS aggregate is illustrated in Figure 1. Due to the high water absorption of OPS, the aggregates were pre-soaked for 24 h in potable water prior to mixing and were in saturated surface dry (SSD) condition during mixing to prevent absorption from occurring during mixing. The properties of OPS
aggregates are different compared to other lightweight aggregates, such as expanded clay, expanded shale and sintered pulverised fuel ash, which are artificially produced.

![Sieve analysis for river sand and OPS aggregate](image)

**Figure 1: Sieve analysis for river sand and OPS aggregate**

<table>
<thead>
<tr>
<th>Properties</th>
<th>River sand</th>
<th>Oil palm shell (OPS)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum grain size (mm)</td>
<td>1.18</td>
<td>12.5</td>
</tr>
<tr>
<td>Shell thickness (mm)</td>
<td>-</td>
<td>0.5-3</td>
</tr>
<tr>
<td>Specific gravity</td>
<td>2.45</td>
<td>1.17</td>
</tr>
<tr>
<td>Bulk unit weight (kg/m³)</td>
<td>1500-1550</td>
<td>500-600</td>
</tr>
<tr>
<td>Fineness modulus</td>
<td>1.40</td>
<td>6.08</td>
</tr>
<tr>
<td>Los Angeles abrasion value (%)</td>
<td>-</td>
<td>4.90</td>
</tr>
<tr>
<td>Aggregate impact value (%)</td>
<td>-</td>
<td>7.51</td>
</tr>
<tr>
<td>Aggregate crushing value (%)</td>
<td>-</td>
<td>8</td>
</tr>
</tbody>
</table>
| 24-h water absorption (%)              | 3.89       | 33                  

**Table 2: Chemical composition of OPS aggregate**

<table>
<thead>
<tr>
<th>Elements</th>
<th>Results (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ash</td>
<td>1.53</td>
</tr>
<tr>
<td>Nitrogen (as N)</td>
<td>0.41</td>
</tr>
<tr>
<td>Sulphur (as S)</td>
<td>0.000783</td>
</tr>
<tr>
<td>Calcium (as CaO)</td>
<td>0.0765</td>
</tr>
<tr>
<td>Magnesium (as MgO)</td>
<td>0.0352</td>
</tr>
<tr>
<td>Sodium (as Na₂O)</td>
<td>0.00156</td>
</tr>
<tr>
<td>Potassium (as K₂O)</td>
<td>0.00042</td>
</tr>
<tr>
<td>Aluminium (as Al₂O₃)</td>
<td>0.130</td>
</tr>
<tr>
<td>Iron (as Fe₂O₃)</td>
<td>0.0333</td>
</tr>
<tr>
<td>Silica (as SiO₂)</td>
<td>0.0146</td>
</tr>
<tr>
<td>Chloride (as Cl⁻)</td>
<td>0.00072</td>
</tr>
<tr>
<td>Loss on Ignition</td>
<td>98.5</td>
</tr>
</tbody>
</table>

The mix design for the OPS concrete in this investigation is based on conducting trial mixes and the proportion of these mixes are adjusted to arrive at an optimum mix proportion, which was used throughout the entire investigation. This mix
proportion consisted of 510 kg/m³ cement, 848 kg/m³ sand, 308 kg/m³ OPS with a water/cement ratio of 0.38. The SP added was 1.4 l per 100 kg of cement. The obtained slump, which was between 50 and 70mm showed that OPS concrete has a medium degree of workability and is within the range of a workable concrete. The air content was in the range of 4.8-5.5% and this relatively high air content could be attributed by the highly irregular shapes of the OPS which prevented full compaction to be achieved.

Ordinary Portland cement (OPC) was used for all concrete mixtures. Palm oil fuel ash and rice husk–bark ash were ground by ball mill until the 95% of the particles passed a sieve No. 325 (opening 45 lm) and were assigned as GPOA and GRBA, respectively. Physical properties and chemical compositions of the materials are shown in Tables 3 and 4.

2.2. Aggregates
River sand with fineness modulus of 2.44 and specific gravity of 2.65 was used as fine aggregate. Crushed limestone with the maximum size of 20 mm and having specific gravity of 2.67 was used as coarse aggregate. 2.2. Concrete mixtures OPC was partially replaced by ground palm oil ash (GPOA) and ground rice husk–bark ash (GRBA) at 20%, 40% and 55%, while the replacement of OFA was 20% and 40% by weight of binder. The binder content of concrete was set as a constant of 300 kg/m³ and mix proportions of concrete are presented in Table 3.

| Table 3: Materials properties                                                                 |
|-----------------------------------------------|---------------------------------|-------------------|
| Materia | Specific gravity | Retained on a sieve #325 (%) | Median particle size, d50 (lm) |
| OPC     | 3.14            | -                              | 14.7               |
| OFA     | 2.19            | 32.1                           | 27.1               |
| GPOA    | 2.43            | 1                              | 8                  |
| GRBA    | 2.15            | 1.9                            | 10.2               |

Note: OPC = Ordinary Portland cement Type I. OFA = Fly ash. GPOA = Ground palm oil fuel ash. GRBA = Ground rice husk–bark ash.

3. TESTING
3.1. Compressive Strength and Structural Bond
Concretes cylinders of 100mm in diameter and 200 mm in height were used to determine the compressive strength. The samples were demolded 24 h after casting and cured in water until the testing ages. The compressive strengths of concretes were determined at the ages of 28 and 90 days.

For each specimen, a single reinforcing bar was placed in the centre of the specimen and both ends of the specimen were provided with an unbonded length of 25mm at each end. The unbonded lengths were provided by attaching a plastic sheathing to the bar for obtaining uniform pressure. The short embedment length of 150mm was selected to avoid yielding of the steel bar under pullout load. For the bond strength determination, two types of curing regimes, namely laboratory air-dry curing (CL) and fullwater curing (CC) were considered. Under both curing
conditions, the specimens were immediately covered with plastic sheets upon casting to prevent excessive evaporation from the fresh concrete and then demoulded after 24 h. In CL curing, specimens were kept in ambient laboratory conditions (RH of 74–88%; temp. of 2573 °C) until the age of test. For CC curing, the specimens were cured in a water tank (water temp. of 2372 °C) until the age of test. The pull-out test was carried out using the Universal Testing Machine (Shimadzu UH-300kN capacity) complete with a modified loading frame. The load was applied on the top of the concrete surface at a uniform rate as per ASTM standards until failure to obtain the ultimate load. Triplicate specimens were prepared for the pull-out test and the bond strength was reported as an average of three tests.

3.2. Water Permeability
The steady flow method was applied to test the permeability of concrete. The coefficient of water permeability was determined by measuring the amount of water passing through the sample and calculated using Darcy’s law and the equation of continuity [7].

Two days before testing the permeability, the samples were prepared by sawing of 40 mm thick slice from the middle of the cylinder. After drying in the laboratory for 24 h, the slice was cast around with 25 mm thick of nonshrinkage epoxy resin to prevent the water leakage. The epoxy resin was allowed to harden and dry for another 24 h. The sample was then installed in the housing cell and then the water pressure of 0.5 MPa was applied. This pressure was recommended and used by Chan and Wu [8]. The time and the amount of water passed through the specimen were monitored until the constant flow rate was obtained.

4. RESULTS AND DISCUSSION
4.1. Properties and Particle Shape of Materials
In Table 4, it revealed that the fly ash and the GRBA can be assigned as class F and class N pozzolan as prescribed by ASTM C 618 [6]. The GPOA cannot be classified as class N pozzolan because the contents of SiO2+Al2O3 + Fe2O3 were less than 70%. It should be noted that the loss on ignition (LOI) contents of GPOA and GRBA were rather high as 10.1% and 11.2%, respectively and 74.8% of GRBA was SiO2. Particle size distribution curves of OPC and the other materials are shown in Figure 4. The median particle size of OFA was 27.1 lm, which was larger than that of the OPC (14.7 lm). Before grinding, palm oil fuel ash and rice husk–bark ash had median particle sizes more than 100 lm. After grinding, the median particle sizes of GPOA and GRBA were reduced to 8.0 and 10.2 lm, respectively. The particle shapes of OPC and the replacement materials are presented in Figure 3. The particle shape of OFA was spherical and smooth surface indicating a rather complete burning. On the other hand, both of GPOA and GRBA had an angular and irregular particle shape, which were similar to that of Portland cement Type I.
Table 4: Chemical composition of cement and replacement materials

<table>
<thead>
<tr>
<th>Chemical Composition (%)</th>
<th>Cement (OPC)</th>
<th>Fly ash (OFA)</th>
<th>Ground palm oil fuel ash (GPOA)</th>
<th>Ground rice husk–bark ash (GRBA)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SiO₂</td>
<td>20.9</td>
<td>41.1</td>
<td>57.8</td>
<td>74.8</td>
</tr>
<tr>
<td>Al₂O₃</td>
<td>4.8</td>
<td>22.5</td>
<td>4.6</td>
<td>0.2</td>
</tr>
<tr>
<td>Fe₂O₃</td>
<td>3.4</td>
<td>11.6</td>
<td>3.3</td>
<td>0.8</td>
</tr>
<tr>
<td>CaO</td>
<td>65.4</td>
<td>15.3</td>
<td>6.6</td>
<td>5.9</td>
</tr>
<tr>
<td>MgO</td>
<td>1.2</td>
<td>2.8</td>
<td>4.2</td>
<td>0.6</td>
</tr>
<tr>
<td>Na₂O</td>
<td>0.2</td>
<td>1.7</td>
<td>0.5</td>
<td>0.2</td>
</tr>
<tr>
<td>K₂O</td>
<td>0.3</td>
<td>2.9</td>
<td>8.3</td>
<td>2</td>
</tr>
<tr>
<td>SO₃</td>
<td>2.7</td>
<td>1.5</td>
<td>0.3</td>
<td>0.5</td>
</tr>
<tr>
<td>LOI</td>
<td>0.9</td>
<td>0.2</td>
<td>10.1</td>
<td>11.2</td>
</tr>
<tr>
<td>SiO₂+Al₂O₃+Fe₂O₃</td>
<td>-</td>
<td>75.2</td>
<td>65.7</td>
<td>76.8</td>
</tr>
</tbody>
</table>

4.2. Water Requirement in Concrete Mixtures

The water-to-binder (W/B) ratios are shown in Table 4. The W/B ratio of OPC concrete was 0.71. The use of OFA could reduce the W/B ratio in the concrete mixture and the W/B ratio was much lower than OPC concrete as the increase in replacement ratios. This result was affected by the spherical particles of OFA. The W/B ratios of GPOA and GRBA concretes were higher than that OPC concrete and tended to increase with the higher replacement ratios. Because the particles of GPOA and GRBA were angular and irregular with some porous particles, they needed more water to lubricate for maintaining the same workability than OPC concrete. It was noted that the W/B ratios of GRBA concrete were larger than that of GPOA concrete.

![Figure 2. Particle size distribution of materials](image)

4.3. Compressive Strength

At the later age, their strengths slightly increased, therefore, 90-day compressive strengths of these concretes were 29.4, 23.7 and 22.3 MPa or 104%, 84% and 79% of OPC concrete, respectively. It was observed that the compressive strength of GPOA20 concrete at 90 days was slightly higher than that of OPC concrete,
although the W/B ratio of GPOA20 concrete was higher. This is due to the filler effects and the pozzolanic reaction of the high fineness of GPOA. Otherwise, the increase in replacement ratio of GPOA to 40% and 55% decreased the strength of concrete. However, the normalized compressive strength of all GPOA concretes increased with the ages. This suggests that the contribution of compressive strength was due to the pozzolanic reaction of GPOA with calcium hydroxide released from hydration of cement.

![Figure 3. Scanning electron microscopy (SEM) of materials: (a) Portland cement type I (OPC); (b) original fly ash (OFA); (c) ground palm oil fuel ash (GPOA); and (d) ground rice husk–bark ash (GRBA)](image)

From the results, it revealed that the incorporation of 20% of these pozzolans did not adversely affect the strength of concrete. An increase in the replacement ratios to 40% and 55% of binder, however, decreased the strength of concrete. For the same replacement ratio, the strengths of GPOA and GRBA concretes were slightly higher than those of OFA concretes, although their W/B ratios were higher than the OFA concretes. This suggested that the strengths of GPOA and GRBA concretes were affected by the finer particles of GPOA and GRBA as compared to OFA particles. Thus the faster pozzolanic reaction of GPOA and GRBA occurred and the filler effect made the concrete denser. In addition, the strengths of GBRA concretes were slightly higher than the strengths of GPOA concretes, although they needed more water in the concrete mixtures. This indicated that GRBA was more reactive than GPOA. This is due to the rice husk–bark ash contains a large amount of SiO2 (74.8%). It is known that the proper burnt and ground rice husk ash (has the content of SiO2 more than 80%) develops the compressive strength of concrete at the early age [9, 10]. Although, the burning temperature of GRBA was quite
high (800–900 °C) and some part of the silica might become crystalline, it had been found that rice husk ash with this high temperature burning could still be successfully used as a good supplementary cementitious material [10, 11].

Table 5: Compressive strength and permeability of concrete

<table>
<thead>
<tr>
<th>Mixed</th>
<th>W/B</th>
<th>Compressive strength (MPa)-Normalized</th>
<th>Permeability · 10⁻¹², k (m/s) – k/kcontrol</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>28 days</td>
<td>90 days</td>
</tr>
<tr>
<td></td>
<td></td>
<td>28 days</td>
<td>90 days</td>
</tr>
<tr>
<td>OPC</td>
<td>0.71</td>
<td>26.1 – 100</td>
<td>28.2 – 100</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2.89 – 1.00</td>
<td>2.05 – 1.00</td>
</tr>
<tr>
<td>OFA20</td>
<td>0.7</td>
<td>26.3 – 101</td>
<td>28.7 – 102</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2.22 – 0.77</td>
<td>0.60 – 0.29</td>
</tr>
<tr>
<td>OFA40</td>
<td>0.65</td>
<td>20.9 – 80</td>
<td>24.4 – 87</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4.67 – 1.62</td>
<td>2.01 – 0.98</td>
</tr>
<tr>
<td>GPOA20</td>
<td>0.73</td>
<td>23.9 – 92</td>
<td>29.4 – 104</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.59 – 0.20</td>
<td>0.25 – 0.12</td>
</tr>
<tr>
<td>GPOA40</td>
<td>0.74</td>
<td>20.7 – 79</td>
<td>23.7 – 84</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.41 – 0.14</td>
<td>0.26 – 0.13</td>
</tr>
<tr>
<td>GPOA55</td>
<td>0.75</td>
<td>18.1 – 69</td>
<td>22.3 – 79</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3.30 – 1.14</td>
<td>2.38 – 1.16</td>
</tr>
<tr>
<td>GRBA20</td>
<td>0.71</td>
<td>27.5 – 105</td>
<td>29.3 – 104</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.90 – 0.31</td>
<td>0.42 – 0.21</td>
</tr>
<tr>
<td>GRBA40</td>
<td>0.76</td>
<td>22.7 – 87</td>
<td>25.6 – 91</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.74 – 0.60</td>
<td>1.33 – 0.65</td>
</tr>
<tr>
<td>GRBA55</td>
<td>0.8</td>
<td>20.0 – 77</td>
<td>24.1 – 85</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5.48 – 1.90</td>
<td>4.02 – 1.96</td>
</tr>
</tbody>
</table>

Table 6: Mix proportions of concrete

<table>
<thead>
<tr>
<th>Mixtures</th>
<th>Cement</th>
<th>OFA (kg)</th>
<th>GPOA (kg)</th>
<th>GRBA (kg)</th>
<th>Fine aggregate (kg)</th>
<th>Coarse aggregate (kg)</th>
<th>Water (kg)</th>
<th>Slump (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>OPC</td>
<td>300</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>915</td>
<td>1080</td>
<td>213</td>
<td>75</td>
</tr>
<tr>
<td>OFA20</td>
<td>240</td>
<td>60</td>
<td>-</td>
<td>-</td>
<td>904</td>
<td>1068</td>
<td>215</td>
<td>65</td>
</tr>
<tr>
<td>OFA40</td>
<td>180</td>
<td>120</td>
<td>-</td>
<td>-</td>
<td>893</td>
<td>1057</td>
<td>195</td>
<td>60</td>
</tr>
<tr>
<td>GPOA20</td>
<td>240</td>
<td>-</td>
<td>60</td>
<td>-</td>
<td>907</td>
<td>1072</td>
<td>220</td>
<td>65</td>
</tr>
<tr>
<td>GPOA40</td>
<td>180</td>
<td>-</td>
<td>120</td>
<td>-</td>
<td>900</td>
<td>1064</td>
<td>222</td>
<td>70</td>
</tr>
<tr>
<td>GPOA55</td>
<td>135</td>
<td>-</td>
<td>155</td>
<td>-</td>
<td>894</td>
<td>1059</td>
<td>225</td>
<td>90</td>
</tr>
<tr>
<td>GRBA20</td>
<td>240</td>
<td>-</td>
<td>-</td>
<td>60</td>
<td>903</td>
<td>1068</td>
<td>214</td>
<td>80</td>
</tr>
<tr>
<td>GRBA40</td>
<td>180</td>
<td>-</td>
<td>-</td>
<td>120</td>
<td>891</td>
<td>1055</td>
<td>229</td>
<td>60</td>
</tr>
<tr>
<td>GRBA55</td>
<td>135</td>
<td>-</td>
<td>-</td>
<td>155</td>
<td>883</td>
<td>1046</td>
<td>240</td>
<td>70</td>
</tr>
</tbody>
</table>

4.4. Water Permeability of Concretes

The water permeability of concrete and the ratio of permeability are given in Table 6. The ratio of permeability is defined as the permeability of concrete containing pozzolanic materials divided by the permeability of OPC concrete at the same age of testing. At 90 days, the permeability of GPOA concretes decreased as compared to that of 28-day. It is interesting to note that the result showed that GPOA20 and GPOA40 concretes gave the lower permeability than the OPC concrete, even though the W/B ratios of the two concretes were higher than the OPC concrete. The permeability of GPOA55 rapidly increased and was higher than the OPC concrete. This may result from the low cement content and the high W/B ratio of GPOA55 concrete [12].

Figures 4 and 5 show the relationship between the permeability of all concrete and the cement replacement levels at 28 and 90 days, respectively. At 28 days, most of concretes had lower permeability than that of OPC concrete, except OFA40 and
GRBA55 concretes and the lowest permeability was observed in GPOA40 concrete. Moreover, the permeabilities of GPOA concretes tended to decrease when the cement replacement ratio increased up to 40%. On the contrary, the permeabilities of OFA and GRBA concretes increased as the cement replacement was more than 20%. At 90 days, the permeabilities of all concretes reduced and were lower than the values at 28 days. The ratios of permeability also had the similar result, except GRBA55 concrete. This suggested that the development of permeation

Figure 4. Relationship between the permeability of concretes and the cement replacement ratios at 28 days

Figure 5. Relationship between the permeability of concretes and the cement replacement ratios at 90 days

Figure 6. Relationship between water permeability and compressive strength of concretes at 28 days
4.5. Durability Performance

The long-term durability of a structure is highly affected by the permeability of concrete and, therefore, the water permeability can be used as an indicator of the durability of OPS concrete. The water permeability obtained from this investigation is illustrated in Figure 8.

From the figure, it is observed that the OPS concrete becomes less permeable with time and also it is greatly affected by the curing regime. At the age of 28 days, when cured under CC condition, the water permeability was about 9 times lower compared to that cured under CL condition. This shows when sufficient amount of water is present, the hydration of cement can continue. Consequently, the total porosity of the concrete is reduced as the probability of pores being either blocked or narrowed down by continued formation of hydration products are increased.

4.6. Relationship Between Compressive Strength And Water Permeability of Concrete

The relationships between the permeability and the compressive strength of concretes at 28 and 90 days are presented in Figures. 6 and 7, respectively. The
permeability of concrete tended to decrease with the increasing in the compressive strength. The figures are divided into four regions in Figures. 6 and 7. Region I represents concretes which have both compressive strength and permeability higher than OPC concrete. Region II indicates concretes which have lower compressive strength but higher permeability as compared to OPC concrete. Region III, which are lower in both of compressive strength and permeability than OPC concrete. Region IV are the preferable concretes, which are more lower permeability and also have higher compressive strength than OPC concrete.

5. CONCLUSIONS
It can be said the following conclusions are understood:
The permeability of GPOA and GRBA concretes depends on the cement replacement ratios, and age of concretes. In general, the permeability of concrete reduces with the increasing in the compressive strength and age of concrete. The optimum cement replacement by GPOA, GRBA and OFA in this experiment is 20%. The higher replacement than this ratio results in the reduction of compressive strength and tends to give higher permeability of concrete. Although GPOA and GRBA increased the amount of water in concrete mixture, the compressive strength of concretes containing 20% of these materials as cement replacement were higher than OPC concrete. With 40% of cement replacement, the compressive strength of GPOA and GRBA concretes were more than 84% of OPC concrete at 90 days. Both GPOA and GRBA are suitable as pozzolanic materials in concrete. This shows a good promise to utilize these waste materials GRBA produces higher compressive strength than GPOA at all cement replacement levels, although lower of permeability of concrete was obtained with GPOA.

REFERENCES
7. Khatri, R.P. and V. Sirivivatnanon, Methods for the Determination of Water
Permeability of Concrete. ACI Materials Journal, 1997. 94(3).
HIGH PERFORMANCE CONCRETE FOR REPAIR OF SEWER NETWORK

T. Parhizkar, A.M. Raiss ghasemi, A.R. Pourkhorshidi
Building and Housing Research Center

ABSTRACT

High performance concrete often consists of ternary mixes of silica fume, fly ash or other pozzolans, and Portland cement. However, the specification of the proportions of these components remains uncertain [1-2]. High performance concrete have found their place in construction and compete with other materials for certain features like high strength, long term durability in severe conditions. In fact, high performance concrete has been designed in order to improve durability and provides easy handling, placement and compaction [3-4]. In this research, a high performance concrete with unique specifications has been studied. One of the most important properties of this type of concrete is its low permeability against different aggressive agents, and reduction of cement content in comparison with other high performance concretes.

Keywords: high performance concrete, fly ash, silica fume, permeability

1. INTRODUCTION

Fly ash and silica fume is a by-product. When used in concrete manufacturing, it is truly a "green" building material because it replaces a portion of the Portland cement, and the resulting emissions associated with its production. Fly ash and silica fume have the consistency of fine powder. Due to the shape, size and chemical composition, it imparts a number of benefits to concrete such as reduced water demand, improved durability and increased strength [1-3]. Use of fly ash and silica fume leads to improved workability- hence ease of handling, placing (pumping), and compacting concrete, reduction in bleeding and improvement in cohesiveness of concrete- hence smooth form finished concrete without honey-combing or segregation, reduction in heat of hydration- hence no threat of thermal cracks which makes it ideal for foundations and large sections. It also comes in handy in designing higher grades of concrete, improved resistance to Chloride attack. Chloride attacks lead to corrosion of reinforcing steel and subsequent distress of concrete [5-6]. Considering the optimized specifications of high performance concrete, it is suitable for repair of concrete, in for example, sewer network.

Design of high performance concrete, involves different factors. The most important factors included water-cement ratio, increase in the amount of cement, using mineral and chemical additives and grading curve of particle size for improvement of mixture density [7-9].
2. EXPERIMENTAL PROGRAM

2.1. Materials

Aggregate: Crushed gravel and sand were used as coarse and fine aggregates, respectively. The properties of aggregates are shown in Table 1. Curves of grading are given in Figures 1 and 2.

<table>
<thead>
<tr>
<th></th>
<th>Specific Gravity</th>
<th>Absorption (%)</th>
<th>Fineness Modulus</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand</td>
<td>2.53</td>
<td>2.6</td>
<td>2.7</td>
</tr>
<tr>
<td>Gravel</td>
<td>2.56</td>
<td>1.46</td>
<td>6.5</td>
</tr>
</tbody>
</table>

![Figure 1. Fine-aggregate grading curve for SN and FSN mixtures](image)

![Figure 2. Coarse-aggregate grading curve for SN and FSN mixtures](image)

Cement: ASTM type II Portland cement was used in this investigation. The composition of cement is shown in Table 2.

Fly ash: Fly ash was prepared in Germany. The composition of Fly ash is shown in Table 2.

Silica fume: A locally produced silica fume (in accordance with ASTM C1240) was used. The composition of Silica fume is shown in Table 2.
Grading of binding material is determined by Particle Size Analyzer method and the relative distribution curve is given in Figure 3.

Superplasticizer-The superplasticizer was conventional carbocilate-based. Chemical properties of the superplasticizer are given in Table 3.

<table>
<thead>
<tr>
<th>Table 2: Chemical compositions of cement, fly ash and silica fume</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type II</td>
</tr>
<tr>
<td>--------</td>
</tr>
<tr>
<td>SiO₂</td>
</tr>
<tr>
<td>Al₂O₃</td>
</tr>
<tr>
<td>Fe₂O₃</td>
</tr>
<tr>
<td>MgO</td>
</tr>
<tr>
<td>CaO</td>
</tr>
<tr>
<td>SO₃</td>
</tr>
<tr>
<td>Na₂O+0.658 K₂O</td>
</tr>
<tr>
<td>C₃S</td>
</tr>
<tr>
<td>C₂S</td>
</tr>
<tr>
<td>C₃A</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Table 3: Chemical and physical properties of superplasticizer</th>
</tr>
</thead>
<tbody>
<tr>
<td>Apparent condition</td>
</tr>
<tr>
<td>PH</td>
</tr>
<tr>
<td>Density</td>
</tr>
<tr>
<td>Quantity of chlorine</td>
</tr>
<tr>
<td>Quantity of alkaline</td>
</tr>
</tbody>
</table>

2.2. Grading of Binding Material

As about 80% of cement particles have a diameter in the range of 10 to 100 micrometer, 50% of the particles of Fly ash have the particle size between 5 to 20 micrometer and 60% of silica fume has a size of less than 1 micrometer. This is with regard to the chemical properties of these materials while considering their mechanical and physical properties, as well as their required durability and the particle size distribution of cement. Fly ash and silica fume and also 70% cement, 20% fly ash and 10% silica fume with the combination curve is given in Figures 3 and 4.
2.3. Concrete Mixtures
Mixture proportions of concrete are summarized in Table 4. Water-cementitious materials ratios(W/Cm) were 0.45. After mixing the concrete, casting performed in the moulds was based on En 12390-2:2000. After one day, under standard conditions, specimens were removed from the moulds and transferred to the standard curing condition until the time of testing.

Table 4: Mixture proportions

<table>
<thead>
<tr>
<th>Mixture</th>
<th>Water (kg/m³)</th>
<th>Cement (kg/m³)</th>
<th>Fly ash (kg/m³)</th>
<th>Silica fume (kg/m³)</th>
<th>Fine (0-0.25mm) (kg/m³)</th>
<th>Coarse (0.25-16mm) (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SN</td>
<td>170</td>
<td>372</td>
<td>-</td>
<td>30</td>
<td>56</td>
<td>1833</td>
</tr>
<tr>
<td>FSN</td>
<td>124</td>
<td>220</td>
<td>63</td>
<td>30</td>
<td>60</td>
<td>1955</td>
</tr>
</tbody>
</table>

2.4. Tests on Fresh Concrete and Results
- Density
Density of fresh concrete was obtained based upon En 12350-6:1999. Results are shown in Table 5.

- Workability
The slump of fresh concrete, was determined based on EN 12350-2:1999 method. Results are shown in Table 5.

- Fluidity in Flow table test
The fluidity of fresh concrete was measured based on EN 12350-5:1997 method. Results are shown in Table 5.

- Air Content
The test performed for measuring air content in concrete was based on per EN 12350-5:1997 method. Results are shown in Table 5.
Table 5: Results of fresh concrete test

<table>
<thead>
<tr>
<th>Mixture</th>
<th>slump (cm)</th>
<th>Flow consistency (cm)</th>
<th>Density (kg/m³)</th>
<th>Air content (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SN</td>
<td>4</td>
<td>37.5</td>
<td>2391</td>
<td>3.8</td>
</tr>
<tr>
<td>FSN</td>
<td>-</td>
<td>-</td>
<td>2466</td>
<td>2.0</td>
</tr>
</tbody>
</table>

2.5. Tests on Hardened Concrete and Results

Tests for determination of mechanical properties and durability of concrete, performed on hardened concrete include: Determination of compressive strength, rapid chloride ion permeability, depth of water penetration under pressure, length change of concrete prisms in sulphate solution. To perform the aforesaid test, samples of cubic, cylindrical and prism shapes were prepared upon EN 12390-1:2000. Dimensions and shapes of moulds for the purpose of each test is given in Table 6.

Table 6. Dimensions and shapes of moulds to be used for hardened concrete tests

<table>
<thead>
<tr>
<th>Type of test</th>
<th>Mould shape</th>
<th>Mould dimensions (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive strength</td>
<td>Cube</td>
<td>100</td>
</tr>
<tr>
<td>Depth of water penetration</td>
<td>Cube</td>
<td>150</td>
</tr>
<tr>
<td>Rapid chloride permeability test</td>
<td>Cylindrical</td>
<td>100 x200</td>
</tr>
<tr>
<td>Length change in sulfate solution</td>
<td>Prism</td>
<td>285x75x75</td>
</tr>
</tbody>
</table>

3. RESULTS

3.1. Compressive strength

Samples (three samples per age) were tested in the age of 7, 28 and 90 days. Compressive strength of samples was measured based on DIN EN 12390-3:2000 and the results are given in Figure 5.

![Figure 5. Results of compressive strength test](image-url)
3.2. Rapid Chloride Permeability
This test is performed on ASTM C 1202-97 method in the age of 28 and 90 days. Test results are given in Figure 6.

![Figure 6. Results of rapid chloride permeability test](image)

3.3. Depth of Water Penetration Test
This test was performed based on EN 12390-9:2000 method at the age of 28 and 90 days and the results are given in Figure 7.

![Figure 7. Results of depth of water penetration test](image)

3.5. Length Change of Concrete Prism in Sulphate Solution
To study performance of concrete mixtures in sulphate, the 5% sodium sulphate solution was used. For this purpose, concrete prisms measuring 75x75x285 mm after 28 days of standard curing were placed in sulphate solution. For adjustment of pH and keeping this parameter constant during the process of test, diluted sulphuric acid was used. Meantime and at the early ages, sulphate solution was changed regularly. Elongation and weight loss of samples at certain intervals and up to 6 months was determined. Results are given in Figure 8.
4. CONCLUSIONS
According to the experimental work:
1. The experimental observation showed the effectiveness of Fly Ash on the development of concrete compressive strength and reduction of the chloride permeability.
2. At the early age, the Fly Ash has fewer effects on the properties of concrete, but after 28 days, it could result in a better performance in contrast to the concrete without Fly Ash.

REFERENCES
چکیده
پژوهشگران مختلف مطالعه‌ای برای دستیابی به منحنی دانه بندی با کمترین تخلخل و در نتیجه کاهش حجم خمیر ایران آموز کرده‌اند. برای مقایسه دانه بندی‌های مخالط در ایران، در این مقاله، دانشگاه علوم و صنعت ایران، دانشجوی کارشناسی ارشد دانشگاه علوم و صنعت ایران، برای اجای اصلاح در روش‌های مخالطه‌ریزی و شکل‌دادن دانه منحنی، به تحقیقات تایید شده در این زمینه اشاره کرده و بررسی دقیقی از چوک حرارتی این دانه منحنی مورد بررسی قرار گرفته است. در این مطالعه، این امکان ایجاد دانه است که فقط از فیلر اصلاح مجازات انجام می‌شود و نیازی به استفاده از VMA نیست. در نهایت، نتایج آزمایش با دانه منحنی در این آزمایشات نشان داده شده است که کاهش حجم خمیر و تاخیر در سمت بازه، باعث کاهش در سمت بازه و افزایش در سمت دیگر می‌شود. بنابراین، دانه منحنی دانه بندی، باید به دانه غیر منحنی دانه بندی توزیع شود.

کلیدواژه‌ها: دانه منحنی، دانه بندی، فیلر، حرارتی، حجم خمیر ایران.
سنگدانه‌ها از ریز پترندی‌های مناسب استفاده نمودر در این صورت احتمال ایجاد توش در نسبت سنتی‌ها کاهش پیدا می‌کند و بنی می‌تواند به راحتی تحت اثر وزن خاری جاری شده و از این جهت قابلیت‌های عبور کند.

تأثیرات دانی بنی روی خواص بنی تازه ناشی از تراکم سنگدانه‌ها، که این تراکم بستگی به محتوی دانه بنی سنگدانه‌ها دارد، چه تراکم سنگدانه‌ها بخشی باشد و تخلخل ما بین آن‌ها کمتر شود و در نتیجه خمیر سیمان بیشتر برای پوشاندن دور سنگدانه‌ها و اسیده تر غلیظ‌تر آن‌ها روی یکدیگر در دسترس خواهد بود. 

[1] آن‌ها توجه به این مطلب رابطه ای برای به دست آوردن درصد عبوری از هر اک و در نهایت بدست اوردن محتوی دانه‌های مناسب آن را دادند که به صورت زیر می‌باشد:

\[ P(D) = (D/D_{max})^q \]

که در آن:

\[ P(D) = \text{درصد عبوری از اک با اندازه جسمی} \]

\[ D = \text{دماکتر درصد سنگدانه} \]

\[ D_{max} = \text{حداکثر درصد سنگدانه} \]

\[ q = \text{نقطه بین‌بندی پیش‌بینی اپارت‌ها} \]

\[ 0 < q < 1 \]

عبارت \( D_{max} \) پارامتر توأم در معادله (1) و (2) می‌باشد که این پارامتر نقش بسیار مهمی در تعیین شکل محتوی دانه بنی سنگدانه‌ها دارد که کاهش آن محتوی دانه بنی سنگدانه‌ها را زیر دانه تا یک افزایش آن محتوی داخل دانه بنی درشت دانه تا شود. 

[1] یافته‌ای که اگر \( q = 0.37 \) باشد تراکم بهینه سنگدانه‌ها به دست می‌آید، بنی به دست می‌آید Fuller به دست می‌آید Andreasen & Anderson.

[2] اصلاح شده را به صورت زیر ارائه دادند:

\[ P(D) = (D_{min} - D) / (D_{max} - D_{min}) = P(D) = (D_{min} - D) / (D_{max} - D_{min}) \]

که در آن:

\[ D_{min} = \text{حداقل اندازه سنگدانه} \]

\[ D_{max} = \text{حداقل اندازه سنگدانه} \]

\[ D_{min} = D_{max} \]

\[ P(D) = \text{درصد عبوری از اک با اندازه جسمی} \]

\[ D_{max} = \text{حداکثر درصد سنگدانه} \]

\[ D_{min} = D_{max} \]

\[ q = \text{نقطه بین‌بندی پیش‌بینی اپارت‌ها} \]

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پیشینه چندمورد قبلبرای تهیه مخلوط بتن خوذي را ارائه داده است. در نهایت برای هر یک از میزانه دانه بندی‌های مذکور، جهت تهیه مخلوط خود تراکم یک بزرگ حجم خمیر سیمان تعیین شده است و مقدار بهینه حجم خمیر سیمان، در این باره را که منجر به بهترین خواص بتن تازه مخلوط، در مقایسه با سایر مقادیر این بزرگ شده است، نیز برای هر میزانه دانه بندی مشخص شده است.

درصد عبوری میزانه دانه بندی‌های سیستم‌ها روش ملی طرح مخلوط پن بتن ایران که در شکل 1 نشان داده شده است، به صورت درصد عبوری حجم‌می‌باشد. زمانی که جرم حجمی سیستانه‌ها یکسان در نظر گرفته شود درصد عبوری حجمی با ویژه تفاوت ندارد. که این موضوع با مقایسه جدول 1-ب با شکل 1 مشخص می‌گردد.اما در این تحقیق به دلیل استفاده از فیلتر، میزانی در منحنی دانه بندی سیستانه‌های سرد استفاده، و متقاوت بودن جرم حجمی فیلتر با سیستانه‌ها، درصد عبوری حجم‌های منحنی بندی افراد طرح عبوری وزنی دانسته‌بای یک بررسی این مطلب این‌جا با توجه به منحنی دانه بندی مورد استفاده درصد حجمی عبوری در یک از اندازه سیستانه‌ها و فیلتر.

شکل 1- منحنی دانه بندی های روشن ملی طرح مخلوط ایران با حداکثر اندازه 9/5 و 19/9 میلیمتر

(الف) منحنی دانه بندی سیستانه‌ها با حداکثر اندازه 9/5 میلیمتر

(ب) منحنی دانه بندی سیستانه‌ها با حداکثر اندازه 19/9 میلیمتر
مشخص شده و به صورت تطبیقی گردیده و در جدول ۱-الف نشان داده شده است. با توجه به جدول ۱، بیشترین اختلاف درصد وزنی عبوری با درصد حجمی عبوری منتجی به دلیل هماهنگی که اندازه‌گیری می‌رتفع مربوط به منتجی دانه بانده ۳۹.۷% بوده است و مقادیر اختلاف برابر با ۴ درصد بوده است.

جدول ۱: الف- درصد وزنی عبوری منتجی دانه بانده روش مایل طرح مخلوط بین ایران با پوست سکن

جدول ۱: ب- درصد وزنی عبوری منتجی دانه بانده روش مایل طرح مخلوط بین ایران بدون پوست سکن

فیلر
پودر سگ همچنین وزن مخصوص ۲۶۴۰ به عنوان فیلر استفاده شده است.

فوق روان کندنه
فوق روان کندنه مور مختص بر پایه پلی کربوکسیلاتی و تولید کارخانجات داخل می‌باشد.

۲-۲ نسبتهای مخلوط
در این تحقیق با منتجی‌های B، C طبق شکل ۱ با حداکثر اندازه ۹.۵ که بزرگ‌تر از روش مایل طرح مخلوط بین ایران می‌باشد، مخلوط‌های منتجی بین خود تراکم ساخته شده است. تحقیقات آزمایشگاهی مقدامی نشان داد که با منتجی‌هایی که ساخت مخلوط‌های تراکم وجود دارد. در تمامی مخلوط‌های ساخته شده نسبت اپسیمی برای برابری با ۲۵۰ تای نگهدارشده و مقدار سیمان برای هر گروه منتجی دانه بانده تغییر نداده است. همچنین لازم به ذکر است که در تمامی مخلوط‌های مذکور به منظور تهیه سکن‌های ریز ریز از ۱۰۱۵ میلی‌متری بینی عبوری از
الک ۰.۱۵ از پودر سنگ عبور داده شده از الک ۰.۱۵ استفاده شده است.

نتیجه‌ی مخلوط‌های ساخته شده با منحنی دانه بندی‌های B, C به حداقل اندام‌های سنگ‌های ۱۹.۵ میلی‌متر در جدول ۲ اشاره داده شده است. این تحقیق معرفی ۰.۵۰۱ به مفهوم منحنی دانه بندی سنگ‌های سیا، حداقل اندام سنگ‌های ۹.۵ میلی‌متر و خط ثبیت و عدد شماره مخلوط ساخته شده می‌باشد. برای سایر معرفی مخلوط‌ها نیز به همین صورت می‌باشد.

جدول ۲: نسبت‌های مخلوط‌های ساخته شده با منحنی‌های B, C به حداقل اندام‌های ۹.۵ و ۱۹ میلی‌متر

<table>
<thead>
<tr>
<th>حجم خمیر (سیمان، آب) (m3/1m3)</th>
<th>فوق روغن کنده (lit/m3)</th>
<th>آب (kg/m3)</th>
<th>سیانه (kg/m3)</th>
<th>پودر سنگ (kg/m3)</th>
<th>منحنی دانه بندی</th>
<th>معرف مخلوط</th>
</tr>
</thead>
<tbody>
<tr>
<td>.۰۲۸</td>
<td>١.٧٥</td>
<td>١۸٠</td>
<td>١٥٧.٥</td>
<td>٣٣٣.٢٨</td>
<td>٣٥٠</td>
<td>٩.٥ C</td>
</tr>
<tr>
<td>.۰۳٧</td>
<td>١.۵</td>
<td>٢٠٠</td>
<td>١٤٨.٦٢</td>
<td>٣١١.٧٣</td>
<td>٢٠٠</td>
<td>٣.٨٥ C</td>
</tr>
<tr>
<td>.۰۲٥</td>
<td>١</td>
<td>٢٠٢.٥</td>
<td>١٣٧.٣٠</td>
<td>٢٩٨.٥٧</td>
<td>٢٠٠</td>
<td>٣.٩٥ C</td>
</tr>
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<td>٠.۵</td>
<td>٢٢٥</td>
<td>١٨٩.٨٠</td>
<td>١٨٧.٣٣</td>
<td>٤٠٠</td>
<td>٩.۵٥ B</td>
</tr>
<tr>
<td>.٠٩٨</td>
<td>١.٦٥</td>
<td>١٨٠</td>
<td>١٥٢.٧٨</td>
<td>٢٣٣.٣٥</td>
<td>٤٠٠</td>
<td>٩.۵٥ B</td>
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<td>٠.٧٥</td>
<td>٢٠٢.٥</td>
<td>١٤٨.٦٢</td>
<td>٣١١.٧٣</td>
<td>٢٠٠</td>
<td>٣.٨٥ B</td>
</tr>
<tr>
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<td>٠.۵</td>
<td>٢٢٥</td>
<td>١٨٩.٨٠</td>
<td>١٨٧.٣٣</td>
<td>٤٠٠</td>
<td>٩.۵٥ B</td>
</tr>
<tr>
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<td>٠.۵</td>
<td>٢٢٥</td>
<td>١٨٩.٨٠</td>
<td>١٨٧.٣٣</td>
<td>٤٠٠</td>
<td>٩.۵٥ B</td>
</tr>
<tr>
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<td>٠.۵</td>
<td>٢٠٢.٥</td>
<td>١٤٨.٦٢</td>
<td>٣١١.٧٣</td>
<td>٢٠٠</td>
<td>٣.٨٥ B</td>
</tr>
<tr>
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<td>٠.۵</td>
<td>٢٢٥</td>
<td>١٨٩.٨٠</td>
<td>١٨٧.٣٣</td>
<td>٤٠٠</td>
<td>٩.۵٥ B</td>
</tr>
<tr>
<td>.٠٣٢</td>
<td>٠.۵</td>
<td>٢٠٨.٥</td>
<td>١٤٨.٦٢</td>
<td>٣١١.٧٣</td>
<td>٢٠٠</td>
<td>٣.٨٥ B</td>
</tr>
</tbody>
</table>

3- شرح آزمایش‌ها

آزمایش گریبان اسلامی، قیف V، حلقه چاهه، آزمایش مشاهده PCI [۵] و VSI مطابق با دستور العمل شده است.
نتایج آزمایش‌ها و تفسیر

نتایج آزمایش‌های انجام شده روی مخلوطها در جدول 3 نشان داده شده است.

جدول 3: نتایج آزمایش‌های انجام شده روی مخلوط‌ها

<table>
<thead>
<tr>
<th>جرم (cm)</th>
<th>حجم خمیر سیمان (m^3/1m^3)</th>
<th>روش</th>
<th>دانه‌بندی</th>
<th>Dmin</th>
<th>Dmax</th>
</tr>
</thead>
<tbody>
<tr>
<td>64</td>
<td>0.30</td>
<td></td>
<td>184</td>
<td>0.35</td>
<td>0.35</td>
</tr>
<tr>
<td>68</td>
<td>0.30</td>
<td></td>
<td>184</td>
<td>0.35</td>
<td>0.35</td>
</tr>
<tr>
<td>67</td>
<td>0.30</td>
<td></td>
<td>184</td>
<td>0.35</td>
<td>0.35</td>
</tr>
</tbody>
</table>

در این تحقیق زمانی که (VSI)، بین صفر تا یک به‌دست‌آمد مخلوط به عنوان مخلوط بین‌خودن‌تر اول گرفته شده است.

*VSI = Visual Stability Index

منحنی دانه‌بندی های C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B، C، B，*
‫–––––––––––––––––––––––––––––– ﺑﺮرﺳﻲ اﺛﺮ ﻣﻨﺤﻨﻲ داﻧﻪ ﺑﻨﺪيﻫﺎي روش ﻣﻠﻲ ﻃﺮح ‪109 / ....‬‬

‫ﻣﻄﻠﻮﺑﺘﺮي ﻧﺴﺒﺖ ﺑﻪ ﺳﺎﻳﺮ ﻣﺨﻠﻮﻃﻬﺎي ﺳﺎﺧﺘﻪ ﺷﺪه ﺑﺎ ﻣﻘﺎدﻳﺮ ﺑﺎزه از ﺧﻮد ﻧﺸﺎن داد‪.‬ﺑﺎزهﻫﺎي ﻣﺬﻛﻮر ﺑﻪ ﻫﻤـﺮاه ﻣﻘـﺎدﻳﺮ‬
‫ﺑﻬﻴﻨﻪ ﺑﺮاي ﻣﻨﺤﻨﻲ داﻧﻪ ﺑﻨﺪيﻫﺎي ﻣﻮرد ﺑﺮرﺳﻲ در ﺟﺪول ‪ 4‬اراﺋﻪ داده ﺷﺪه اﺳﺖ‪.‬‬
‫ﺟﺪول ‪ :4‬ﺑﺎزه ﺣﺠﻢ ﺧﻤﻴﺮﺳﻴﻤﺎن ﻻزم ﺑﺮاي ﺗﻬﻴﻪ ﻣﺨﻠﻮط ﺧﻮد ﺗﺮاﻛﻢ ﺑﺮاي ﻣﻨﺤﻨﻲ داﻧﻪﺑﻨﺪيﻫﺎي‬
‫روش ﻣﻠﻲ ﻃﺮح ﻣﺨﻠﻮط ﺑﺘﻦ اﻳﺮان‬
‫ﻣﻨﺤﻨﻲ داﻧﻪ ﺑﻨﺪي‬
‫‪9,5C‬‬
‫‪9,5B‬‬
‫‪19C‬‬
‫‪19B‬‬

‫ﺑﺎزه ﺣﺠﻢ ﺧﻤﻴﺮ ﺳﻴﻤﺎن‬

‫ﺣﺠﻢ ﺧﻤﻴﺮ ﺳﻴﻤﺎن ﺑﻬﻴﻨﻪ‬

‫)‪(m3/1m3‬‬

‫)‪(m3/1m3‬‬

‫‪0,28-0,383‬‬
‫‪0,307-0,39‬‬
‫‪0,23-0,34‬‬
‫‪0,3-0,37‬‬

‫‪0307‬‬
‫‪0,345‬‬
‫‪0,268‬‬
‫‪0,33‬‬

‫ﺑﺎ ﻣﻘﺎﻳﺴﻪ ﻧﺘﺎﻳﺞ آزﻣﺎﻳﺸﻬﺎي اﻧﺠﺎم ﺷﺪه روي ﻣﺨﻠﻮﻃﻬﺎي ﺳﺎﺧﺘﻪ ﺷﺪه ﺑﺎ دو ﻣﻨﺤﻨﻲ داﻧﻪ ﺑﻨﺪي ‪ C,B‬ﺑﺎ ﺣﺪاﻛﺜﺮ اﻧـﺪازه‬
‫‪ 19‬ﻣﻴﻠﻴﻤﺘﺮ ﻣﺸﺨﺺ ﮔﺮدﻳﺪ ﻛﻪ ﺣﺠﻢ ﺧﻤﻴﺮ ﺳﻴﻤﺎن ﺑﻬﻴﻨﻪ ﻻزم ﺑﺮاي ﺗﻬﻴﻪ ﻣﺨﻠﻮط ﺧﻮد ﺗﺮاﻛﻢ ﺑﺎ ﻣﻨﺤﻨﻲ ‪C‬ﺑﺎ ﺣﺪاﻛﺜﺮ‬
‫اﻧﺪازه ‪ 19‬ﻣﻴﻠﻴﻤﺘﺮ ﻛﻤﺘﺮ از ﺣﺠﻢ ﺧﻤﻴﺮ ﺳﻴﻤﺎن ﺑﻬﻴﻨﻪ ﻻزم ﺑﺮاي ﺗﻬﻴﻪ ﻣﺨﻠﻮط ﺧﻮد ﺗـﺮاﻛﻢ ﺑـﺎ ﻣﻨﺤﻨـﻲ ‪ B‬ﺑـﺎ ﺣـﺪاﻛﺜﺮ‬
‫اﻧﺪازه ‪ 19‬ﻣﻴﻠﻴﻤﺘﺮ ﺑﻮده اﺳﺖ‪.‬ﺿﻤﻨﺎ ﺧﺼﻮﺻﻴﺎت ﺑﺘﻦ ﺗﺎزه ﺳﺎﺧﺘﻪ ﺷﺪه ﺑﺎ ﻣﻨﺤﻨﻲ ‪ C19‬ﻣﻄﻠﻮﺑﺘﺮ از ﻣﺨﻠﻮط ﺳﺎﺧﺘﻪ ﺷـﺪه‬
‫ﺑﺎ ﻣﻨﺤﻨﻲ ‪ B19‬ﺑﻮده اﺳﺖ‪.‬در ﻧﺘﻴﺠﻪ ﻛﻠﻴﻪ ﻣﻨﺤﻨـﻲ داﻧـﻪ ﺑﻨـﺪيﻫـﺎي ﺣﺎﺻـﻞ از ﻣـﺪل ‪Andreasen&Anderson‬‬
‫]‪[1‬ﻣﺎﺑﻴﻦ ﻣﻨﺤﻨﻲ ‪ B,C‬ﻣﻨﺎﺳﺐ ﺑﺮاي ﺳﺎﺧﺖ ﻣﺨﻠﻮط ﺧﻮدﺗﺮاﻛﻢ ﻣﻲﺑﺎﺷﻨﺪ‪.‬در ﺣﻘﻴﻘﺖ ﻣﻨﺨﻨﻲ داﻧﻪ ﺑﻨﺪيﻫـﺎي ﻣﺘﻨـﺎﻇﺮ‬
‫ﺑﺎ ﻣﻘﺎدﻳﺮ ‪ q‬ﺑﻴﻦ ‪ 0,1‬ﺗﺎ ‪ 0,35‬ﻣﻨﺎﺳﺐ ﺑﺮاي ﺗﻬﻴﻪ ﻣﺨﻠﻮط ﺧﻮد ﺗـﺮاﻛﻢ ﻣـﻲﺑﺎﺷـﺪ‪ .‬در ﺻـﻮرﺗﻴﻜﻪ ‪Redix&Browser‬‬
‫]‪[3‬ﻣﻘﺪار‪ q=0.25‬ﻣﻨﺎﺳﺐ ﺑﺮاي ﺳﺎﺧﺖ ﻣﺨﻠﻮط ﺧﻮد ﺗﺮاﻛﻢ داﻧﺴﺘﻪ اﻧﺪ‪.‬از ﻃﺮف دﻳﮕـﺮ ﻫﺮﭼـﻪ ﻣﻨﺤﻨـﻲ داﻧـﻪ ﺑﻨـﺪي‬
‫ﺣﺎﺻﻞ از ﻣﺪل ‪[1] Andreasen&Anderson‬ﺑﻪ ﺳﻤﺖ ‪ C‬ﻣﺘﻤﺎﻳﻞ ﺑﺎﺷﺪ ﻳﻌﻨﻲ ‪ q‬ﺑﻪ ﺳﻤﺖ ‪ 0,1‬ﻣﻴﻞ ﻛﻨـﺪ ﻣﺨﻠـﻮط‬
‫ﺳﺎﺧﺘﻪ ﺷﺪه ﺧﺼﻮﺻﻴﺎت ﺑﺘﻦ ﺗﺎزه ﻣﻄﻠﻮﺑﺘﺮي ﺧﻮاﻫﺪ داﺷﺖ‪.‬‬
‫‪[5] Larrard‬ﭘﺎراﻣﺘﺮﻫﺎﻳﻲ ﺑﺮاي ﺑﺮرﺳﻲ ﻣﻨﺤﻨﻲ داﻧﻪ ﺑﻨﺪيﻫﺎي ﻣﻨﺎﺳﺐ ﺑﺮاي ﺗﻬﻴﻪ ﻣﺨﻠﻮط اراﺋﻪ داده اﺳﺖ ﻛـﻪ ﻳﻜـﻲ‬
‫از اﻳﻦ ﭘﺎراﻣﺘﺮﻫﺎ‪ ،‬ﭘﺘﺎﻧﺴﻴﻞ ﺟﺪاﺷﺪﮔﻲ ﻣﻲﺑﺎﺷﺪ ﻛﻪ از راﺑﻄﻪ زﻳﺮ ﺑﻪ دﺳﺖ ﻣﻲآﻳﺪ‪:‬‬
‫*‪Si = 1 –Фi/Фi‬‬
‫)‪(3‬‬
‫)‪(4‬‬

‫)‪S = Si(max‬‬

‫‪=Фi‬ﺣﺠﻢ واﻗﻌﻲ ذرات ﺟﺎﻣﺪ ﻣﺮﺗﺒﻪ‪i‬‬
‫ﻣﻲﺗﻮاﻧﻨﺪ اﺷﻐﺎل ﻛﻨﻨﺪ ‪i‬ﻣﺎﻛﺰﻳﻤﻢ ﺣﺠﻤﻲﻛﻪ ذرات ﺟﺎﻣﺪ ﻣﺮﺗﺒﻪ *‪=Фi‬‬

‫‪= S‬ﭘﺘﺎﻧﺴﻴﻞ ﺟﺪاﺷﺪﮔﻲ‬
‫ﺑﺮاي ﻣﺤﺎﺳﺒﻪ ﭘﺘﺎﻧﺴﻴﻞ ﺟﺪاﺷﺪﮔﻲ ﭘﺎراﻣﺘﺮ‪ Фi‬ﻛﻪ در راﺑﻄﻪ )‪ (3‬و)‪ (4‬اﺳﺘﻔﺎده ﺷﺪه اﺳﺖ از ﺗﻘﺴﻴﻢ ﺟﺮم ذرات ﻣﺮﺗﺒـﻪ ‪،i‬‬
‫ﻛﻪ در ﻣﺮﺣﻠﻪ ﺑﻪ دﺳﺖ آوردن ﻧﺴﺒﺘﻬﺎي ﻣﺨﻠﻮط ﺑﻪ دﺳﺖ آﻣﺪه اﺳﺖ‪ ،‬ﺑﻪ ﺟﺮم ﺣﺠﻤﻲذرات ﺣﺎﺻﻞ ﺷﺪه اﺳﺖ‪.‬ﭘـﺎراﻣﺘﺮ‬
‫*‪ Фi‬ﻧﻴﺰ از ﺟﻤﻊ ﻣﻘﺪار‪ Фi‬ﺑﺎ ﺗﺨﻠﺨـﻞ ﻣﻮﺟـﻮد در داﻧـﻪ ﺑﻨـﺪي ﺳـﻨﮕﺪاﻧﻪﻫـﺎ ﻛـﻪ ﺑـﺎ ﻛﻤـﻚ دﺳـﺘﻮراﻟﻌﻤﻞ ‪ASTM‬‬
‫‪7] C28,ASTM C29‬و‪[6‬ﻣﺤﺎﺳﺒﻪ ﺷﺪه‪ ،‬ﺑﺪﺳﺖ آﻣﺪه اﺳﺖ‪.‬‬
‫زﻣﺎﻧﻴﻜﻪ ‪ Si=0‬ﺑﺎﺷﺪﺑﻪ اﻳﻦ ﻣﻌﻨﺎﺳﺖ ﻛﻪ ذرات ﻣﺮﺗﺒﻪ ‪ i‬ﺑﻪ ﻃﻮر ﻛﺎﻣﻞ ﻣﺘﺮاﻛﻢ ﺷﺪه اﻧﺪ و اﻣﻜﺎن ﺟﺪاﺷﺪﮔﻲ وﺟﻮد ﻧـﺪارد‬


حواله ایده آل می‌باشد که در واقعیت غیر ممکن است در نتیجه هر چه پتانسیل جدا شدگی منحنی دانه بندي به سمت صفق میل کند مطلوب خواهد بود.

به اساس ارای پرا اپارامتر نمو داری به نام نمودار پراکنده ارائه شده است که این نمودار برای منحنی C بیان می‌کند.

دیاگرام بی پراکنده برای منحنی دانه بندي با B,C

با حداکثر اندازه سنجش 19.5 میلیمتر در شکل زیر نشان داده شده است.

دیاگرام بی پراکنده برای منحنی 9.5

با حداکثر اندازه سنجش 9.5 و 19 میلیمتر

پتانسیل جدانعادگی برای منحنی های B,C حداکثر اندازه 19 میلیمتر با توجه به شکل 3 به ترتیب برای با B,C، نمودار منحنی های B,C، حداکثر اندازه 9.5 میلیمتر به ترتیب برای با B,C

همانطور که بیان شد منحنی دانه بندي 9.5 در حداکثر اندازه 19.5 میلیمتر می‌باشد. به این ترتیب ساخت مخلوط بین خود تراکم بودند و هر چه منحنی دانش بندي مورد استفاده بود منحنی C دیرکتر شود خواص بین تراکم مخلوط مظاهره‌ها بوده است. با توجه به این موضوع و شکل 3 منحنی دانه بندي هایی که پتانسیل جدانعادگی آنها بین 771 و 779 تا 785 تا 19 میلیمتر بوده در حداکثر اندازه 9.5 میلیمتر باند مناسب برای ساخت مخلوط خود تراکم جوانه بوده چه پتانسیل جدانعادگی منحنی مورد استفاده به حد پایین باند شده و به باند شده منحنی

بهینه پارامتر پتانسیل جدانعادگی به نهایت کافی برای تغییر منحنی دانه بندي مناسب نمایان چرا که منحنی دانه بندي باند مناسب منحنی دانه بندي می‌باشد. می‌تواند به هنون داشته که پتانسیل جدانعادگی آن به شکل تغییر منحنی دانه بندي باند مناسب برای تغییر مخلوط نمایان. به این ترتیب می‌باشد در نمودار پراکنده فاکتور دیگری نیز علاوه بر پتانسیل جدانعادگی می‌باشد می‌باشد. به هنون تناول نمودار پراکنده فاکتور تناول

در نمودار پراکنده می‌باشد که هر چه مقدار S(min) و S(max) افزایش یافته باشد. در نهایت در نمودار پراکنده می‌باشد این تفاوت برای منحنی
دی‌های روش متمایل به تریب ترکم بودن.


[2] Francois de Larrard, Concrete mixture proportion .First published 1999 by F:& FN
Spon, an important of Routledge H New Fetter Lane, London EC4P 4EE, pp. 63-75.

5. Precast/Prestressed Concrete Institute, (PCI), "Interim Guidelines for the use of Self Consolidating Concrete", Chicago, USA, 2003.


مرور طرح‌های اختلاط و خواص پن‌های خودتراکم

چکیده
در این مقاله مرور نسبتاً جامعی بر روی ترکیب و خواص مهم پن‌های خودتراکم ارائه شده است. پن‌خودتراکم برای غلبه بر مشکلات عملیات لرزاندن توسعه داده شده و به علت خواص مناسب آن، به سرعت گسترده یافته. خرم و ملات مورد استفاده در بن‌خودتراکم بايد دارای لزجت و نیز قابلیت تغییر شکل زیادی باشد و با برقراری تعادل بين این دو، ضمن تأمین قابلیت تغییر شکل و روانی زیاد از جهادگی اجزا جلوگیری شود. بنابراین استفاده از مواد افزودنی شیمیایی و پودری مختلف نیاز است. تعیین مقداری و نسبی‌های مناسب مواد نیازمند انجام اختلالاتی آزمایشی و آزمون‌های خودتراکم نظیر جریان ل، جریان اسلامی و قیف می‌باشد. محدود کردن درصد سنجشگان به همراه استفاده از مقادیر زیاد پودرهاي مخلوطی و گلخته مناسب‌ترین روان کننده‌برای پوستی‌های این خودتراکم است. جرم زیاد پودرهاي مخلوطی که در بن‌های خودتراکم استفاده مي‌شود می‌تواند اثراتی محدودی روی خواص آن داشته باشد. پودر نشک ماهی، پودر نشک ماهی مناسب فاقد حلال‌های محکم و کاهش قابل ملاحظه دارد می‌تواند باعث بیشترین مزایا و کاهش مسئله‌های خودتراکم در برای آش بیشتر به پن‌های معمولی شود. این استفاده از نتایج آزمون و تجربیات موجود روی پن معمولی را برای این نوع بن نامناسب می‌سازد.

کلیدواژه‌ها: پن، خودتراکم، طرح اختلاط، پودر

1- مقدمه
یکی از پارامترهای مهم برای رسیدن به پن با دوام مناسب، مترکم کردن بن است. برای این منظور اغلب نیاز است تا از عملیات لرزاندن با وبره استفاده شود. با کاهش تخلخل و هواه درون بن، مقاومت لازم به دست آمده، دوام افزایش یافته و از شکل گیری بین معیوب جلوگیری شود. عملیات لرزاندن از مشکلات اساسی در این صنعت به شمار می‌روند. مشکلات مختلف مانند کمپیو نسبی کارگران ماهی، سهولانگاری، افتادگی، بهبودی جسمی و روانی ناشی از لرزاندن و با پوششی دسترسی مناسب در حالی که دارای ترکیب زیادی از میلدرها هستند. باعث می‌شود تا عمل لرزاندن به طور کامل و صحیح انجام نگردد، در نهایت مشخصات مطلوبی از بن

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یکی از مباحث عمومی که در دنیای علمی چنین اتفاق می‌افتد، عدم وضعیت مناسبی است که باعث ناخوشایندی می‌شود و نتایج منفیی در عملیات اجرایی تهیه و به رفتار سازند.

این موضوع، در محیطی می‌تواند یک سیستم SCC تبیین این تلاش‌ها و بوده است. استفاده از این تهیه و نتایجهای زیادی روابط بین خودترکام و دلایل گسترش نسبتاً در مصارف بهبود در دسترس قرار گرفته است. بررسی مقدماتی و تجربیات خودترکام، در این زمینه باعث بهبود در دقت جدیدی از طریق مدارکی و اگزامیناتوری فردی، به روند در این زمینه افزایشی در آن مورد است. به طور خاص، در حد مینیمم، افزایش میزان این برای این اتفاقات اثبات شده است، ولی جنگجویی از این اتفاقات بسیار در حذف این اتفاقات آن است. به کمک این سیستم، گزارش‌های هزاران روانکار، در این زمینه افزایشی در این اتفاقات است. به طور خاص، در حد مینیمم، افزایش میزان این برای این اتفاقات اثبات شده است، ولی جنگجویی از این اتفاقات آن است. به کمک این سیستم، گزارش‌های هزاران روانکار، در این زمینه افزایشی در این اتفاقات است. به طور خاص، در حد مینیمم، افزایش میزان این برای این اتفاقات اثبات شده است، ولی جنگجویی از این اتفاقات آن است. به کمک این سیستم، گزارش‌های هزاران روانکار، در این زمینه افزایشی در این اتفاقات است.
استفاده از پنده توانمند و خودتراکم (به شرطی که عبارت سیمان در آن کاهش یابد) یکی از راههای مؤثر برای کاهش دی اکسیدکردن، کاهش مشکلات گرم شدن زمین و خفظ محتوی زیست است. زیرا در این بینه سیمان به طور مؤثرتری استفاده شده و نیز جایگزینی درصدی از سیمان به وسیله پرکندگی صورت می‌گیرد.

۳- تاریخچه بین خودتراکم

بین خودتراکم نخست در سال ۱۹۶۸ توسط اوکامورا (Okamura) و آرامیشی‌هاي اساس در نشان داد که نتیجه توکیو توسط اوکامورا و میکاوا (Meakawa) برای توزیع این بین سرعت گرفت [۵-۷]. اولین تألیف در سال ۱۹۸۸ با استفاده از مواد و مصالح موجود در آزمایش‌های شد و نتایج مناسبی از نظر جمع‌شده‌گی ناشی از یک کمی و سخت شدن. گرامی هیدروژن، سختی و سایر خواص به دست آمد و در ۸ و ۹]، بین توانمند را به عنوان یکی از راه‌های کردن گزاره مقاومت و دوبار بالا در اثر نسبت آب به سیمان افزایش یافت. باید گزاره نام نوع بین توسط اوکامورا و همکارانش غیرتایج و تحت عنوان "بین خودتراکم توانمند" به عنوان "بین خودتراکم" [۱۰ و ۱۱] تفاوت بین های خودتراکم و توانمند در این امکان بین توانمند، جریان‌پذیری پنده تا حدی بهبود یافته است. اما تفاوت نمی‌تواند تحت وزن خود، قلب، یا واقع بین میکرواها یا بردن به عبارت سیمان به موجبی لوش زیاد.

مائلات اوکامورا و اوکامورا در کنفرانس‌های مختلف بین المللی، توجه به بین خود تراکم را افزایش داد [۱۲ و ۱۳]. در سال ۱۹۹۶ بین های ساکورا که این ایجاد در پوروزه‌های SA همچنین و حواری بین سیاستگذاری SCC در پوروزه‌های SA نامگذاری، و اعمال در پردازش‌های SA در سال ۲۰۰۵. [۱۴] منشتر. صنعت بین پیش‌ساخته SA در امریکا نیز به سال ۲۰۰۰ بین خودتراکم را برای ساخت انواع SA و غیر‌ساختی SA و پالاسی فراخوان مورد استفاده قرار داد. کاربرد بین خودتراکم در ساخت قطعات پیش‌ساخته SA پنده نبوده است، بعضی نام‌ها در سال ۲۰۰۰ به طور بسیاری در سال ۲۰۰۰، همه آنها در این اثر در صنعت بین امریکا نیز با تبادل عمیق در حال افزایش است [۱۵].

۴- خودتراکم و مفهوم جگالی ترکم

جگالی تراکم مفهومی نمایشگر برای دسته‌بندی بینهای توانمند و خودتراکم است. جگالی تراکم به معنای "نسبت
حجم مواد جامد به حجم توده‌ای ذرات جامد قابل قسمت تراکم سیگناته و سیمان بحث نمود. تراکم سیگناته مستقیماً با اندازه گیری دیگری که اندازه‌گیری قابل قسمت تراکم سیگناته باعث نشان می‌دهد که سیگناته بیشترین پیکدهرگاهی فشاری شده بوده است. در نهایت سیگناته که با پیکدهرگاهی کمتر از 0.07 نموده است، این نشان دهنده یک روش پیکدهرگاهی کوچک در مقایسه با سایر روش‌هاست. این می‌تواند نشان‌دهنده باشد که در ارائه سیگناته به عنوان یک آزمون سیگناته شناخته شود.

BS [20 آمد این چنین می‌تواند با اندازه‌گیری مختلف و یا نسبت مناسب در مخلوط وجود داشته باشد. دلایل این افزایش قوافل بین دانه‌ها و بین دانه‌ها برجستگی ویژه و سطح توسط ماده ریزین باید به اضافه‌ی مقدار ثابت ماده سیمانی، خمیری که اضافه بر مقدار لازم برای کنترل خاک وجود دارد. باعث می‌شود تا دانه‌ها به نحو بهتری با خمیر پوست آن توزیع و جریان بهتری به دست آید. بنابراین این نتیجه سیگناته تأثیر به سازگاری در تقاضای خمیر و کارایی مخلوط دارد. کاهش تقاضای خمیر، علاوه بر بهبود کارایی، باعث پیوست کردن ابعاد بهترین تری می‌شود. بنابراین نتیجه تمام این‌گونه از نظر زیست‌شناسی و طراحی‌های فلزی ناشی از اثر جدایی‌ها باعث می‌شود تا چگالی تراکم کاهش یابد.

استرورون [197] یک مدل کامپیوتری برای تعیین اندازه تراکم انتظار می‌دهد که این مفهوم چگالی تراکم را در می‌تواند به مواد سیمانی نیز سطح داده نمگونه که تراکم سیگناته نقاطی سیمان را مشخص می‌سازد. تراکم مواد سیمانی نیز تقاضای آب را تعیین می‌کند. مواد سیمانی کونگکوان در هنگام سیگناته مختلف سکته را بهبود یک‌پایه و حریز می‌کند. فنج [209] شکل حاضر که مخلوط کننده سیمان در هر دو اندازه‌گیری مشاهده می‌شود که افزودن میکروسیل به مواد خالق توجه چگالی تراکم سیگناته را افزایش می‌دهد. این نشان داده که در یک افزودن میکروسیل به ماده سیمانی (HS-SCC) در جریان (Kwan) در خود ماده سیمانی (HS-SCC) در یک میکرون همکاری می‌کند با افزودن میکروسیل به ماده سیمانی باعث افزایش میکروسیلیس باعث پیروی می‌شود.

وی این بهبود کارایی را به سیستم سیال و بودن میکروسیل را نشان داد که با در میکروسیل سیمان را بهبود زیر در روند خمیر موجود می‌باشد. هیلا [246] نشان داد که مخلوط سیمان سیمان با خاترین باضری بهبود در اندازه و سیستم سیمانی باعث می‌شود تا افزایش مکاتب و تغییرات در درسکابه‌داری و باعث شود بهبود می‌یابد.
میکروسکوپ‌های نوری و الکترونی بررسی کرد. مشاهده شد که تراکم اثر تابعی بر روی تخلخل و بهبود منطقه اختلال دارد. بنابراین تخلخل منطقه اختلال در بین معمولی برتره‌ها شد. عموماً بیشتر از بین خودتراکم است که نیروی بیانی فشاری کمتر و نفوذ بیشتر اکسیژن می‌شود. در عین حال، مقادیر کمتر جریان سیمان و اندازه‌های نزدیک به محدوده در بین معمولی، این اثرات را کاهش می‌دهد. اثرات تأمین پالم‌رشه‌های مختلف، نیروی تقلیل کارایی و تاثیرگذار منطقه اختلال، بررسی میکروسکوپی موضوع به صورت یک روش تکرارپذیر را دشوار و پیچیده می‌سازد [28].

در مورد اندازه‌گیری چگالی تراکم سیمان در مراجع مختلف بحث شده است [19, 27-30]. برای بین‌های توانمند و خودتراکم توصیه شده است که چگالی جریان با افزودن فوق روان کننده، اندازه گیری شود [19].

افراش چگالی تراکم، علاوه بر مقاومت، باعث بهبود کارکرد بین نیز می‌شود و با افزایش چگالی، سیمان، قابلیت جریان بافت خمیر بهتر یافته و تبدیل مقاومت بین در برابر جوشادنی‌های افزایشی می‌یابد. این مشخصات به دست‌یابی به خاصیت خودتراکمی کمک زیادی می‌نماید.

از امواج‌های واگن [19] به‌ویژه اساسی قابلیت جریان بافت خمیر سیمان با افزایش چگالی تراکم را به خصوص در سیمان‌های بی‌پای ای تری و پایین مخاطر اسلام داده است. افزایش سیمان، تغییر داد شکل مقادیر جریان اساسی در پایین‌های مختلف مخلوط و آب به سیمان در آزمایش‌های وی نشان می‌دهد. نسبت‌ها در این شکل حجم‌هایی مقرر بالایی و پایینی مخاطر اسلام بی‌پاین ممکن است. و با افزایش افزایش آن در بین، یک طرح اختلال سه مرحله‌ای شناخته می‌شود. مقادیر جریان خمیر سیمان، سوخته و بین را پیش‌بینی می‌کند. در مرحله نیز در نظر گرفته می‌شود که چگالی تراکم سیمان، قابلیت چگالی تراکم را تعیین می‌کند. در آزمایش‌های اسیدی، مقادیر ماندگار، تغییرات و بدون سوخته بین نیز جریان ذیلی مقادیر اساسی دارند.

این در مراحل ممکن است در نظر گرفته می‌شود که چگالی تراکم سیمان و سگنودهای ریزتر از mm 1/2 تقاضایی خمیر را مشخص کرد و خمیر اضافی مانند تعیین کننده جریان ذیلی مات است. در مرحله سوم بین نیز مانند ماده، اضافه سگنودهایهای درشت‌تر از 1/2 بی‌پاین می‌شود که در ان چگالی تراکم سگنودهایهای زیرتر از mm 1/2 تقاضایی مات را مشخص می‌کند. در این ماده اضافی اعمال تعیین کندنی کارایی مخلوط است. و در این گزارش نیز که تا در ان نمونه‌های بین که جداسازی مشاهده شد. دانه‌های زیرتر از 1/2 بی‌پاین، همراه با خمیر باقی مانند.

شکل 1 - مقادیر جریان خمیر سیمان در نسبت‌های مختلف ماده و آب به سیمان [19].
5- طرح اختلاط‌های بین خودتراکم

اصولًا یک نظر بیشتری به سبب اجازه‌ای که دارایی چگالی‌های مختلف هستند، می‌تواند این بخش‌ها را در نظر گرفته باشد. مناسبی است بگوییم که خودتراکم انرژی جریان‌های متفاوت به طور رایج استفاده از بالاترین لزجی که قابل کار راهنمایی ندارد. در این صورت، قابلیت تغییر شکل کاهش یافته و به عملیات لزج‌دهان نیاز است. اما برای رسیدن به محاسبه، از آزمون‌های خودتراکمی از آزمون‌های مختلف این آزمون‌ها را شامل بوده و از این طریق در داده‌های به دست آمده، نتایج بهبود در انرژی جریان‌های متفاوت، مقدار کاهش در داده‌های آزمون‌ها و این افزایش افزایش در حال واقع می‌باشد. در مقایسه، نتایج این افزایش در حال واقع می‌باشد.

شدو [31].

بدین‌وایه که طرح اختلاط‌های ویژه به ضرورت به روز می‌آید، دوام و شرایط محیطی و غیره دارند. مقادیر مناسب مواد را تا نمی‌توان بدون اختلاط‌های متفاوت، تا به طور گرفته، بنابراین بالا گرفته می‌باشد. این روش‌هایی که در این موارد از آزمون‌ها در راه می‌بردند، و تغییر کاهشی در انرژی جریان‌های متفاوت، به دست آمده‌های مربوط به نتایج بیشتری همراه است.

(معادله 1) شاخه‌ای که پیش از شکل، درجت بیشتری آزمون‌های خودتراکمی و همچنین، این روش‌ها می‌توانند در بررسی موارد زیر، تغییر یابند.

$\Gamma_c = \left( S_{nf} - S_{nf}^* \right) / S_{nf}^*$

که در آن $S_{nf}$ و $S_{nf}^*$ قطع‌هایی از آزمون‌های خودتراکمی و $\Gamma_c$ قطع محرک اسلاپ‌های است.

(معادله 2) شاخه‌ای که تغییر یابند:

$R_c = \frac{10}{t}$

که در آن $t$ زمان اندازه‌گیری شده بر حسب نظرهای آزمون‌های خودتراکمی. این نظرهای ممکن است از میان قیف‌های این بزرگی تغییر نماید. آزمون‌های خودتراکمی و قیف، می‌توانند برای رسیدن به محاسبه، به‌طور مختلف استفاده از آزمون‌های خودتراکمی یا بهترین مورد وجود داشته که نشان می‌دهد. این شاخه‌ها می‌توانند برای تغییر یابند. تغییر‌های محلی به این ضریب نسبت به به پیوست و مقدار فرق‌های کنده و برای دست آوردن از این قابلیت تحت شرایط مختلف مناسب، استفاده شوند. اکمراه و اولی شی [6] ارتباط بین وکتورهای خودتراکمی و نتایج آزمون‌های خودتراکمی را به کردمان.
روش‌های طرح اخلاط

خودتراکمی، طریقه‌ای مشخص برای ایجاد ذرات و روندهای فیزیکی در محلول‌ها است. این روش برای ایجاد ذرات و روندهای فیزیکی در محلول‌ها استفاده می‌شود. این روش برای ایجاد ذرات و روندهای فیزیکی در محلول‌ها استفاده می‌شود. این روش برای ایجاد ذرات و روندهای فیزیکی در محلول‌ها استفاده می‌شود.
سیمان را در قالب به وسیله فشار به اندازه دو میلی‌متری برای دست دادن، درصد سیمان، درصد سیمان و ریزی نسبت 4 به 1 را برای نسبت حجمی سیمان‌های ریز به درصد مورد نظر، اما اگر نسبت کمتر (1) از سیمان به ریز است، هم‌زمان با حذف میکرو‌پودر و پودر رویده شده در نظر گرفته می‌شود. 

شکل 3- رابطه بین نسبت آب به پودر و نسبت حجمی ریز‌دادن‌ها در ملات

شکل 2- رابطه بین میزان جریان نسبی و نسبت حجمی آب به پودر

اگری‌دیگر مخلوطی که مقدار آب به مقدار کننده هستند. تنش اصلی آب در بخش مرزی آب به وسیله پودر و سیمان‌های ریز بر حسب درصدی میزان سیمان در ملات نسبت به آب به مقدار کننده است. در نمونه یک‌پوش سیمان ریز مقدار جریان و سرعت قیف برای خیار و ملات سابقه نسبت به آب به پودر است. هر چه مقدار سیمان‌های ریز افزایش می‌یابد، نسبت تنش کاهش یافته، سرعت قیف لازم است.
نقش فوق‌روان کندن در مشابه‌ی آب آزاد است. اما مقایسه بسیاری کمتری از این ضروری است. برخلاف آب آزاد، تغییرات در سرعت قیف به‌طور محسوس گیرنده است. فوق‌روان کندن بر خلاف آب آزاد، باعث بروز اندکانی نیست. اما این امر در آب‌های دیگر ممکن است.

اکنون، مطابق با امواج‌های آب و درفتی فوق‌روان کندن، موارد اختلاف، تغییرات در سطح آب و باعث نرخ فیلتر بروز می‌کند.

شکل 1- رابطه بین نسبت آب آزاد به پودر و میزان سختی

شکل 5- رابطه بین نسبت آب آزاد به پودر و سرعت قیف

نگاه‌می‌شود که تغییرات در سطح آب به‌طور محسوس تغییر می‌کند. اما در این موارد، این تغییرات به‌طور محسوس نیست.

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طرح، اصول ارائه شده در مطرح اختلافاتی با توجه به فضاهای تخته‌سازی، درصد وزنی تابو، نسبت وزنی آب به پودر و حجم پودرهای به حجم ملات را بهتر و بررسی کرد. برای این موانع تابو، درصد حجم سنگدانه‌های درشت 3/2/ درصد حجم خمیر 3/1/ درصد پودر به مقدار 500 کیلوگرم بر متر مکعب، نسبت آب به پودر بیست و هشت است. به استداری دو مورد مشاهده شد. پودر در محدوده 415/64 تا 445/65 کیلوگرم بر متر مکعب بوده است. در نهایت با توجه به مخلوط‌های پودر، یک VMA و بودن بیش از پایان‌های مخلوط‌های با کمتر قدر مFormData (با بکارگیری درصد پودر، مقدار درصد خمیر 30 در مقدار میانگین) شده است. نسبت‌های آب به پودر در محدوده 0/28 تا 0/28 قرار می‌گیرد.

نسبت آب به پودر در دارای اثرهای قابل توجهی بر روی خواص بنی‌های تازه و سخت شده است. اما در بنی‌های درشت آن، در مقدار پودر فقط حضور پودر در پودر و سپس، درصد خمیر در مخلوط‌های پودر، نسبت VMA و درصد سفید، درصد اثرات پودر و سه درصد اثرات پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پودر و سه درصد پو
در روش سو ابتدا با استفاده از معه‌برداری درجه تراکم، مقادیر سنگبال‌های ریز و درشت محاسبه می‌شود. این جمله معمولاً زاینده سه گروه: ۰۲۰ و ۰۱۵ و ۰۱۰ دیده‌شده حدود ۰۵ درصد و زن واحد تراکم شکل شکل‌گیری (۳۲۹) با استفاده از معادلات داده‌شده. در حالیکه یگانی شده در شکل‌گیری (۳۲۹) با استفاده از معادلات داده‌شده. در حالیکه یگانی شده در شکل‌گیری (۳۲۹) با استفاده از معادلات داده‌شده.

جدول ۱: ویژگی‌های بتن خودتراکم، بین‌شکل‌شده توسط انجماد عمران زاینده [۳۷]

<table>
<thead>
<tr>
<th>کلاس قابلیت پرسش بتن</th>
<th>شرایط ساخته‌شده (دما سایت، نسبت V/C)</th>
<th>مقدار كندنده (kg/م³)</th>
<th>حجم نقطه سنگال‌های درشت به ازار خازم</th>
<th>جریان پسیوی برای هر سال (mm)</th>
<th>مقاومت به جوانگی سنگال‌ها (کی‌گرمی)</th>
</tr>
</thead>
<tbody>
<tr>
<td>≥۴۰۰۰</td>
<td>۶-۰-۳۰</td>
<td>≥۲۵۰</td>
<td>≥۲۰/۰۳۳</td>
<td>≥۵۰/۰۰۰</td>
<td>≥۵/۰۰۰</td>
</tr>
<tr>
<td>≤۵۰۰</td>
<td>۱۰-۰-۲۵</td>
<td>≥۲۰۰</td>
<td>≥۱/۰۰۰</td>
<td>≥۱۰/۰۰۰</td>
<td>≥۱۰/۰۰۰</td>
</tr>
<tr>
<td>≥۲۰۰۰۰</td>
<td>۶-۰-۳۰</td>
<td>≥۲۵۰</td>
<td>≥۲۰/۰۳۳</td>
<td>≥۵۰/۰۰۰</td>
<td>≥۵/۰۰۰</td>
</tr>
</tbody>
</table>

دلیل شرایط بتن خودتراکم، بین‌شکل‌شده می‌باشد. درصد جاگردایی اندک و حدود ۵ درصد گرفته می‌شود.
500 و از بردن رمالید بین صورت این نسبت 47 و جهه L اسقف بود. آزمون ها و واحدهای مورد نیاز برای این
ارزیابی تحت شرط است [40].

در صورت نیاز، افزودنی VMA نیز به مخلوط افزوده شود.
روش های اخلاقی دیگر تیز توسط پژوهشگران پرستی و ارائه شده است. برخی از این طرح ها برای بررسی
امکان استفاده از مصالح ارزان محلی راهان شدن نشان داده شده که برخی دیگر نیز مطرح از ارزیابی
(Ozbay) خواه مطالعات عمیق تر با بررسی جدید پرداختن مختلف است. یکی [46 و 47 برای تعیین نسبت مخلوط
خودتراکم، از دملعبه آماری استفاده و روند انجام شده است. وی خواص خودتراکم و مقاومت شماری
زرع مخلوط ها را بر اساس آزمون های متعدد و روش های آماری با استفاده از مقادیر محاسباتی للامین در
غلطی های اصلاح کندنه لجح و فوق روان کندنه و حجم سگافل اهمیت دارد. است. ضرایب همینگی خوبی
برای روابط به دست آمده و گزارش شده است [47]. ارگانیزه هر ک از سنجش نیز بررسی شده است.

6- مواد فوق روان کندنه و اصلاح کندنه‌های لجح

دستیابی به بین خودتراکم بدون استفاده از مواد افزودنی شیمیایی جدید به این صورت امکان‌بندی نمود.
پژوهش‌های قراراگاهی روي اثر مواد افزودنی فوق روان کندنه و اصلاح کندنه‌های لجح در بین خودتراکم صورت

از کل و دوکان [36] اثر یکنده فوق روان کندنه از نوع کیلوامتر N-Vinyl ار رای خواص باز های نازه و
سخت شده مطالعه کردند. برای مخلوط به نسبت های آپ به سیمان بین 20 تا 45، قطر اسلاپ بین 500 و 700 میلی متر و مقاومت فشاری نمونه در 28 روز، بین 20 تا 45 میکرون سطح تغییر کردند. آزمایش ها نشان داد که مشخصات 7 فی به سرعت قیف به بیشتری در جریان خمیر سیمان با یکند پایین تر از سرعت فوق روان کندنه تغییر تابث بوده، مستقل از نسبت آپ به سیمان است. مقدار بالای فوق روان کندنه منجر به نسبت
پایین تر از سرعت فوق روان کندنه به سرعت جریان شد. این نسبت به عنوان شاخص برای تعیین اثر فوق روان کندنه روی قابلیت جریان خمیر سیمان و لجح آن، از نظر دست یافتن به خودتراکم یکپارچه شد، رابطه بین
مقدار فوق روان کندنه و آن آن به سبب نوع سیمان با افزودنی شیمیایی متفاوت انت، تا بسته است که نسبت
شاخص تغییر شکل به شاخص از برای ملات در نسبت اثر فوق روان کندنه به بود، مستقل از نسبت آپ به
پودر است [6].

سوآ و پلیمر [49] معمولاً در فوق روان کندنه از نوع پلی کربو اتیلن ار و نفتین فرمالدید سولفور را در یک تی
40 گلاکسی مایکالک با یک درک مقداری و گزارش کردند. پلی کربو اتیلن ار فقط به انتهای سیم سوم نفتین فرمالدید
سولفور به رای سری ان اسلاپ از 45 به 150 میلی متر ناز بود و تأثیر بهتری هنگام داد. از نظر دیگر، مواد اصلاح کندنه لجح، بی‌مهری به دلایلی در آپ با ون مولکولی بالا هستند که برای بالا رساندن

سومین کنفرانس بین‌المللی بن و توسعه، 1388
ازجت آب استفاده می‌شود. این ترکیبات باعث افزایش پیوستگی بین تازه شده، زمینه جذب‌سازی و آب افتادگی را کاهش می‌دهند [51]. اگرچه استفاده از یک طرح اختلاط مناسب با مقدار کافی پودر می‌تواند به راه آمدن شاددار کننده لزوم را برطرف سازد، اما در برخی اوقات نیاز استفاده از آنها ضروری یا مفید است. تاکادا (Takada) و دیگر [53] از اندازه‌گیری و پارس [53] خیاط [Khayat] و فیروز [Ferraris] پایه پی‌سازارهای بر روی بین خودتراکم را در شرایط مختلف تحقیق کرده‌اند. افزودن فوق باعث افزایش نسبت آب به سیمان است و جریان استلام 65 میلی‌متر و زمان قیف 4-11 ثانیه بر اثر استفاده از درصد آزاده افزودن حلال در 50 درصد افزودن محصولات پایه سیمانی را در کرده است. در این مورد، طبق‌به‌پژوهی، نیز به ویژه فعالیت این استحکام‌کننده بررسی شده و اثرات آنها بر بهبود ضایعات رولوژیکی و جغیره سیستم‌های پایه سیمانی و نیز اثر روی خواص بین تازه و سخت شده، حث شده است. از جمله طبق‌به‌پژوهی‌های میل و واگنام (Mailvaganam) [55] و کاوالی [56] مور شد است.

7- حرارت هیدرسیون
بتنهای خودتراکم نسبت به بتن‌های معمولی دارای مقادیر زیادی پودر هستند. برخی از بودرها خشکی و برخی دارای واکنش هیدرولیک هستند. اگر پودری مشابه تخیلی در شرایط نوترون‌یابی روان می‌گردد، بتن‌های هیدرولیک خشکی بر روی قرار آدم‌هایی حاوی از مقدار بتن‌های خودتراکم کاررئی انجام شده و نیاز به مطالعات و تحقیقات پیشرفته در مرحله تکنیکی است [57].

پوپ (Poppe) [57] آزمایش‌های ایزوترومال و آداباتیک هیدرسیون را با ارزیابی حرارت تولیدی از دو نوع بتن خودتراکم انجام داده و نتایج آن را با دو نوع بتن معمولی مقایسه کرده است. آزمایش‌های ایزوترومال روی مخلوط بتن‌های خودتراکم و پودرهای مرطوب کرده، به سیمان‌های خودتراکم کردن شمای دهات، شیل‌سازی و پودرهای معدنی. برای بالای 90% حجم زیر 50 میکرون بود. محتوی شده تولید حرارت بر حسب 48/7 به‌عنوان نتایج از زمان برای آزمون‌های ایزوترومال ارائه شده است. برای تأمین انواع سیمان، شدت تولید حرارت نمونه‌های خودتراکم کارکرد بتن‌های پودر پوش‌شده از آن برآورد شده است. این مقدار پودر سنگ بتن پوش شده، حداکثر شدت حرارت آزاده فیکتیو به دست آمده است. این مقدار به پودر تولید ریزتریت، کنگری مشابه می‌باشد، به‌طوری که اول است. یعنی برای افزایش پودر تولید حرارت، اگر در شدت تولید حرارت افزایش شده و با ادامه افزایش کوارتزیت، مجدداً شدت تولید حرارت کاهش یافته است. (شکل‌های 6 و 7). نهایتاً زمان رسیدن به حداکثر شدت تولید حرارت در نمونه‌های دارای کوارتزیت نسبتاً بسیاری از نمونه مرجع
می‌پانند. در نمونه‌های خودترکم‌داری پودر‌سازی‌های آهک پروپیونیک دارای نیز به‌طور کاملاً به‌طور صورت‌داری نیز در کل تأثیر آمادگی‌های ایزوترال را تأثیر دارد [57].

در ادبیات علمی موضع، اثر پودر‌سازی‌های آهک در هیدراسیون سیمان، اغلب اوقات به اثر آن در شدت تولید حرارت محدود شده است. برخی نویسندگان بر این قائل هستند که سیستم‌های پودری، زمانی که یک مقدار بالا کاهش نیافته و فرآیندهای هیدراسیون در ساختار اولیه سریعی می‌شود، کادرهای دووال [58] پیشنهاد داده‌اند که دانه‌های آهک به عناوین سایت‌های هسته‌سازی غیرهمگونی برای رسوب هیدرات‌های آهک و بیشترین شدت عمیق را دارند.

در برخی منحصراً این قالب [57] یک پک دوم نیز مشاهده شد. یک فرض برای علت ظهور این پک، می‌تواند این باشد که در سیمان‌های تولیدی اکثر آهک C3A در سیمان‌های جدید (حدود 2/7) باعث آن شود. این تبدیل ارتباطی با سیستم‌های میتواند به عنوان سایزهای هسته‌سازی غیرهمگونی برای رسوب هیدرات‌های آهک و شدت بالای شفاف است.

Schema 7، شدت تولید آهک در مخلوط سیمان CEM I 42.5 R و CEM I 52.5 R.

Schema 6، شدت تولید آهک در مخلوط سیمان CEM I 42.5 R و CEM I 52.5 R.

پس از دریافت اثرات حشرات هیدراسیون در سیمان، نتایج پک دوم تأثیر می‌کند. این دو نظر به ترتیب نتایج در مورد حشرات بر این تأثیر بر سیمان‌های پودری اشک‌های سیستم‌های برای C3S و C3A مطرح کرد. این هسته‌های هیدراسیون برای C3S و C3A می‌توانند به سیستم‌های پودری و با دمای نشانه‌های پودری، را بیشترین شدت عمیق را دارند.

پک CEM I 52.5 R سیمان LA HSR در محقق این مطالعات ویژه‌ای (Ye) [59] اثر پودر سنگ آهک بر روی حرارت هیدراسیون در بین خودترکم‌ها را نشان می‌دهد. مطالعات این نتایج پک را تأثیر می‌کند. ۲۰ درصد از مورد حشرات در مورد حشرات به سیستم‌های برای C3S و C3A مطرح کرد. این هسته‌های هیدراسیون برای C3S و C3A می‌توانند به سیستم‌های پودری و با دمای نشانه‌های پودری، را بیشترین شدت عمیق را دارند.
8- مقاومت فشاری

برعلاً ۶۰۰ اثر آفتابی، پودر سنگ آهن در بین خودتراکم با عبار سیمان ۴۵۰ کیلوگرم به متر مکعب و نسبت آب به سیمان ثابت ۰/۵ را بررسی کرد. آفتاب های مقاومت فشاری، با آفتاب پرکندن خشکی تا حد کمی مشاهده شد. به طور متوسط با ۶۴/۵/۰% آفتاب پرکندن، کمتر از ۵/۰/۴% آفتاب مقاومت فشاری، SCC و روزه شاهد شد. در جدول دوم، مقاومت فشاری مدل‌ها، از روزه شاهد ۳۸ روزه اثره شده بین ۲۰ تا ۱۰۰ مگاپیکسیال متغیر به دو خوشه که از این میان حدود ۶۸۰% طرح‌های اختلاف دارای مقاومت بالاتر از بودن. به عبارت دیگر، اکثر طبق مدل خودتراکم با مقاومت‌های مشابه با انتخاب می‌تواند، و با نسبت آب به مدل با وجود دارد. در اینجا، از خوشه بین‌های عمومی، مقاومت آسرسی به سیلیر چسباندن، و به‌طور کلی به جای سادات، تنریه بسته شده، به علت مشکلات نرم‌اند در فضاهای بین قابلیت‌ها و نظریات، ضمناً اصلی باعث ضعف شده و مورد مطالعه است که استفاده می‌شود. در اینجا، انتظار می‌رود که مشخصات بین سخت‌شده خودتراکم تندیک و مشابه با پن و توان‌می‌باشد.

9- دوام

در بین های عمومی (پای اریج) عمل تراکم به وسیله آب رژیم صورت می‌گیرد، که از این نکته است. در حالت از رژیم داخی، با فرض این که عملیات به خوبی صورت گیرد، حجم در داخل محدوده‌های که تحت تأثیر آب‌زدایی است، تراکم یکسان در دبایش می‌شود. در عمل دوام یک بعدی به انتخاب مناسب دقت در بنی ریزی، تراکم پرداخت نهایی و عمل آوری دارد. عدم تراکم مناسب به سوی بافت‌های بافته و فسفرات در فضاهای بین قابلیت و نظریات، اصلی باعث ضعف می‌شود. در اینجا، بسته شده، به عبارت دیگر، عوامل مهتری باید توجه به انتزاع خود، از منافعی که اجاه ورود به آن را می‌دهد، به پن تغییر می‌شود. به عبارت دیگر عملیات ناقص با ساخت‌سازی رژیم باعث تقویت می‌شود که به عنوان ابزاری، عامل تاثیرگذار بیشتری بر روی خودتراکم و تراکم خوشه‌های داشت. انتظار می‌رود که تراکم در بین خودتراکم با خوشه مناسب، علی‌رغم از آن، تراکم به دوام، و همچنین به پیگیری‌های این خوشه‌ها و عوامل مهتری و خودتراکم بوده است. اما به احتمال زیاد به اینکه همواره با مقدار خشکی خودتراکم پردازش می‌شود. این ها با اینکه همواره با مقدار خشکی خودتراکم پردازش می‌شود.

مراجع:
[۱] بیانیهچهره گروه علوم کشاورزی (که اینجا آمده بودند) برای دانستنی که به احتمال زیاد به جمله ماه‌هایسیگما ۹۰ بین توان‌می‌باشد و انتظار می‌رود که تراکم در بین خودتراکم با خوشه مناسب، علی‌رغم از آن، تراکم به دوام، و همچنین به پیگیری‌های این خوشه‌ها و عوامل مهتری و خودتراکم بوده است. اما به احتمال زیاد به اینکه همواره با مقدار خشکی خودتراکم پردازش می‌شود. این ها با اینکه همواره با مقدار خشکی خودتراکم پردازش می‌شود.
پرورش نمونه‌های مختلف بین خودتراکم و بین معمولی لرزانه شده با توان 400 روز در معرض محصول سدیم (Na2O) 18٪ در اب مقطور، قرار داده شده که به طور انجام‌داده‌های کیوری، و نیmph است، ارزیابی کرد، اثر روش اخلاط به، با در نظر گرفتن دو روش مختلف، ازمایش چه اول روش معمولی که در آن تمام مصالح شکست یا آب برای دو دقیقه مخلوط شده و سپس فوق روان کننده اضافه شده و اخلاط برای ۱/۳ دقیقه دیگر ادامه یافت، در روش دوم، اخلاط تمام مصالح به جز پودر سنگ اهن برای ۷۵ دقیقه و سپس اضافه کردن پودر سنگ اهن، واکنش اتمی تر، مثلاً به آب برای ۲۴ دقیقه مورد استفاده قرار گرفت، روش دوم وقتی نیست. آزمایش‌ها نشان داد که میزان حساسیت بیشتر پودرسنگ اهن به تهاجم سدیم، در صورت استفاده از بین خودتراکم با مقیار زیاد پودرسنگ اهن در مناطق دیگر و تهاجم سدیم درنگنده، دو برابر کاهش یافت. بنابراین پرورش این کننده به دلیل کاهش میزان حساسیت بیشتر پودرسنگ اهن در مناطق دیگر و تهاجم سدیم است، از بین خودتراکم با مقیار زیاد پودرسنگ اهن استفاده نشود.

ایران‌زار (Irassar) [62] اخیراً تهاجم سدیم بور رانی مصالح سیمانی حاوی پودرسنگ اهن را مورد نموده است. این مطالعات آزمایشگاهی روی نمونه‌های خیب، ملات و پنیر با ترکیبات مختلف در برابر تهاجم محلول‌های سدیم و سدیم می‌باشد. سدیم مختلف در پودر دوم نشان داد، پودر در برابر تهاجم محلول‌های پودر سنگ اهن در جامه‌ها که آب بی‌سفید به دفع شیء ۴٪ کرم بر لیتر به تهجر می‌گردد پودر سنگ اهن در جامه‌ها که آب بی‌سفید به دفع شیء ۴٪ کرم بر لیتر به تهجر می‌گردد.
‫––––––––––––––––––––––––––––– ﻣﺮور ﻃﺮحﻫﺎي اﺧﺘﻼط و ﺧﻮاص ﺑﺘﻦﻫﺎي ﺧﻮدﺗﺮاﻛﻢ ‪129 /‬‬

‫اراﺋﻪ ﻧﻜﺮده اﺳﺖ‪ .‬ﺗﺮﮔﺎرد و ﻛﺎﻟﻴﻨﻮوﺳﻜﻲ )‪ 67] (Trägård and Kalinowski‬و ‪ [68‬روي دوام ﺑـﺘﻦ ﺧـﻮدﺗﺮاﻛﻢ ﺑـﺎ‬
‫ﭘﻮدر ﺳﻨﮓ آﻫﻚ در ﺑﺮاﺑﺮ ﻣﺤﻴﻂﻫﺎي داراي ﺳﻮﻟﻔﺎت و ﻳﻮن ﻣﻨﻴﺰﻳﻢ ﺗﺤﻘﻴﻖ ﻛﺮدﻧﺪ‪ .‬ﻧﻤﻮﻧﻪﻫﺎي داراي ﭘﺮﻛﻨﻨﺪه ﺳـﻨﮓ‬
‫آﻫﻚ داراي دوام ﻛﻤﺘﺮي ﺑﻮدﻧﺪ و ﻣﺨﻠﻮطﻫﺎي ﺑﺎ ‪ 50 kg/m3‬ﭘﻮدر ﺳﻨﮓ آﻫﻚ ﺿﻌﻴﻒﺗﺮ از ﻣﺨﻠﻮطﻫـﺎي ﺑـﺎ ‪kg/m3‬‬
‫‪100‬و ﻳﺎ ﺑﺪون آن ﺑﻮد‪ .‬ﭘﺲ از دورهﻫﺎي آزﻣﻮن )‪ 22‬و ‪ 48‬ﻣﺎه( ﻧﻤﻮﻧﻪﻫﺎ دﭼﺎر ﺗﺨﺮﻳﺐﻫـﺎ و ﭘﻮﺳـﺘﮕﻲﻫـﺎي ﺳـﻄﺤﻲ‬
‫ﺷﺪه و ﺗﺮﻛﻴﺒﺎت ﺗﺎﻣﺎﺳﻴﺖ‪ ،‬ﮔﭻ و اﺗﺮﻧﺠﻴﺖ در آﻧﻬﺎ ﻳﺎﻓﺖ ﺷﺪ‪.‬‬
‫‪3‬‬
‫ﻓﺮﻳﺒﺮت و اﺷﺘﺎرك )‪ [69] (Friebert and Stark‬ﻧﻴـﺰ روي دوام ﻧﻤﻮﻧـﻪﻫـﺎي ﺧـﻮدﺗﺮاﻛﻢ داراي ‪ 200kg/m‬ﭘـﻮدر‬
‫ﺳﻨﮓ آﻫﻚ ﻣﻄﺎﻟﻌﻪ ﻧﻤﻮدﻧﺪ‪ .‬ﻧﻤﻮﻧﻪﻫﺎ ﺑﺮاي ‪ 4‬ﻣﺎه در ﻣﻌﺮض ﻣﺤﻠﻮل ﺳﻮﻟﻔﺎت ‪ 33/8‬ﮔﺮم ﺑﺮ ﻟﻴﺘﺮ ﻗﺮار داده ﺷﺪ‪ .‬ﻧﺘـﺎﻳﺞ‬
‫ﻣﻘﺎوﻣﺖ ﻛﺸﺸﻲ ﻧﺴﺒﻲ روي ﻧﻤﻮﻧﻪﻫﺎي ﺧﻮدﺗﺮاﻛﻢ و ﺷﺎﻫﺪ ﻣﺸﺎﺑﻪ ﺑﺎ ﻫﻢ و ﻣﻌﺎدل ﺗﻘﺮﻳﺒﺎً ‪ 0/7‬ﺑﻪ دﺳـﺖ آﻣـﺪ ﻛـﻪ ﺑـﻪ‬
‫ﻧﻔﻮذﻧﺎﭘﺬﻳﺮي ﺧﻮب ﻧﻤﻮﻧﻪ ﻧﺴﺒﺖ داده ﺷﺪ‪.‬‬
‫ﻧﻔﻮذﭘﺬﻳﺮي ﺑﻴﺸﺘﺮ و ﻛﻤﺘﺮ ﻧﺴﺒﺖ ﺑﻪ ﺑﺘﻦﻫﺎي راﻳﺞ‪ ،‬ﻫـﺮ دو ﺑـﺮاي ﺑـﺘﻦ ﺧـﻮدﺗﺮاﻛﻢ ﮔـﺰارش ﺷـﺪه اﺳـﺖ‪ .‬ژو )‪(Zhu‬‬
‫]‪ [7069‬دوام ﺑﺘﻦ ﺧﻮدﺗﺮاﻛﻢ را در ﻣﻘﺎﻳﺴﻪ ﺑﺎ ﺑﺘﻦ راﻳﺞ ﺑﺎ ﻣﻘﺎوﻣﺖ ﻣﺸﺎﺑﻪ‪ ،‬از ﻃﺮﻳﻖ اﻧﺪازهﮔﻴﺮي ﻧﻔﻮذﭘﺬﻳﺮي اﻛـﺴﻴﮋن‪،‬‬
‫ﺟﺬب ﻣﻮﻳﻴﻨﻪ و ﻧﻔﻮذ ﻛﻠﺮﻳﺪ ارزﻳﺎﺑﻲ ﻛﺮده اﺳﺖ‪ .‬وي دو رده ‪ C-40‬و ‪ C-60‬را ﺑﺎ ﻣﻘﺎوﻣـﺖﻫـﺎي ﻣﺸﺨـﺼﻪ ‪ 40‬و ‪60‬‬
‫ﻣﮕﺎﭘﺎﺳﻜﺎل‪ ،‬ﺑﺮاي آزﻣﻮﻧﻪﻫﺎي ﻣﻜﻌﺒﻲ‪ ،‬ﺗﻬﻴﻪ ﻛﺮد‪ .‬ﺑﺮاي ﻫﺮ رده‪ ،‬ﺳﻪ ﺳﺮي ﻣﺘﻔﺎوت از ﻣﺨﻠﻮطﻫﺎي ‪ SCC‬و دو ﺳـﺮي‬
‫ﻧﻤﻮﻧﻪﻫﺎي ﺑﺘﻦ راﻳﺞ ﻟﺮزاﻧﺪه ﺷﺪه‪ ،‬ﺗﻬﻴﻪ ﺷﺪ‪ .‬ﻃﺮح اﺧﺘﻼطﻫﺎي ﺑﺘﻦ ﺧﻮدﺗﺮاﻛﻢ‪ ،‬ﺷﺎﻣﻞ ﭘﻮدر ﺳﻨﮓآﻫﻚ‪ ،‬ﺧﺎﻛﺴﺘﺮ ﺑﺎدي‬
‫و ﻳﻜﻲ ﻫﻢ ﺑﺪون ﭘﻮدر و ﻓﻘﻂ ﺑﺎ اﺻﻼحﻛﻨﻨﺪه ﻟﺰﺟﺖ ﺑﻮد‪ .‬ﺑﺮاي ﻃﺮح اﺧﺘﻼط از آزﻣـﺎﻳﺶﻫـﺎي ﺣـﺪس و ﺧﻄـﺎ ﻳـﺎ از‬
‫روش اروﭘﺎﻳﻲ اﺳﺘﻔﺎده ﺷﺪ‪ .‬ﺑﺮاي ﻧﻤﻮﻧﻪﻫﺎي ﻣﺮﺟﻊ )ﺑﺘﻦ راﻳﺞ( ﻧﻴﺰ دو ﻧﻮع ﻧﻤﻮﻧﻪ‪ ،‬ﻳﻜﻲ ﺑﺎ ﺳﻴﻤﺎن ﭘﺮﺗﻠﻨﺪ و دﻳﮕـﺮي ﺑـﺎ‬
‫ﺳﻴﻤﺎن ﭘﺮﺗﻠﻨﺪ و ﺧﺎﻛﺴﺘﺮ ﺑﺎدي ﺑﺎ ﻛﺎراﻳﻲ ﻣﺘﻮﺳﻂ )اﺳﻼﻣﭗ= ‪80mm‬ـ‪ (50‬ﺑﺎ اﺳﺘﻔﺎده از روش ‪ [71] DOE‬ﺗﻬﻴﻪ ﺷﺪ‪.‬‬
‫ﺟﺮﻳﺎن اﺳﻼﻣﭗ ﻧﻤﻮﻧﻪﻫﺎي ‪ SCC‬و ﻣﻘﺪار اﺳﻼﻣﭗ ﻧﻤﻮﻧﻪ ﻣﺮﺟﻊ ‪) C60‬ﻛﻪ داراي ﺧﺎﻛﺴﺘﺮ ﺑﺎدي ﺑﻮد( ﺑﺎ ﺗﻨﻈﻴﻢ ﻣﻴﺰان‬
‫ﻣﺼﺮف ﻓﻮقروانﻛﻨﻨﺪه ﺗﻨﻈﻴﻢ ﺷﺪ‪ .‬ﭘﻮدرﺳﻨﮓ آﻫﻚ ﺑﺴﻴﺎر رﻳـﺰ )‪ ٪98<30 µ‬و ‪ (٪20<2 µ‬ﺑـﺎ ﺧﻠـﻮص ﺑـﺴﻴﺎرﺑﺎﻻ‬
‫)‪ (٪99/3 CaCO3‬و ﺧﺎﻛﺴﺘﺮ ﺑﺎدي آﺳﻴﺎ ﺷﺪه ﻣﻄﺎﺑﻖ ﺑﺎ اﺳﺘﺎﻧﺪارد ‪1‬ـ‪ BS3892‬ﺑﻪ ﻋﻨﻮان ﭘﻮدر اﺳﺘﻔﺎده ﺷﺪﻧﺪ‪ .‬از ﻳﻚ‬
‫ﻧﻮع ﻓﻮقروانﻛﻨﻨﺪه ﺗﺠﺎري ﺑﺮاي رﺳﻴﺪن ﺑﻪ ﺟﺮﻳﺎن اﺳﻼﻣﭗ )‪650 (mm‬ـ‪ 600‬ﺑﺮاي ﺑﺘﻦﻫﺎي ﺧﻮدﺗﺮاﻛﻢ ﺑﻬﺮهﮔﻴـﺮي‬
‫ﺷﺪ‪ .‬اﺻﻼحﻛﻨﻨﺪة ﻟﺰﺟﺖ از ﻧﻮع ﺻﻤﻎ وﻻن در آن ﻧﻮع ‪ SCC‬ﻛﻪ ﭘﻮدر ﻧﺪاﺷـﺖ‪ ،‬اﺳـﺘﻔﺎده ﺷـﺪ‪ .‬ﻧﻤﻮﻧـﻪﻫـﺎي ﻣﻜﻌـﺐ‬
‫)‪ 150(mm‬و اﺳﺘﻮاﻧﻪاي ‪ Φ150×300‬ﺗﻬﻴﻪ ﺷﺪ‪ .‬ﻧﻤﻮﻧﻪﻫﺎ ﭘﺲ از ‪ 24‬ﺳـﺎﻋﺖ از ﻗﺎﻟـﺐ ﺑﻴـﺮون آورده ﺷـﺪه و ﻣﻄـﺎﺑﻖ‬
‫اﺳﺘﺎﻧﺪارد ‪ BS‬ﻋﻤﻞآوري ﺷﺪﻧﺪ‪ .‬ﭘﺲ از ‪ 7‬روز‪ ،‬آزﻣﻮﻧﻪﻫﺎي )‪ Φ100(mm‬از ﻧﻤﻮﻧﻪﻫﺎي اﺳﺘﻮاﻧﻪاي ﻣﻐﺰهﮔﻴـﺮي ﺷـﺪه‪،‬‬
‫ﻣﻘﻄﻊﻫﺎي ‪ 15‬ﺗﺎ ‪ 20‬ﻣﻴﻠﻲﻣﺘﺮ از دو ﺳﺮآﻧﻬﺎ ﺟﺪا ﺷﺪ‪ .‬ﻣﻐﺰه ﺑﺎﻗﻲﻣﺎﻧـﺪه ﺑـﺮاي آزﻣـﺎﻳﺶ ﻧﻔـﻮذﭘـﺬﻳﺮي‪ ،‬ﻗﺎﺑﻠﻴـﺖ اﻧﺘـﺸﺎر‬
‫)‪ ،(diffusivity‬ﻧﻔﻮذﭘﺬﻳﺮي اﻛﺴﻴﮋن‪ ،‬ﺟﺬب ﻣﻮﻳﻴﻨﻪ آب و ﻗﺎﺑﻠﻴﺖ اﻧﺘﺸﺎر ﻛﻠﺮﻳﺪ اﺳﺘﻔﺎده ﺷﺪ‪ .‬ﻧﺘﺎﻳﺞ ﻧﻔﻮذﭘﺬﻳﺮي اﻛﺴﻴﮋن‬
‫ﻧﺸﺎن داد ﻛﻪ ﺑﺮاي ﻧﻤﻮﻧﻪﻫﺎي )‪ ،40(MPa‬ﻫﺮ ﺳﻪ ﻧﻤﻮﻧﻪ ﺧﻮدﺗﺮاﻛﻢ ﻧﻔﻮذﭘﺬﻳﺮي ﻛﻤﺘﺮي از ﻣﺨﻠﻮطﻫـﺎي راﻳـﺞ دارﻧـﺪ‪.‬‬
‫ﺑﺨﺼﻮص ﻃﺮح اﺧﺘﻼطﻫﺎي ﺧﻮدﺗﺮاﻛﻢ ﻛﻪ درآﻧﻬﺎ ﭘﻮدرﻫﺎي ﺳﻨﮓآﻫﻚ و ‪ PFA‬اﺳﺘﻔﺎده ﺷﺪ‪ ،‬ﺿﺮاﻳﺐ ﻧﻔﻮذي در ﺣـﺪ‬
‫ﻓﻘﻂ ‪ 30‬ﺗﺎ ‪ 40‬درﺻﺪ ﻃﺮحﻫﺎي ﻣﺮﺟﻊ داﺷﺘﻨﺪ‪ .‬ﻧﻤﻮﻧﻪﻫﺎي ‪ SCC‬ﺑﺎ اﻓﺰودﻧﻲ اﺻـﻼحﻛﻨﻨـﺪه ﻟﺰﺟـﺖ ﻛـﻪ داراي ﭘـﻮدر‬
‫ﻧﺒﻮدﻧﺪ‪ ،‬ﺑﻪ ﻃﻮر ﻗﺎﺑﻞ ﻣﻼﺣﻈﻪاي داراي ﺿﺮﻳﺐ ﻧﻔﻮذﭘﺬﻳﺮي ﺑﺎﻻﺗﺮي ﺑﻮدﻧﺪ‪ .‬ﻧﺘﺎﻳﺞ ﻗﺎﺑﻠﻴﺖ ﺟﺬب آب ﻧـﺸﺎن داد ﻛـﻪ در‬
‫ﻣﻘﺎوﻣﺖ )‪ ،40 (MPa‬ﺟﺬب ﺑﻪ ﻃﻮرﻗﺎﺑﻞ ﻣﻼﺣﻈﻪاي در ﺗﻤﺎم ﻧﻤﻮﻧـﻪﻫـﺎي ‪ SCC‬ﻛﻤﺘـﺮ از ﻧﻤﻮﻧـﻪﻫـﺎي ﻣﺮﺟـﻊ ﺑـﻮد‪.‬‬
‫درﻣﻘﺎوﻣﺖ )‪ ،60 (MPa‬ﻧﻤﻮﻧﻪ ﻣﺮﺟﻊ داراي ﺧﺎﻛﺴﺘﺮ ﺑﺎدي‪ ،‬ﺿﺮﻳﺐ ﺟﺬﺑﻲ ﺗﻘﺮﻳﺒﺎً ﻣﺸﺎﺑﻪ ﺑﺎ ﻧﻤﻮﻧﻪﻫﺎي ‪ SCC‬داﺷﺖ‪.‬‬
‫رﻣﻀﺎﻧﻴﺎﻧﭙﻮر ]‪ [60‬آزﻣﻮن ﻧﻔﻮذﭘﺬﻳﺮي ﻳﻮن ﻛﻠﺮاﻳﺪ ﻃﺒﻖ اﺳﺘﺎﻧﺪارد ‪ ASTM C1202‬را روي ﭼﻨﺪ ﻧﻤﻮﻧﻪ ﺑﺘﻦ ﺧﻮدﺗﺮاﻛﻢ‬


مقدمه
یکی از مشکلات مهم بین در ساختمان‌ها، مقاومت آن در برای آتش است. مقاومت لازم در برای آتش ایجاد گردش مقدرات و این نامه‌های ساختمانی تغییر می‌مودند [72]. در مورد مقاومت بین‌المللی در برای آتش پژوهش‌ها و آزمایش‌های به‌شماری انجام شده است و نتایج آن در دسترس انسان است. همانی هرچه بهتر است. تغییرات خاص مختلف بین‌المللی و سبک در نارنج‌پردازی و در میان بالا را بیش‌تری همیشه متفاوت‌تری و جزئی‌تری را به رنگ‌بندی بین‌المللی معمولی با بین‌المللی متفاوت‌تری و جزئی‌تری را به رنگ‌بندی می‌شود [73-75] Harmathy

پیش‌بینی آزمون‌های آتش، تفاوت‌های قابل توجهی را به رنگ‌بندی بین‌المللی معمولی با بین‌المللی متفاوت‌تری و جزئی‌تری مفاهیم نظیر مقادیر مختلف پودر، مقادیر تراکم و وجود مواد افزودنی، باعث می‌شود. تفاوت‌های آتش در حالی دارای خواص پیچیده‌تری باشد.

نکته‌های جزئی و تغییرات ساختاری سطح‌ها به عنوان عوامل اصلی پیدایش پوست‌های نام‌می‌کند. هرنس[77] جدیدی عامل برای رنگ نام برد و پیش‌بینی ادامهٔ نقش او در تغییرات متفاوت‌تری و جزئی‌تری را به رنگ‌بندی می‌شود. تفاوت‌های بین‌المللی در آتش دارای بین‌المللی، هرنس [78] از نظر رنگ‌بندی در برای آتش به میزان زیادی واابسته به خواص ریز‌سئیک. این آتش با این وجود اغلب پژوهش‌های آتش روز، آزمون‌های مقایسه زیادی متمرکز شده، فقط تعداد انگیز از پژوهش‌ها به دیدگاه تغییر ریزسئیک، مانند تغییرات خلل و فرق، توزیع اندازه‌گیری‌ها و اتصال منفی‌ها
پرداختن، به این دلیل بی (Ye) [78 و 79] تغییرات ریزساختار چند نوع خمیر سیمان، با الیاف پلی پروپیلن و بدون آن، بر اثر افزایش دما بررسی کرده است. آزمون‌های خمیر سیمان بین خودتراکم، خمیر سیمان بین توانمند و خمیر سیمان سنتی با نسبت‌های اختلافی یکسان با بین‌های مربوطه، اما بدون سنگدان، ساخته شده. مخلوط‌های شیرین نور SCC استفاده شده در این مطالعه با سیمان CEM I CEM II/B-S با افزایش دمای دو بخش، نتیجه‌گیری شده است. در این مطالعه با استفاده از تخلخل سنت چیپس و میکروسکوپ الکترونی SEM روشنی (TM) تغییر کیفی خمیر سیمان خودتراکم در دماهای مختلف به وسیله آنتل خودتراکم و سنگدان، قابل شناسایی و در دمایه، انرژی تولید آن، در شکوه‌های مختلف حاصل از دمای هم‌اکنون و پلی کپسند. 

در پروپیلن، با خودتراکم در باوعده‌های بین 190 و 250 درجه سانتی‌گراد، آگهی‌های آفتابی از انجام شده، نشان داده شد که در تغییرات دما و مقدار ثابت‌های سنتی، باعث افزایش یافته در اختلافات بین‌های مربوطه شده است. 

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توانست منفاوت است. [79].
وجود پودرهای پرکننده، مانند پودر سنگ آهن، از موارد مهمی است که باعث تفاوت رفتار بین خودتراکم در برابر آتش با سایر بتن‌ها می‌گردد. [62]، بیان می‌شود که است.
اخلاط‌های خودتراکم باعث می‌شود تا چگالی تراکم به نحو قابل توجهی به‌پایه وقت و در دست‌بازی به‌وژه در خاصیت جریان اولیه و سرعت جابجایی تأثیر به سری‌بازی دارد. به‌عنوان نمونه، محدودیت درصد هرمز درست و ریز نیز، علاوه بر تنظیم قابلیت‌های دیگر از بین مانده، با مفهوم چگالی تراکم قابل تفسیر است. هر چه مقدار مقدار نیز، علاوه بر تنظیم قابلیت‌های دیگر از بین مانده، با مفهوم چگالی تراکم قابل تفسیر است. هر چه مقدار

مشکلات‌های ریز افزایش یافته مقدار آب افزایش یافته به‌وژه درست و ریز نیز، علاوه بر تنظیم قابلیت‌های دیگر از بین مانده، با مفهوم چگالی تراکم قابل تفسیر است. هر چه مقدار

- 3

جهت زیادی از پودرهای معدنی در تیپ‌های خودتراکم استفاده می‌شود که اثرهای متعددی روی خواص دارد. هنگامی که پودرهای خلیف مانند پودر سنگ آهن باعث افزایش حرارت هیدراسيون می‌شود، مقابله پژوهش‌های افراد مختلف نشان می‌دهد که تدریجی تغییر درون و پودرهای به‌وژه درست و ریز نیز، علاوه بر تنظیم قابلیت‌های دیگر از بین مانده، با مفهوم چگالی تراکم قابل تفسیر است. هر چه مقدار

- 4

به علت جدید پودر بنی خودتراکم، خودضریب وسیعی از پژوهش‌ها بر روی آن نیز است. نیاز به حجم بالای پودر، حوزه‌های وسیعی از پژوهش‌ها بر روی آن نیز است. نیاز به حجم بالای پودر، حوزه‌های وسیعی از پژوهش‌ها بر روی آن

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خلاصه

پتنهای معمولی که با روش‌های صنعتی طراحی، مخلوط، عمل و در نهایت در حجم و ترکیب و پذیرش، پتنهای نسل جدید دستیابی به مشخصات مورد نظر برای کاربردهای ویژه و با برای دستیابی به ارایه کاربرد. این پتنهای توامنده (HPC)، برای رفع این مشکلات رو به گسترش می‌باشد. برای ساخت این نوع پتنهای ویژه معمولاً از مصالح رایج ولی با نسبت آب به مواد سیمانی کم و مقدار سیان زیاد استفاده می‌کنند.

مواد افزودنی شیمیایی و معدنی (جایگزین سیمان) بیانی سبب کاهش عملیات سیمان شود. اما از نجات که روش طرح مخلوط خاصی برای این نوع بینها وجود ندارد و به دلیل کم بودن نسبت آب به مواد سیمانی، ارزش تولیدی و بهبود نسبت آب و بودن و سیمان در این نوع بینها از مواد سیمانی نسبتاً زیاد استفاده می‌کند که این امر از این نکته به گرزش برگرداند، باید افزایش جمع‌شدنی و احتمال کاهش دوم نیز می‌شود.

روش بیشتر شده‌ای در این مقاله که بخشی از تأثیر تحقیق انجام شده روی نوع طرح مخلوط بین می‌باشد، بر اساس فلسفه دستیابی به حاکمیت تراکم می‌باشد. در این روش، نحوه توزیع دانه‌ها یک نوع انتخاب برای طرح مخلوط بینهای توانمند خواهد بود. این امر باعث کاهش دیده می‌شود که این دارای نفوذ پذیری به کم بوده و در ضمن میزان سیمان زیاد آن (۴۰ درصد) کاهش یابد. کاربرد این روش می‌تواند باعث کاهش بینهای توانمند شود، نهایتاً سبب افزایش مقاومت و دوم نسبت به بینهای HPC مشابه می‌شود. بنابراین می‌گردد که سبب کاهش مصرف سیمان نیز می‌گردد.

کلیدویژه: بینهای توامنده، مواد جایگزین سیمان، دوم، حاکمیت تراکم، نفوذپذیری، منحنی توزیع دانه‌ها

۱- مقدمه

پتنهای توانمند (High Performance Concrete) بر اساس مشخصات مواد و مصالح موجود و برای دستیابی به یکسرو خواص مورد نظر، روابطی با شرایط محیطی، افزایش دوام و عصر مقیاس و در نهایت قابل توجهی بودن از نظر اقتصادی، طراحی می‌شود (۱). امروزه بینهای توانمند کاربردهای متنوعی از جمله استفاده در ساختمان‌هایی که در شرایط انرژی‌های مهار می‌باشد، ارائه‌رسیده، نیاز به تولید، قطعات بسته، پل و غیره، داشته و کاربرد آن در نیز رو به افزایش می‌باشد (۲۰). بطور معمول نگاه پتنهای توامنده به پتنهای معمولی، در کاربرد مواد افزودنی شیمیایی و معدنی (جایگزین سیمان) و
کاوش نسبت آب به سیمان می‌باشد. استفاده از این مواد امکان کاهش نسبت آب به سیمان و مقدار سیمان را بوجود آورده و در حین حال سبب افزایش دومین نیز می‌شود. با توجه به اینکه عملکرد سیمان مواد آرزان قیمت هستند (با مجموعه جانی کارخانه پاپاشین و یا بصورت طبیعی وجود دارند)، استفاده از مواد یکی از مواد یا دوام آفودنسی شیمیایی سبب افزایش شدید نیز می‌گردد.

یکی از مواد یا دوام آفودنسی سیمان که در اکثر بتن‌های نوامد مورد استفاده قرار می‌گیرد، کاهنر بادی بینی‌ها و گرد کنی نسبت به چهار بتن گیرد. در بتن‌های نوامد به خصوص مشخصات کاهج سبب می‌شود، ولی به علت داشتن شکل کروی دانه‌ها و افزایش دوام بر این همچنین افزایش بودن در بتن‌های نوامد با کاهش خاکستر بنا در محدوده 6 دور سیالی از این مواد کاهنر سیمان می‌باشد. بهترین خاکستر بنا در صورتی که به شکل دانه‌ای دور سیالی سبب می‌شود که از دیدگاه درمانی به روانی مانند، افزایش گرفته (1) دستسپارس و انتخاب روز افزایشی ناشی از یک چگالی (بیان‌سپارک) و ناشی از خشک شدن در هنگام استفاده از دور سیالی باید مورد توجه قرار گیرد.

مطابق توجه است که انتخاب نوع ماده یا دوام سیمان، تأثیر درصد اختلال مواد سیمانی و همچنین دیگر اجزای بتن می‌باشد. استفاده از روش‌های جدیدی ممکن است بتواند با استفاده از سیستم‌های آزمایشی تعطیلی دارد و به عنوان محدودیت‌های که در این مورد توانایی تولید نیاز باشد [13]. عدم تغییر مشکل این روش‌ها عدم امکان درستی به پیشنهاد ضریب تأثیر مواد افزودنی می‌باشد، این امر مستند به نظر ریسین، روندها آوندهای سیمان، ماده‌زیستی، نسیب آب به وسیله ریسین و تراکم ذیلی مورد نظر، می‌باشد (8، 9)، یکی از کارهای انجام شده، پیشنهاد دارد که نظر نور در تأثیر مواد یا دوام سیمانی، افتکر خیلی نشان داد. برای اصلاح نتیجه ناخوشایند می‌باشد. از ارائه داده، تراکم ذیلی یا دوام سیمانی در حیطه مقدار ماده یا دوام سیمانی (C+K) در مقادیر ماده یا دوام سیمانی (C+K) می‌باشد. این نتیجه بعدها به نوع ماده یا دوام سیمانی افزودنی استفاده کرده، که به وسیله ضریب تأثیر مواد افزودنی در حیطه مقدار ذیلی (C+K) می‌باشد. این نتیجه نتیجه ناخوشایند می‌باشد. این نتیجه NQNC=1-1 до استفاده از fa0.2 تأثیر K مورد قرار داشته است. در این حیطه، 6 در نظر گرفته شد.

عموم درگیری که منجر می‌شود تغییر قرار گرفته ولی اهمیت بسیار مهم در آشفته‌های افزایشی تراکم ذیلی و دوم می‌باشد. نحوه توسعه دانه‌ها (اعم از سیمان‌هاید و دیگر مصالح جاده‌سازی) می‌باشد. استفاده از منحنی‌های ایده‌آل می‌تواند در افزایش تراکم، کاهش تداخل و همچنین کارایی بین سیستم متریک باشد. در ادامه آشفته تغییر توسعه دانه‌ها و قطعه مورد استفاده به تخصیص ارائه شده است.

- مبانی و اصول طرح اختلال

چنان‌چه اشاره شد، اگر پتانسیل در افزایش تراکم سیمانها، فضای بین دانه‌ها را به حداکثر ممکن کاهش داد، نفوذپذیری کاهش ییدا کرده و در نتیجه دوم بتن دانه خواهد یافت. در این راستا با توجه به تحقیقات هیله (14) به هدف کاهش نفوذپذیری که بر اساس نظرهای Fuller & Thompson مصالح سیمانی انجام شده‌است، توزیع دانه‌ها به کمک آمی کردن که ضمن حفظ کارایی ملات و یا بتن، دوم آن در مقابل عوامل شیمیایی مساح افزایش یابد.
جهت کاهش ضخامت ناحیه انتقال که بیشتر در اطراف سنگدانه‌ها درشت دیده می‌شود می‌توان نسبت ریزدانه‌ها را نیز افزایش داد. همچنین با منظور کاهش تغذیه‌بری در خمیر سیمان، باید ضمن کاهش تخلخل، توزیع خرید به نحوی اصلاحات در کارآیی به دوام، در این تغییر و توزیع مهافی افزایش یابد. با توجه به اینکه مانند سیمان به نحوی استحکام نسبت به سیاورد سیرز (میکروپی) نیا باید، انتفاخ‌های یکی از قطعی کوکچک در بخش‌های دهانه‌ای سیمان جهت بالا بردن جرم کوچک خشک، لازم بودن نور می‌رسد چرا که اولاً دهانه ریز از یک یکدیگی طراحه و تخلخل را کاهش می‌دهد. نتایج توزیع خریدات مناسب‌تر شده و نتایج در صورتی که، ریزای سیلیس و افزایش را باشد می‌تواند سرعت هیدرولیسیون افزایش داده و قسمتی از هیدروکسید کلسیم آزاد را به صورت برسانند.

منحنی‌های دانه‌بندی مطلوب، عموماً بر اساس آزمایش‌های تجربی و محاسباتی تئوری، ارائه گردیده‌اند. از این Fuller Rissel و Graf Fuller & Thompson در اشاره کرد که در این بین، منحنی جمله می‌توان به منحنی‌های Fuller & Bolomey و Rissel را نسبت داده است [15].

از طرف دیگر، برای دستیابی به میان‌بینی نفوذ‌پذیری کم و دوام زیاد، باید با کاهش خاله و فرک و افزایش درک مرحله‌ای درک که به دوام و ریز دیده‌ای با بینی‌بندی‌ای وجود دارد، این موقعیت می‌تواند برای بررسی در این تحقیق برای دستیابی به بینی یک حداکثر تراکم با استفاده از حداکثر میزان مواد چسبانده می‌باشد. در این راستا با استفاده از روابط ریاضی، توسعه نشانه‌ها (اعم از مصالح سنگی و مصالح چسبانده) یک نوعی از تکنیکی که مصلح سنگی ریزدانه و مصالح سنگی می‌باشد. در این بخش‌ها، می‌توانند سنگی ریزدانه در شرایط زیر به تراکم مخلوط گردند. شایع‌ترین استفاده‌ها در عمل منحنی دانه‌بندی معنی‌دار استاندارد قسمت برداشت دانه‌های کوکچک از 150/0.5/0 روی بر نمی‌گیرد در صورتی که برای دستیابی به یک دانه‌بندی بهبود و تراکم حاصل اهمیت می‌باشد.

استفاده از منحنی دانه‌بندی Fuller & Thompson Fuller & Thompson اگرچه دارای مزایایی است هر ساده در فضای می‌باشد ویلی و(F&T) F&T منحنی که تست شد. این مورد گذشته بودن کامل تمام دانه‌ها، کاهش کارایی و استفاده از یک منحنی F&T F&T تقیدی کاهش با استفاده از (توسط هله ما) [14]) منحنی F&T F&T است. منحنی اصلاح شده آن در دو حال (مقیاس خاص و اُتاترونی) نشان داده شده‌اند. همان‌گونه که مشاهده می‌شود در خشک درسته‌اند و میان‌بندی (بزرگتر از 2 میلی‌متر) منحنی اصلاح شده دارای مقایسه بیشتری، نسبت به منحنی F&T F&T می‌باشد و بالعکس برای دانه‌های کوکچک از 2 میلی‌متر دارای مقدار ریزدانه کمتری نسبت به منحنی F&T F&T می‌باشد [19].

![شکل 1] منحنی‌های دانه‌بندی F&T و اصلاح شده در مقیاس خطی
شکل 2- منحنی‌های دانبندی F&T و اصلاح شده، در مقایسه تکاری‌پذیری

مزایای دانبندی F&T و اصلاح شده آن، به شرح زیر می‌باشد [14 از 19):

1. این نوع دانبندی با مقاومت فشاری برابر در مقایسه با دانبندی بیوسته نیاز به سیمان کمتری دارد.
2. در حقیقت با کاهش نسبی قطر سیم دانبند رقیق اول و دانبند بزرگ از خود نشان می‌دهد.
3. قابلیت تراکم زیر نمایه در اثر کاهش درصد میان دانبند، به‌طور محسوس افزایش می‌یابد. زیرا دانبند‌ها با درصد حجمی بیشتر در دشت‌های جایافشود، بین تریک و بستگانه، تراکم‌پذیری بیشتری حاصل می‌شود و هوای داخل فضای بین سیم‌های دانبند تخلیه می‌گردد.
4. قرارگیری سیم‌های دانبند در این نوع مخلوط بین قبلی نگهداری آب را افزایش می‌دهد.
5. موارد 2 و 3 باعث عمل آوری پهن‌ترین می‌گردد و درصد آب مورد نیاز نیز متفاوت‌کننده می‌یابد.

از نظر کاهش هزینه‌های کارگاهی نیز کاهش جیهان دانبند، مناسب بوده ویژاپسندگان حمل، انتقال، کن‌نمودن و عمل آوری می‌تواند میابد.

یکی از مهم‌ترین ویژگی این استفاده است این منحنی، تعیین نسبت جیهان که کلیه دان‌های موجود در پن، ام از سیم‌های و مواد سیم‌اندازه می‌باشد. بر اساس این منحنی می‌توان سهم هر یک از مواد جامع تشکیل دهنده پن را تعیین نمود، به‌عنوان مثال درصد مشخص شده برای دان‌های کوچکتر از 3/500/0 میلیمتر، بیانگر مقدار مواد سیم‌اندازه خواهد بود (به‌عنوان مواد رقیق‌کننده موجود در سیم‌اندازه‌ها).

3 مطالعات آزمایشگاهی

3-1 مصالح مصری

3-1-1 مصالح سیمانی

در انتخاب مواد سیمانی (مواد سیمانی و موشه‌های دارند) هوشمند از نظر قرار گرفتن. اول از دیدگاه توزیع دان‌های در منحنی دانبندی [14 از 19] استفاده می‌شود که در آن ریخت تغییر مصالح از این سیمانی به مواد سیمانی یکسانی به دانبندی و با انتخاب شونده که توزیع دان‌های تا حد ممکن توزیع در منحنی ایده‌آل باشد. از جهت دیگر باید مواد سیمانی به دان‌های انتخاب و با یکدیگر مخلوط گردد که ترکیب شیمیایی آنها خواص مکانیکی و فیزیکی و همچنین دوام خصوصی سیمان را برآورده نماید. در این راستا کاهش مقدار آب به حداقل مقدار ممکن (کاهش نسبت آب به مواد سیمانی) نیز یکی از پارامترهای مؤثر باشد. 

شکل 2- منحنی‌های دانبندی F&T و اصلاح شده، در مقایسه تکاری‌پذیری

شکل 2- منحنی‌های دانبندی F&T و اصلاح شده، در مقایسه تکاری‌پذیری

شکل 2- منحنی‌های دانبندی F&T و اصلاح شده، در مقایسه تکاری‌پذیری

شکل 2- منحنی‌های دانبندی F&T و اصلاح شده، در مقایسه تکاری‌پذیری

شکل 2- منحنی‌های دانبندی F&T و اصلاح شده، در مقایسه تکاری‌پذیری

شکل 2- منحنی‌های دانبندی F&T و اصلاح شده، در مقایسه تکاری‌پذیری
سیمان: سیمان مصرفی از نوع ۲ و محصول کارخانه سیمان تهران بوده و مشخصات شیمیایی آن در جدول ۱ آرائه شده است. روش انجام آزمون استاندارد ملی ایران به شماره ۳۸۹ می‌باشد.

خاکستر بادی: با توجه به اینکه در حال حاضر در داخل کشور خاکستر بادی تولید نمی‌شود، خاکستر بادی مورد استفاده در این پروژه از کشور آلمان تهیه گردید. این خاکستر بادی از نوع سیبا ریز (Ultra fine) با نوع بوده و دانه بندي (particle size analyser) انجام شده در نمونه شکل ۳ آرائه شده است. به منظور تعیین خواص مکانیکی، فیزیکی و شیمیایی خاکستر بادی از روش استاندارد EN 196-21 و نیز ASTM C 311 استفاده گردید.

دوده سیلسیس: دوده سیلسیس مصرفی در این پروژه تولید داخل کشور و از کارخانه ارنا تهیه شده است. آزمون های مربوط به تعیین خواص شیمیایی دوده سیلسیس مورد استفاده که طبق استاندارد ASTM C 1240 انجام گرفته است در جدول ۱ آرائه شده است.

در طرح مخلوط بیشتر از، از آن سیمان، دوده سیلسیس، خاکستر بادی و ویلر استفاده شده است. با توجه به توزیع ذرات سیمان، خاکستر بادی و دوده سیلسیس مورد مصرف در این پروژه (حدود ۸۰ درصد ذرات سیمان، قطعی به ۱۰۰ میکرون، ۵۰ درصد ذرات خاکستر بادی بین ۲ تا ۵ میکرون و ۵ درصد ذرات دوده سیلسیس درای قطعی کمتر از ۱ میکرون، مطابق شکل ۳ پوشارا) و همچنین به دلیل خواص شیمیایی هر یک از این مواد و همچنین خواص مکانیکی، فیزیکی و دوام بتن درصد هر یک از این مواد بشر حذل انتخاب گردید:

سیمان: ۷۰ درصد وزنی
خاکستر بادی بسیار ریز: ۲۰ درصد وزنی
دوده سیلسیس: ۱۰ درصد وزنی

جدول ۱: تجزیه شیمیایی سیمان و میکروسیلس

<table>
<thead>
<tr>
<th>ترکیب شیمیایی (%)</th>
<th>سیمان</th>
<th>میکروسیلس</th>
<th>ترکیب شیمیایی (%)</th>
<th>سیمان</th>
</tr>
</thead>
<tbody>
<tr>
<td>SiO₂</td>
<td>5/7</td>
<td>۲۴/۷۴</td>
<td>MgO</td>
<td>۹/۷۱</td>
</tr>
<tr>
<td>CaO</td>
<td>۱/۷</td>
<td>۲۳/۳۱</td>
<td>Al₂O₃</td>
<td>۲/۶۸</td>
</tr>
<tr>
<td>SO₃</td>
<td>۱/۸</td>
<td>۱/۷۹</td>
<td>Fe₂O₃</td>
<td>۴/۶</td>
</tr>
<tr>
<td>Na₂O + 0.658 K₂O</td>
<td>۱/۳۷</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
سیمیان، ذرات توزیع داغی، ذرات پودر و سیلیس و برکترو میکروپورشه

شکل 3- منحنی‌های توزیع قرارت سیمیان، باکستریا، دوده‌سیلیس و پودر کوارتز میکروپورشه

جدول 2: مشخصات ویژه و شیمیایی مصالح سنگی

<table>
<thead>
<tr>
<th>ویژگی</th>
<th>وزن خشک (g/cm³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>شیشه</td>
<td>2/07</td>
</tr>
<tr>
<td>فیلر</td>
<td>0/17</td>
</tr>
</tbody>
</table>

3-1-3- ماده افزودنی شیمیایی

قروی روان کننده سحری بر پایه کرواسیاتِ‌ها و با نام تجاری structuro به‌دست آمده است، این نوع فوق فوق روان کننده از سلس جدید مواد افزودنی می‌باشد. از محتمل مزایای این نوع فوق روان کننده، افزایش پسیار زیاد روی‌های بدن جداسازی قرار نگرفته بلکه در مدت زمان طولانی‌تر نیز از گیره‌ها، این موارد می‌باشد. مشخصات این فوق روان کننده در جدول 3 ارائه شدند.

جدول 3: مشخصات ویژه و شیمیایی فوق روان کننده

<table>
<thead>
<tr>
<th>ویژگی</th>
<th>وضعیت ظاهری</th>
</tr>
</thead>
<tbody>
<tr>
<td>pH</td>
<td>5/5</td>
</tr>
<tr>
<td>وزن جسم</td>
<td>5/7 Kg/litre</td>
</tr>
<tr>
<td>کمتر از 0/1 درصد</td>
<td>میزان کثر مورد نیاز</td>
</tr>
</tbody>
</table>

3-2- تعیین نسبت اختلاف سنگانه‌ها (ریز، درشت و فیلر)

همان‌گونه که شش شاخص، یکی از نقاطی اصلی این نوع توانمند با نوآوری راپید آن استفاده از روش دستی‌باین به حداکثر تراکم (Dense Packing)، با اصلاح نحوه توزیع ذرات دانه‌ای (اعضای از مصالح سنگی و مصالح چسبانده) می‌باشد. که به‌این معنی از منحنی‌های توزیع شده قروی و ناپسند استفاده کرده، همچنین جهت امکان از بهبود
عملاکرد این نوع بتن‌ها مانند رابط، طرح‌هایی نیز بر اساس روش سنتی بهدرسانی شد. برای طرح‌های توزیعی، دو طرح با نام‌های SPD و FSPD و دو طرح سنتی نیز دو طرح با نام‌های SN و FSN شناخته شدند. نسبت اختلاف سنگکانهای دو طرح اول مطابق با مخاطب شده فکر و تامسون و برای دو طرح سنتی نیز از منجنی ترکیبی استفاده شد. استفاده شده‌است، منجنی دانه‌بندی طرح‌ها در شکل 4 نشان داده شد.

شکل 4- منحنی دانه‌بندی مجموع سنگکانه‌ها (ریز و درشت)

### 3-3 تعبیه نسبت آب به سیمان

با توجه به اینکه در بتن‌های توانمند نسبت آب به ماد سیمانی بسیار پایین می‌باشد، تا دوباره بتن در برای عوامل مهادی خصوصاً کریستال سیلت و ماد سیمانی افزایش یابد، و همچنین طبق آینه‌نامه بتن ایران (آیآ) حداقل نسبت آب به سیمان (با ماد سیمانی) برای شرایطی که احتمال تهاباج کریستالی و خطرگوی میلگرد وجود دارد، 400 و حداقل سیمان350 کیلوگرم در 30 دقیقه که احتمال تهاباج سیلتی (فوق العاده شدید) می‌باشد، برای شالوده‌های بتنی جدید، حداقل نسبت آب به سیمان 4/0 و حداقل سیلزیم 400 کیلوگرم در 30 دقیقه که احتمال تهاباج سیلتی (فوق العاده شدید) می‌باشد. همچنین این توصیه شده است، در این تحت‌طابع نیز سعی شده است نسبت آب به ماد سیمانی متوسط به 3/0 محدود شود. البته نسبت آب به سیمان ارائه شده در آب، نسبت آب به ماد سیمانی موثر نمی‌باشد.

در هر مواردی است که از ماد فاکتری معنی‌دار (دود سیلیک سیتیک در کارتر بادی) استفاده می‌شود، برای اصلاح تاثیر این مؤاد از فاکتور تأثیر استفاده می‌شود. در این تحت‌طابع مقدار فاکتور اصلاح (k) طبق استاندارد EN 206-1:2000 انتخاب گردید. بر این اساس، برای کارتر بادی (که کفین استاندارد مقیاس 450 در صورت ترکیب شدن با سیمایی به دست آمده است) استفاده‌ای از فاکتور خاکسیا به مقدار 346/5MPa برای 0/7 و برای سیمایی به مقدار 36/5MPa برای 0/7 و بیشتر، این ضریب با 0/2 و به مقدار 0/3 می‌باشد. همچنین برای دود سیلیسیا با 0/7 توصیه شده است. روشهای اعمال این ضریب در تصحیح نسبت آب به سیمان با کمک رابطه زیر انجام می‌شود:

\[
\text{water} = \frac{\text{cement}}{(K \times (\text{silica fume or fly ash})}
\]
3

**4-3 تعيين نسبت اختلاط**

مخلوط‌ها بر اساس دو نوع طراحی شده‌اند. سری اول شامل مخلوط‌هایی می‌باشد که حاوی دوده‌سیلیس و خاکستر بوده و سه نوع روش‌های طرح مخلوط را بیان می‌کند. طراحی دوم شامل سه نوع خاکستر و باعث دمای و ترکیب ساختاری سنتزه‌ها شده به‌شکل مخلوط یکی اسید بالی به مقدار ۱۲، ۱۷، و ۱۳ كیلوگرم به متر مکعب شده‌اند.

*آب مهیج، حاوی طرح و بنن*.

بنیادی طرح دوده‌سیلیس، دوده و روش‌های سنتزه‌ها در جدول ۱ نشان داده شده است. طرح مخلوط یکسان به شناسه FSPD و طرح FSDP بازه‌ای از اسیدیت به شناسه SN و اکسیدیت به سه نوع FSPD و SFDP و اکسیدیت به سه نوع FSPD و SFDP مشخص می‌شود.

<table>
<thead>
<tr>
<th>طرح</th>
<th>سیمان</th>
<th>دوده طرح</th>
<th>در ورود سیلیس</th>
<th>در سان</th>
<th>در دوده طرح</th>
<th>در ورود سیلیس</th>
<th>در سان</th>
<th>در دوده طرح</th>
</tr>
</thead>
<tbody>
<tr>
<td>اسس</td>
<td>۱۲۰</td>
<td>۲۱۳</td>
<td>۱۳۵</td>
<td>۱۸۰</td>
<td>۱۹۵</td>
<td>۲۰۰</td>
<td>۲۰۵</td>
<td>۲۱۰</td>
</tr>
<tr>
<td>اکسید</td>
<td>۱۲۰</td>
<td>۲۱۳</td>
<td>۱۳۵</td>
<td>۱۸۰</td>
<td>۱۹۵</td>
<td>۲۰۰</td>
<td>۲۰۵</td>
<td>۲۱۰</td>
</tr>
<tr>
<td>اکسیون</td>
<td>۱۲۰</td>
<td>۲۱۳</td>
<td>۱۳۵</td>
<td>۱۸۰</td>
<td>۱۹۵</td>
<td>۲۰۰</td>
<td>۲۰۵</td>
<td>۲۱۰</td>
</tr>
</tbody>
</table>

7-۴-۳- اختلاط بتن

پس از اعمال تصحیح مقدار رطوبت موجود در سنتزه‌ها، هر یک از اجزاء به تفکیک توزین شده و سپس
ابتدا سنگدانه‌ها و بعد مصالح سیمانی؛ آب و در انتهای ماده افزودنی فوق روان کننده به داخل مخلوط کن ریخته شده جهت انجام اختلاط از یک مخلوط کن تآری (Pan mixer) استفاده گردید. ظرفیت اسی مخلوط کن ۲۵۰ لیتر و چرخش نیز بهانهٔ یک بسته عمده و در جهت عکس حرکت دیگ بوده است.

۳-۴-۳‌ ساخت و عمل‌وری آزمون‌ها

پس از انتخاب اختلاف بین، آمونه‌ها بین ساخته، نگهداری و عملیات آزمون‌ها بر اساس استاندارد EN 2000:2-2390، بین در قالب‌های مورد نظر جای‌گذاری و متراکم گردیدند. آزمون‌ها پس از یک‌کروز نگهداری در شرایط استاندارد (اطلاق مروج) از قالب خارج و به داخل حوضه عمل اوری با دمای ۲۳٪ متفق و تا زمان انجام آزمایش در شرایط مذکور عمل اوری شدند.

۳-۵- آزمایش‌های بتن تازه

جهت تعیین خواص بتن تازه (ساخت نشده) آزمون‌های تعيين وزن مخصوص، درصد هوا موجود و روانی به روش اسلامی، و تعیین دمای بتن انجام گردید که نتایج آن در جدول ۵ آسان شد.

جدول ۵: نتایج آزمایش‌های بتن تازه

<table>
<thead>
<tr>
<th>وزن مخصوص (Kg/m³)</th>
<th>هوا محسوس (Cm)</th>
<th>شناسه مخلوط</th>
</tr>
</thead>
<tbody>
<tr>
<td>۲/۸</td>
<td>۲۳۹۱</td>
<td>۲۲</td>
</tr>
<tr>
<td>۱/۶</td>
<td>۲۲۹۶</td>
<td>۲۱</td>
</tr>
<tr>
<td>۱/۴</td>
<td>۲۰۵۵</td>
<td>۲۰</td>
</tr>
<tr>
<td>۱/۵</td>
<td>۱۹۳۳</td>
<td>۲۱</td>
</tr>
</tbody>
</table>

۳-۶-۱- مقاومت فشاری

به منظور تعیین مقاومت فشاری و بررسی روند افزایش آن، آزمون‌های مکملی با ابعاد ۱۰۰‌میلی‌متر تپه گردیدند. آزمون‌ها (در هر سن، سه آزمون) در سین ۷ و ۲۸ و ۹۰ روز مورد آزمون قرار گرفتند. نتایج آن‌ها به مقاومت فشاری آزمون‌ها بر اساس استاندارد EN 2000:3-۲۳۹۰ و نتایج آن در شکل ۵ آسان شد.

شکل ۵ - نتایج آزمایش تعیین مقاومت فشاری
3-6-4- تغییرات طول منشورهای بتنی در معرض محلول سولفات

به منظور بررسی عملکرد محلول‌های بتنی در معرض سولفات، از محلول‌های سدیم 5 میلی‌متری درصد استفاده گردید. بدین منظور منشورهای بتنی با بعدهای 28 × 28 × 28 میلی‌متر همگی در دو گروه آماده شدند. گروه‌های پایین‌تر از pH ناپایداری کار گذاشتند. شدید بود، ساختمان و پس از 28 روز عمل آوری در محلول‌های قرار داده شدند. جهت تنظیم pH نسبت نکه
4- نتیجه گیری
همانگونه که اشاره گردید، در این تحقیق طرح مخلوط بین توانمند نسل جدید که با تغییر در دانه بندی ستگدانه، استفاده از خاکستر بادی و همچنین یک نمونه پودر کوارتز، ارائه شده است. این نوع بین توانمند بهعلت تراکم زیاد و حداکثر تخلخل، نفوذپذیری بسیار کم و مقاوم در برای شرایط حیاتی (اسید سولفوریک، سولفات سدیم، کلرید و...) دارد. ضمن آن که کیفیت سیمان نیز در آن کاهش و کارایی مخلوط افزایش یافته است.

بدين منظر آزمایش یافته، نمونه‌های سطح نشان دهنده از افزایش عمق نفوذ آب تحت فشار، نفوذپذیری بیشتر در آنجا که کلرید به روش تستی شده، انساب ناشی از تهاتر سولفات، بعنوان برنامه آزمایشگاهی مد نظر قرار گرفت.

نتایج آزمایش مقاومت فشاری نشان می‌دهد، نمونه‌های مخلوط SPD که در آن از منحنی دانه بندی اصلاح شده استفاده شده و بدون خاکستر بادی می‌باشد. دارای پیشترین مقاومت در سنین مختلف F&T سبینان (استفاده شده است) اگر چه در سنین 7 و 28 روز دارای مقاومت‌های به سیستم متناسب با SPD بوده ولی در سن 90 روز با آهنگ افزایش مقاومت 90 به 90/50 N/mm² به 90/2 درصد، در سن 90 روز دارای مقاومت 95 N/mm² به 90/50 N/mm² این کدی آهنگ افزایش مقاومت در سنین اولیه در طرح‌های اولیه، در طرح‌های FSN و FSD نیز که دارای خاکستر بادی می‌باشد، مشاهده می‌باشد. در مقابل طرح‌های اولیه، که به عنوان بین‌های توانمند رایج بررسی قرار گرفته‌اند، ارائه دارای مقاومت SN و FSN به وقوع پیش‌بینی نسبت به مقاومت بین توانمند برای همه دارای افزایش مقاومتی 90 به 50 N/mm² از 90 به 57/50 N/mm² 40 درصد می‌باشد.
در خصوصی پارامترهای دوم که در این تحقیق مورد بررسی قرار گرفت نیز طرح مخلوط برشنهای دارای شاخص‌های بسیار پهپدی نسبت به تنها نمونه را پاسید. اگرچه هم مخلوط‌ها را می‌توان جز بتن‌های با دوام لقی نمود.

3- طرح SPD که بر اساس فلسفه حاکم تراکم طراحی شده، وی در آن از خاکستر بادی استفاده نشده، از لحاظ پارامترهای دوم کم‌پایین‌تر از طرح FSPD می‌باشد. شایان ذکر است طرح FSPD می‌باشد.

فشاری بیشتر می‌باشد.

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ENVIRONMENTAL impact assessment of concrete industry in Iran

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ABSTRACT

Constructional industry as the largest consumer of natural materials produces the most portion of the wastes in the country. Concrete being the most commonly used building material in Iran that can have pronounced effect on the production of the waste. Concrete is composed of sand, gravel, crushed rock or other aggregates held together by a hardened paste of hydraulic cement and water. Therefore, evaluation of effects of concrete industry on environment is one of the important cases to be considered. In this paper environmental impact assessment (EIA) of concrete industry is performed, and appropriate recommendations to minimize the effect of environmental impact are given.

Keywords: concrete, environmental impact assessment, sustainable development, IRAN

1. INTRODUCTION

Sustainable development involves meeting present needs without compromising the ability of future generations to meet their needs [1].

The ecological criteria for sustainable development are the preservation of biodiversity and adoption of human activities to the natural resources and tolerance of nature [2]. For this purpose there is increasing concern now that the choice of construction materials must also be governed by ecological considerations.

At the beginning of the 20th Century, the world population was 1.5 billion; by the end of that Century it had risen to 6 billion. Considering that it took 10,000 years after the last ice age for the population to rise to the 1.5 billion mark, the rate of growth from 1.5 to 6 billion people is remarkable [1]. Unfortunately, our choices for technology have turned out to be wasteful, because decisions are based on short term and narrow goals of enterprise rather than a holistic view of the full range of consequences from the use of a technology. Only 6% of the total global production of materials, some 500 billion tons a year, actually ends up in consumer products, whereas much of the virgin materials are being returned to the environment in the form of harmful solids, liquids, and gaseous wastes. The greatest environmental challenge today is that of the human-made climate change due to global warming caused by steadily rising concentration of green-house gases in the earth's atmosphere during the past 100 years [3].

An (EIA) is an assessment of the possible impact (positive or negative) that a
project may have on the natural environment. The environmental impact of building products consists of procurement of raw materials, the manufacturing process and also the use of energy resources during transportation – all of which to some extent burden the environment. Environmental burdens of the cement industry consist of limestone quarrying, burning and grinding of clinker. Extraction, excavation, manufacturing and transportation of aggregates and distribution of the final products are elements for EIA of concrete industry [2].

After water, concrete is the second most widely consumed substance on earth. Using concrete minimizes the depletion of our natural resources. Its ingredients come directly from readily available materials: water, aggregate (sand and gravel or crushed stone), and cement. Cement is composed of 75% limestone, the most common mineral on earth. Although extracting any raw material from the earth takes a toll on the environment, extracting the raw materials for concrete has a lower impact than that of other construction materials. Because the ingredients for concrete are so plentiful, supplies are virtually inexhaustible.

The goal of this paper is to identify the environmental impacts of concrete and its products which in-turn can lead to determining options for improving environmental effects.

2. EIA OF CONCRETE INDUSTRY IN IRAN
I.R. Iran is located in the center of the Middle East and bridges the Caspian Sea to the Persian Gulf and the east of Asia to the west of Asia. Because of its strategic location it is one of the important countries in the region. Iran has complex climate ranging from subtropical to subpolar and that it is possible at the same time to witness the climatic conditions of all four seasons in the different parts of its territory. The building industry is one of the most important industries in Iran, concrete being the most widely used material. Therefore the concrete industry is very important from the point of view of EIA.

2.1. Concrete Components
Ordinary, concrete typically contains about 12 percent cement, 8 percent mixing water, and 80 percent aggregate by mass. The 11.5 billion tones-a-year concrete industry is thus the largest user of natural resources in the world. The demand for concrete is expected to grow to approximately 18 billion tons (16 billion tones) a year by 2050 [4]. This means that, in addition to 50 million tones of cement (current annual production of cement in Iran), the concrete industry in Iran is consuming annually 137 million tones of fine and coarse aggregate together with 15 billion liters of mixing water.

2.1.1. Cement
The examination of concrete manufacturing shows that the cement, which usually makes up 10–15% of concrete, is the main environmental polluter. Cement manufacturing covers material and energy flows during the extraction of materials, and the production processing such as raw meal, clinker burning, grinding and transportation of the product. Because of the high temperatures used during cement
production and the decomposition of calcium carbonate, the cement accounts for over 60% of energy used in concrete manufacturing. The amount of cement production and consumption per capita in Iran is shown in figure 1. Estimated world cement production and CO₂ emission in cement manufactures are shown in figure 2 and 3 respectively [2].

![Graph 1](image1.png)

**Figure 1.** Cement production and consumption per capita in IRAN (1963-2007) [5]

![Graph 2](image2.png)

**Figure 2.** World Cement Production
The most serious problem with cement industry is that it is a major CO₂ emitter causing global warming. With every ton of cement produced, almost a ton of CO₂ is emitted [6]. About 0.5 tons comes from the decomposition of limestone and the balance is generated by power plant supplying electricity to turn the kiln and ball mills to grind the cement plus the fuel burned to fire the kiln. All other sources of CO₂ emission such as operating ready mix trucks adds only a minor. In terms of conventional concrete mixtures (not containing fly ash, slag or silica fume), about 480 kg of CO₂ is emitted per cubic meter of concrete or 20 kg of CO₂ per 100 kg of concrete produced. All of this amounts to about 7% of the total CO₂ generated worldwide [7]. Enhanced efficiency is not likely to change this but the replacement of some of the cement by a supplementary cementing material not associated with CO₂ emissions can substantially reduce these emissions.

**Nitrous Oxide Emissions**

Nitrous oxide emissions come from burning gasoline, coal or other fossil fuels. Ozone is formed when nitrogen oxides and volatile organic compounds mix in sunlight. The volatile organic compounds come from sources ranging from industrial solvents to volatile resins in trees. Ozone near the ground can cause a number of health problems such as asthma attack, sore throat, coughing and other health difficulties. In addition, nitrous oxide, carbon dioxide and methane are the most important greenhouse gases [8].

The NOₓ emissions from Canadian cement kilns range from 1.5 to 9.5 kg/tones of
clinker produced with a proposed limit of 2.3 kg of NOx per tone [6]. Using 2.3 kg of NOx per tone, the world release of NOx by the 2130 million tones of cement produced in the year 2020 would be 4.85 million tones of NOx. This is a fifth of the NOx released in all of continental Asia in a year [9]. Reduction in nitrous oxides is normally achieved by reducing the burning temperature or by injecting ammonia compounds into the high temperature exhaust stream [6]. This seems like a good idea but when these actions are taken to reduce the NOx in coal fired electric power generating stations, it adversely affects the quality of the fly ash produced. The fly ash then needs to be treated to remove the unburnt coal and ammonia gas before it can be used in concrete mixtures and several plants doing this are in operation.

- **Particulate Air Emissions**

Particulate emissions from the exhaust gases range from 0.3 to 1.0 kg/tone. It is normally very rich in sodium and potassium which have vaporization temperatures of only 883°C and 774°C respectively. In the past, before there was a concerted effort to capture the particulate emission, the sodium and potassium plume from cement plant chimneys settled over the countryside where it helped to combat acid rain. Now it is mainly carried out in the clinker stream where it creates problems with alkali aggregate reaction [10].

### 2.1.2. Aggregate

The coarse and fine aggregate content in concrete products is approximately 80% and it covers < 3% of emissions and energy used. The environmental burdens from procurement of aggregate consist of:
- using raw-materials;
- using land; and
- using energy (in extraction, excavation and crushing of stone materials and in transportation) which causes emissions into air.

Besides these, crushing causes dust emission and quarrying causes land and stone waste. The energy used in gravel excavation and crushing is much less than in production of building materials where heating or grinding is employed (i.e., cement production). Energy used in stone crushing depends on the desired size fraction. Table 1 shows an example of energy used for gravel excavation and Table 2 in quarrying and crushing.

**Table 1: Energy used in gravel excavation [2]**

<table>
<thead>
<tr>
<th>Gravel excavation</th>
<th>0.02 MJ/kg</th>
<th>70%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravel transport (10 km)</td>
<td>0.01 MJ/kg</td>
<td>30%</td>
</tr>
<tr>
<td>Total</td>
<td>0.03 MJ/kg</td>
<td>100%</td>
</tr>
</tbody>
</table>

**Table 2: Energy used in gravel production [2]**

<table>
<thead>
<tr>
<th>Quarrying + Crushing</th>
<th>0.05 MJ/kg</th>
<th>80%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Removal and transportation</td>
<td>0.01 MJ/kg</td>
<td>20%</td>
</tr>
<tr>
<td>Total</td>
<td>0.06 MJ/kg</td>
<td>100%</td>
</tr>
</tbody>
</table>
2.1.3. Water
The water shortage in Iran is a serious problem and this intensified by seasonal rainfalls. Only 10 percent of the country receives adequate rainfall for agriculture; most of this area is in western Iran. Concrete manufacturing uses normal tap water to make up about 0.06 – 0.10 kg/kg concrete. The total water supply system consumes very little energy for water purifying and delivery so the overall environmental impact remains small [2]. On average of each ready mix about half cubic meter of concrete remains in the truck per day. After concrete is discharged there is still about 300 kg of solids (cement, sand and stone) that it is necessary to washed out, with about 1000 liters of water. In the past the returned concrete and the solids were dumped in a pit at the job site or at the plant. Considering that this represents 2 to 4% of the total concrete produced, it is now considered too valuable to waste and can be recycled or reclaimed as sand and gravel. To reclaim the sand and gravel a “reclaimer” is used. It involves adding water to the returned concrete and then agitating it followed by wet screening to obtain the sand and gravel. Also, the cement-water slurry from the reclaimer, the wash out water, water to clean the outside of the truck, plus any stormwater in the past usually was directed into somewhat inefficient settling basins and then into a local water course [11-12].

2.1.4. Chemical Admixtures
The most common admixtures in concrete are plasticizers and air entrainers. The plasticizers used include lignosulphonate salts, hydroxyl-carboxylic acid, modified melamine, naphthalene and polymers. These are synthetic organic compounds which have a deflocculating and dispersing effect. They act on the forces between solid particles suspended in water by reducing the surface tension of water. Plasticizer content in concrete is typically very low 0.002 – 0.1% (by weight of cement) so their effect on energy use and emissions of concrete products is very low. Plasticizers are non-volatile compounds which mean that their effect on the indoor climate is also inconsequential. Air entraining admixtures are invariably organic substances which help to generate microscopic bubbles of air in the fresh concrete to improve concrete frost resistance. Air entraining agents are based on carboxyl acid salts, alkyl sulphonates, and phenolethoksylates. The most common air entraining agents are made from pine oil and they are alkali metal salts. Pine oil consists of fat and resin acid compounds, which are produced from sulphate pulp processing. Air entraining admixture content in concrete is also typically very low 0.002 – 0.02% (by weight of cement) so their effect on energy use and emissions of concrete products is very low [2].

2.1.5. Mineral Admixtures
- Fly Ash
Substitution of cement or natural aggregates with industrial by-products can be done in concrete production. From an environmental point-of-view this result is saving our natural resources and land. It is possible to substitute concrete aggregate with wastes from metal productions, mining industry or mineral stone industry by-
products such as ferrochrome slag or blast-furnace slag. In some cases when using by-products the crushing and transportation can consume more energy than in procurement of the natural resources. Fly ash from power plants can be used as a substitute for cement or filler. Ash which contains desulphurization products rich in sulphate or sulphite is not suited for making concrete. By using fly ash the environmental profile of concrete or concrete products can be affected only by the ash transport burden. Approximately 10–30% of cement content can be substituted by fly ash without much effect on concrete properties [2]. Currently in Iran fly ash is not produced but natural pozzolanic materials and silica fume are available and can be used.

- **Blast-furnace slag**
  Cement can also be substituted by ground granulated blast-furnace slag. Blast-furnace slag is a by-product of crude iron production and its economical value is negligible compared to the crude iron. As such blast-furnace slag is not suited for use as concrete binder because to achieve hydraulic properties it needs to be cooled fast, dried and ground to typical cement fineness. Blast-furnace slag processing uses less energy and causes considerably less emissions than cement manufacturing, so already a small amount of cement substitution with blast-furnace slag lowered environmental burdens. Blast-furnace cement has 10% blast-furnace slag addition but it could be increased to approximately 70% of the total binder content. Compared to concrete composed of 100% cement, the appropriate use of blast-furnace slag in concrete products as a cement substitute decreases environmental effects [2]. The bad quality of blast-furnace slag in Iran limited its content in concrete to 10-15%.

**2.2. Durability of Concrete for Eia**

One of the most important considerations in the concrete industry is the durability of concrete. When a concrete structure has inadequate durability it causes solid waste generation sooner than it is expected. From an environmental point of view it is essential that every concrete structure should continue to perform its intended functions, which are maintaining its required strength and serviceability during the specified or traditionally expected service life. Improving concrete durability can cause a decrease in solid waste generation in building industry. Therefore quality control of concrete in the Persian Gulf environment has a long record of stigma attached for its harsh climate, desert features and saline waters that do not render the longevity of concrete is very important [13]. Use of supplementary cementitious materials such as silica fume and blast furnace slag with Portland cement has increased durability of concrete in the Persian Gulf region and therefore decreased solid waste form short-lived concrete.

**2.3. Recycle of Industrial and Building Wastes for Use in Concrete**

Recycled aggregate concrete has become the focus in the past decades due to its great environmental effect. In the USA, about 30 million tons of concrete has to be discarded each year. This number increased from 55 million tons in 1980 to 162
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million tons in 2005, approximately tripled in less than 30 years. China faces a more serious problem to cope with discarded concrete with the increasingly rapid urbanization process. For example, Shanghai City alone, wastes concrete 20 million tons annually. Other reasons may also lead to waste concrete. An 8.0 magnitude earthquake in Sichuan province, China on 12th May, 2008 caused collapse of at least four millions houses, which produced tremendous discarded concrete. Similar problems are presented in Iran. Recycling of concrete is, therefore, becoming increasingly important to ensure sustainable development both in world [14].

Waste management and disposal is a major environmental concern in many countries and increasingly becoming a significant environmental, health, and aesthetic problem that is not easily solved [15]. Therefore due to the increasingly serious environmental problems presented by hazardous industrial wastes, the feasibility of burying these materials as aggregate in concrete would be of great interests.

3. CONCLUSION & DISCUSSION

- Constructional industry as the largest consumer of natural materials produces the most portion of the wastes in the country. The concrete industry has potential for environmental pollution.
- With the exception of CO₂ and NOₓ emissions, by using our current technology all the perceived environmental problems with concrete can be effectively resolved. The concrete industry needs to focus on these two greenhouse gases.
- In the concrete industry the easiest and most effective way to reduce greenhouse gases is to increase the use of such silica rich by-products as slag and silica fume and natural pozzolanes, thereby reducing the amount of cement used per cubic meter of concrete.
- Concrete industry generates about 7% of the total CO₂ generated globally and if we assume that only 18.5% of the cement can be replaced with slag or fly ash, then the CO₂ reduction would be 300 million tons per year world wide.
- Over the past decade the average annual increase in CO₂ emissions has been 1.3 percent or nearly 300 million tons a year. Therefore our industry could greatly help in easily reducing global warming and at the same time enhance the properties of the concrete.
- Improvement in the durability of concrete in corrosive environments such as the Persian Gulf that has a long record of stigma attached for its harsh climate, desert features and saline waters, can help to reduce solid waste generated from short-lived concrete in these regions.
- The recycling of industrial and building wastes for use in concrete can have significant effect for the environment.

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WASTE CEMENT AND CONCRETE MANAGEMENT AS COST-EFFECTIVE AND ENVIRONMENT FRIENDLY: PRINCIPLES AND PERSPECTIVES

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ABSTRACT
The use of recycled aggregates in concrete opens a whole new range of possibilities in the reuse of materials in the building industry. The utilization of recycled aggregates is a good solution to the problem of an excess of waste material, provided that the desired final product quality is reached. The studies on the use of recycled aggregates have been going on for 50 years. In fact, none of the results showed that recycled aggregates are unsuitable for structural use. Using the recycled aggregate is a cost-affective and environmental friendly solution which is required general waste concrete management knowledge. This paper is focusing on waste cement and concrete management for optimizing the construction costs. The ways for reducing the green house gases (GHG) at cement and batch plants are suggested. A comparison among Iran industrial utilized cement volume and other countries was performed. Finally, it can be concluded that application of recycling concrete could reduce the costs by reducing truck traffic, providing the Non-Renewable Resource, Better Trucking Utilization (Reduced Costs), Allow down to 10% Deleterious Materials in Iran.

Keywords: waste concrete management, utilized cement volume, cement plants, concrete recycling, GHG reduction.

1. INTRODUCTION
The amount of construction and demolition waste (CDW) has increased considerably over the last few years. The recycling and the reuse of this material is necessary, considering the impact that the use of natural resources and non use of CDW is causing. This would not happen if the use of recycled material were possible.
The largest CDW obtained is concrete [1, 2], and it is the most used construction material nowadays. The studies with respect to the applicability of recycled concrete aggregates (RCA) are extended around the world.
Furthermore, Industrial waste is causing more and more environmental pollution. Dirty water and poisonous gases are released from factories and workplaces in ever increasing quantities. This has led to a new appraisal of a company’s obligation to society. A company is now seen as having as much responsibility for avoiding
environmental pollution outside the factory as for maintaining cleanliness inside. Companies must establish standards for disposing of their waste in a way that will not pollute the environment. The application of waste materials in cement and concrete industry can improve the ecology cycle and prevent environmental pollution. So, waste cement and aggregate for generating new types of waste concrete is needed to total management.

2. COMPARING IRAN UTILIZED CEMENT AND CONCRETE STATUS WITH OTHER COUNTRIES

Unfortunately, new concrete technology is applied rarely except special or national construction projects in Iran. Existing structures are almost heavy and low strength and advanced concrete knowledge haven’t utilized in design and construction of structures that causes in vulnerable structures subjected to ground shaking. Relation between research centers of universities and construction industry could be optimum alternative for implementation of new concrete technology in real scales.

Used cement is just 10.8% in Iran Industry. Used cement in ready mixed concrete part is 8.64% of total used cement in Iran. 2.16% of used cement is subjected to construction of concrete segments while Japan used cement is 86.4% which for ready mix concrete and concrete segments is 73.2 and 13.2% respectively. United states used cement volume is 66.7% too and according this, 55.7% and 11% of cement is related to ready mix concrete and concrete segments part respectively. Turkey and Russia used cement volume in industry is 74.4 and 71.3% respectively. 62 and 52% of used cement in mentioned countries is assigned to ready mix concrete and 12.4 and 19.3% is assigned to concrete segment respectively [3]. Comparison of industrial used cement volumes among countries is shown in figure 1.

On the other hand, utilized cement infrastructure is still incorrect in Iran. Significant part of utilized cement generate by hand. Failed structures in past
strong earthquakes have proved that existing structures is vulnerable especially in Tehran metropolitan. Past Tehran earthquakes have occurred each 150 years. Forcing the ready mixed concrete plants for issuing quality justification is effective solution as more than 75% of ready mixed concrete plants have received standard certificate. In addition, light weight structures construction for improving the seismic performance of structures is important while 20% of light weight building materials utilized in country and remained materials is exported to out of country. Application of light weight concrete in buildings is required the three administration of managers, investors and illuminates cooperation.

3. MANAGEMENT PRINCIPLES IMPORTANCE

3.1. Improves Understanding about Cement and Concrete Industry
From the knowledge of principles managers get indication on how to manage the waste concrete industry. The principles enable managers to decide what should be done to accomplish given tasks and to handle situations which may arise in waste concrete management. These principles make managers more efficient.

3.2. Direction for Training of Managers
Principles of management provide understanding of management process what managers would do to accomplish what. Thus, these are helpful in identifying the areas of management in which existing & future managers should be trained.

3.3. Role of Management
Management principles makes the role of managers sensitive. Therefore these principles act as ready reference to the managers to check whether their decisions are appropriate. Besides these principles define managerial activities in practical terms. They tell what a manager is expected to do in specific situation.

3.4. Guide to Research in Management
The body of management principles indicate lines along which research should be undertaken to make management practical and more effective. The principles guide managers in decision making and action. The researchers can examine whether the guidelines are useful or not. Anything which makes management research more exact & pointed will help improve management practice.

4. CEMENT PRODUCTION RELEASE GHG PROCESS
Cement production generates GHG from two main sources: calcination and fuel combustion. Calcination is the chemical process in which calcium carbonate (CaCO3) is heated to high temperatures, converting it to lime or calcium oxide (CaO), and releasing carbon dioxide (CO2). So it is not surprising that the main type of GHG from cement production is carbon dioxide (CO2). The amount of CO2 released due to the calcination process alone usually varies from 50 to 60 percent the total amount of CO2 released during cement production. The remaining 40 to 50 percent is mainly due to fuel combustion. The contribution of each of these sources (calcination and fuel combustion) depends on energy efficiency. The
percent of CO2 released from fuel combustion in efficient cement plants tends to be lower since less fuel will be needed to produce the same amount of cement. Figure 2 shows the cement plant.

Figure 2. Cement plant

4.1. Ways to Reduce Cement GHG at Cement and Batch Plants

4.1.1. Blending SCM at cement plants
Blending cement with Supplementary Cementitious Materials (SCMs) reduces GHG emissions. Common SCMs in use include slag, fly ash, silica fume, and calcined clay. Using two or more SCMs together with portland cement is referred to as a ternary cement mix. Proper use of ternary mixes comprised of fly ash and slag produce not only less but also better quality concrete. The addition of SCM at cement plants has the potential to significantly impact GHG savings.

4.1.2. Environmentally friendly fuel for cement kilns
Use of environmentally friendly fuels would reduce GHG emission by using less carbon intense fuels. Although coal is one of the most efficient and cost effective fuels for heating a kiln, it is also one of the most intense in terms of the CO2 emissions. Therefore, it is important to use alternative fuels instead, such as recycled materials. For example, In 2005, fuel combustion from coal constituted about 73 percent of all emissions from fuel combustion by cement plants in California. This number has decreased since 1990 when coal was responsible for about 85 percent of all fuel combustion emissions.

4.1.3. Using of interground limestone
The limestone addition strategy consists of replacing cement with interground limestone. Since interground limestone is added at the end of the cement production line, the cement-related greenhouse gas (GHG) emissions will be reduced proportionally to the amount of limestone added. The GHG savings arise from avoiding GHG emissions associated with cement production during fuel combustion and calcination in the kiln.
The maximum GHG savings generated by this strategy is 5 percent, which is the maximum limestone allowance per American Society of Testing Material (ASTM C 150), a major nationwide cement specification. Since the effect of limestone addition to cement had not been studied in detail, Caltrans sponsored a comprehensive study, use of raw limestone in portland cement, designed to evaluate the three primary indicators of concrete performance: strength, drying shrinkage, and permeability[4]. It was found that limestone improved strength and permeability (at early ages).

Since 2007 Caltrans has been accepting 2.5 percent limestone addition. After concluding the limestone study, Caltrans will accept the full 5 percent but implement a performance-based specification to control shrinkage. Although 5 percent of limestone is allowed per ASTM C 150, it is estimated that the statewide limestone addition may not exceed 3.5 percent based on manufacturing limitations. According to reports from the Portland cement association (PCA), the estimated average nationwide is only 2.5 percent [5].

4.1.4. Production efficiency improvements
Significant GHG emission reduction for California is not expected of this particular strategy. One of the reasons being that the cement industry in California is already among the most energy efficient in the world. It has been reported that one of the most recent cement plants built in California has a GHG intensity of only 0.02 below that of the 2005 California average GHG intensity factor of 0.86 ton of CO2 per ton of cementitious material.

All cement plants in California except one have precalciner. These pre-heaters significantly improve energy efficiency by heating limestone prior to its placement in the cement kiln. Another process-related piece of equipment that significantly improves energy efficiency is the dry kiln compared to wet kiln usage: All cement plants in California have dry kilns. Using one kiln instead of multiple is also recommended to further improve energy efficiency. Only one cement plant in California uses multiple kilns. According to the California Cement Industry (2008), the energy efficiency of California cement plants is 15 percent better than the average U.S. value since 1995.

Another reason for a small GHG emission reduction is the fact that production efficiency improvement only affects 40 percent of the GHG emission from cement production. The other 60 percent of GHG emissions comes from calcination, which is a natural chemical process inherited from the cement production process in which limestone (CaCO3) is converted to calcium oxide (CaO) by releasing carbon dioxide (CO2) in the presence of heat, as follows:

\[
CaCO_3 + \text{heat} \rightarrow CaO + CO_2
\]

Since fuel combustion is responsible for only 40 percent of the GHG emissions, a reduction in energy consumption of 4.5 percent accomplished by California Portland, only reduced GHG emissions by 1.8 percent. California Portland was
4.1.5. Optimizing cement content at batch plants
Optimizing cement content can be prescribed as a strategy to reduce GHG. In some cases, a higher amount of cement is used because of the desired early concrete strength. For instance, a homeowner or general contractor may need concrete with a compressive strength of only 14 MPa. This 14 MPa concrete only needs about 135 Kg of cement to gain this strength at 28 days. That strength requirement can be met if the homeowner or contractor allows more time for concrete to gain strength. Another option for homeowners and general contractors would be the use of admixtures to accelerate the strength gain of the concrete mix. While there is a cost to these admixtures, they can be used to reduce GHG through the reduction of cement. To optimize the amount of cement, concrete mixes can also reduce the amount of water used since this results in a stronger concrete. Reducing the amount of water would also involve some additional cost for plasticizer or water-reducing admixtures. This cost may be compensated for by cost savings of optimized cement content.

Example: when a mix for a concrete driveway replaces 25 percent of the cement with fly ash, and uses half the normal amount of cement, as much as 5.7 tones of CO2 can be saved per driveway, assuming each ton of cement emits 0.9 tons of CO2. This is equivalent to the CO2 emissions from about one passenger car for the entire year, as the average passenger car emits about 5.2 tones of CO2, based on data from the Environmental Protection Agency (EPA).

To calculate this emission, it was assumed that each car travels 19,300 Kilometer per year, gets about 8,631 Km per cubic meter and each cubic meter of fuel emits 2,235 tones of CO2 per cubic meter. To achieve this savings, it may be necessary to keep cars off of the driveway longer. If it is necessary to get strength faster so vehicles can access the driveway, these GHG savings can still be obtained by adding an accelerator to the mix at a concrete cost increase of about 10 percent.

4.1.6. Reducing concrete waste
This strategy seeks to significantly reduce the concrete waste occurring at batch plants. It is estimated that approximately 5-8 percent of the concrete that is made in California every year is returned to the batch plants as waste. Concrete waste (or concrete returned to batch plants) is generated for the following two main reasons: 1) a load of concrete is not completely used, or 2) a load of concrete is rejected by an inspector due to the mix not meeting some specified characteristic. The worst case for a return in terms of GHG is when the plastic concrete is separated back to sand, gravel, and water, and the cement was then truly a waste product. Concrete is almost always left over at the end of a job. The main reason is because it is more cost-effective to overestimate rather than be short of the material needed. Here are a few ways to reduce waste, and consequently GHG:
1. Better estimating of total concrete requirements.
2. Use of volumetric trucks to handle the exact needs of the last quantities of the day.
3. Design locations to receive the returned or left over concrete. One of the ways to re-use the concrete would be to make concrete blocks for later sale. Another is to use that last truck to make sidewalks that may have been planned for a later placement.

5. APPLICATION OF RECYCLING CONCRETE AS WASTE AGGREGATE IN CONCRETE

5.1. Aggregates are Required for Construction Projects
Aggregates are composed of rock fragments that may be used in their natural state or after mechanical processing such as crushing, washing, and sizing. Natural aggregates consist of both sand and gravel, and crushed stone. Recycled aggregates consist mainly of crushed concrete and crushed asphalt pavement. Construction aggregates make up more than 80 percent of the total aggregates market, and are used mainly for road base, riprap, cement concrete, and asphalt. Aggregates provide bulk, strength, and wear resistance in these applications. Construction aggregates increased from 36 percent of all raw materials used in the United States in 1900 to 70 percent in 1958, a compound annual growth rate of 1.15 percent. From 1958 to 1998, Americans have maintained their use of construction aggregates at 70–73 percent of their total raw material demand [6].

5.2. Concrete Recycling
Aging U.S. infrastructure, decreasing availability of landfill space, and environmental concerns work together to increase concrete recycling. There are two approaches to recycling concrete. One alternative is to haul the concrete debris to a permanent recycling facility, usually close by to minimize transportation costs, for crushing and screening. The other approach is to do the crushing and screening at the demolition site where the aggregate is reused as soon as it is processed. Recycling at the demolition site reduces heavy materials hauling, thereby reducing transportation costs, energy use, and wear and tear on roads and equipment. Figure 3 shows the schematic flow of concrete recycling. Figure 4 shows the stages of concrete recycling [7].

![Figure 3. Schematic flow of concrete recycling](image-url)
5.3. Recycling-Small Market Share, But Large Tonnage
Construction Materials Recycling Association, Lisle, Illinois, states that about 100 million t of concrete is recycled annually into usable aggregates. Aggregates produced from recycled concrete supply roughly 5 percent of the total aggregates market (more than 2 billion t per year), the rest being supplied by aggregates from natural sources such as crushed stone, sand, and gravel. Preliminary data indicate that in 1998, 3,400 U.S. quarries produced about 1.5 billion tones of crushed stone, of which about 1.2 billion tones was used in construction applications. About 5,300 sand and gravel operations produced more than 1.0 billion t of construction aggregates in 1998. Application of recycling concrete could reduce the costs by reducing truck traffic, providing the Non-Renewable Resource, Better Trucking Utilization (Reduced Costs), Allow down to 10% Deleterious Materials in Iran.

5.4. Concrete Recycling Product
The bulk of the aggregates recycled from concrete—an estimated 68 percent—is used as road base. The remainder is used for new concrete mixes (6 percent), asphalt hot mixes (9 percent), high-value riprap (3 percent), low-value products like general fill (7 percent), and other (7 percent) [8]. The low usage rate of recycled aggregates from concrete (15 percent) in high-value new concrete and asphalt hot mixes, compared to the higher usage rates in lower valued products, is related to quality issues, both real and perceived. State agencies have been slow to accept recycled aggregates from concrete for high-quality uses such as road surfacing. Specifications, based on considerable research and favorable in-service experience, have allowed its use mostly as road base material. Some States are
experimenting with the conversion of existing worn-out concrete roads to rubble-in-place. The old concrete surface is broken up and compacted, and asphalt pavement is placed over the enhanced base, composed of the original base and the new layer of compacted rubble.

6. LIFE CYCLE ENERGY AND CO2 OF HIGH QUALITY RECYCLED AGGREGATE BY HRM

There is a developed technology for producing high quality aggregate from demolished concrete using a "heating and rubbing method" (HRM) [9]. Using this technology, aggregate can be recycled as raw, material for ready mixed concrete, while fine powder (HRM powder) from cement paste can be recycled as raw material for cement, cement admixture, or soil stabilizer. The HRM uses a considerable amount of energy to heat and rub concrete. Life cycle CO2 and energy of the recycled aggregate are calculated to evaluate this technology. The recycled aggregate is produced from demolished concrete and the HRM powder is used for a soil stabilizer in case 1-1. The HRM powder is used as part of cement raw materials in case 1-2. The production of crashed stone, which is the most popular aggregate, is calculated in case 2. The result of life cycle CO2 is shown in figure 5. In case 1-1 and 1-2, the life cycle CO2 is a negative value because the deduction of CO2 emission during cement manufacturing by the powder is much larger than the emission during recycled aggregate production. In case 2, the CO2 emission from crashed stone production is very small but still positive. This method is proved to be very effective to reduce CO2. As for the life cycle energy, its use of recycled aggregate is greater than that of crashed stone as ordinary aggregate because the deduction of energy consumption during cement manufacturing by the powder is relatively small [9].

Figure 5. Life cycle CO2 of high-quality recycled aggregate by HRM
7. CONCLUSION
Concrete recycling has proven to be profitable, but its use has limitations. Transportation costs need to be kept low, which forces the market to be urban-oriented. The market for recycled aggregates may be restricted by user specifications and prejudices. Finally, the availability of feedstock into recycling plants is fixed by the amount of demolition taking place, which generally places the activity within older, larger cities. Depending on the size of the recycling facility, entry into the aggregates recycling business requires a capital investment. Processing costs for the aggregates recycler again depending on the size of the operation. The larger operations distribute costs over more units of output. The average production capacity for a fixed site recycling operation should be determined. Prices for the various aggregate products made from recycled concrete should be evaluated from region to region.

Recyclers often have the opportunity to charge a fee for accepting concrete debris, especially where landfill space is running short and charges for depositing materials into landfills are high. In such cases, the added revenue can compensate for a lower market price for the recycled aggregate product. As natural aggregate producers dominate the market, they tend to set the terms that recyclers can obtain. The future for recycled aggregates will be driven by reduced landfill availability, greater product acceptance, continuing government recycling mandates, and the continuing decay of a large stock of existing infrastructure, as well as by the demands of a healthy economy.

On the other hand, the industrial utilized cement infrastructure could be improved by observing the ways to reduce cement GHG at cement and batch plants, optimizing cement content at batch plants, reducing concrete waste, application of recycling concrete as waste aggregate in concrete in Iran.

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FEASIBILITY OF CONCRETE USAGE IN RURAL HOUSING IN IRAN

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ABSTRACT
Concrete has been used since the 1970s in construction of rural houses in Iran. Despite extensive use of concrete in rural housing in Iran and the huge allocated budget, the quality of construction is not acceptable. The aim of this paper is to evaluate the influence of using concrete in construction of rural houses in Iran. For this purpose, the effective factors on the development process of rural housing have been identified first. They could be counted as:

1. Efficiency and Effectiveness
2. Compatibility with climate, economy and social aspects
3. Dynamism and Flexibility
4. Stability and Durability
5. Cultural continuity and visual desirability

Then, sample cases -25 villages- have been surveyed through these factors by authors. Minimum of 3 houses have been surveyed in each village. The result of this research recognizes some problems of using concrete in rural housing in Iran. The most common difficulties are providing materials with good quality, lack of technical knowledge in villagers as the main workforces and lack of required equipments.

Keywords: rural housing, feasibility, concrete

1. INTRODUCTION
According to development of rural settlements in Iran, current use of concrete in building construction of rural areas has been begun since the seventieth decade (c.c.). Concrete has been used in the following methods of construction in Iran:

- Bearing wall system with tie beams and columns
- Structural reinforced concrete with beam and column or moment-resisting frame
- Reinforced concrete slab

In the above classification, the first one has the maximum usage in rural settlements of Iran. Despite extensive use of concrete in rural housing in recent years in Iran, the quality of construction has not been acceptable, according to the experts' views. This lack of desirability is related to structural and architectural aspects. However, efforts undertaken in the development of rural housing by making use of new material and methods of construction- such as concrete- have
not considered all related aspects of housing development in rural areas. This paper is going to evaluate use of concrete in building construction by a general and multi-aspect overview considering physical aspects of rural concrete constructions which is related to social, economical and cultural properties of rural housing. The result of this research is to declare the difficulties of making use of concrete in renovation of rural housing in Iran.

2. RESEARCH METHOD
Effective factors on renovation process of rural housing are identified first. Identification of these factors is a process in itself and is not covered in this paper. However, a summarized explanation of steps of exploiting the factors is mentioned in continuation of the report. Final factors and evaluation of the influence of using concrete in Rural Construction by them has been introduced. The factors are based on derivation of a conceptual framework effective on selection of proper rural housing development method, using the existing resources. In this condition, the framework is chosen out of the following domains:
1. Theories including identification of the concept of village and rural housing, theories connected with the way of interfering in man-built environment, especially rural regions (with emphasis on theories relying on dwellers' participation) and theories related to proper technology
2. Internal experiences including macro-policies and programs and experiences on construction and reconstruction of rural housing
3. Experiences of other countries (with emphasis on countries that have similar condition as Iran)
The research is based on surveys in rural samples. The research method is Qualitative and based on observation (profound observation) and interview (focus group). The factors and sub factors have given a checklist for surveying. Each factor has been supported by a question in questionnaire that the surveyor answers by observation.

3. SAMPLING
Sample cases have been selected through a categorized random sampling method. Statistical community consists of villages that have minimum of 15 implemented projects of rural house renovation. Also minimum of three years might have passed from implementation of the projects. In accordance with the sampling method, statistical community has been categorized in a few clusters through the following criteria:
1. Climate categorization of Iran
2. Cooperation of villagers
3. Population
Then, a portion of each cluster has been calculated on the scale of each cluster to the whole amount of final samples. Due to validation of sampling, amount of final samples has been set to 25 villages. Finally, in accordance with the portion of each cluster in statistical community, sample cases have been selected randomly from each cluster. Sample cases have been surveyed by authors with prepared checklists trying not to intervene in the prevailing situation of the villages and houses. A
minimum of 3 houses have been surveyed in each village. Checklist focuses on five major aspects of renovated rural houses:

1. Efficiency and Effectiveness
2. Compatibility with climate, economy and social aspects
3. Dynamism and Flexibility
4. Stability and Durability
5. Cultural continuity and Visual desirability

Each aspect has been surveyed by a number of correlated indicators mentioned below (refer to 4). Achieved data has been categorized and analyzed. Finally, guidelines are provided in accordance with the results.

4. FINAL FACTORS, MEASUREMENT COMPONENTS AND THEIR EFFECT ON DEVELOPMENT OF HOUSING

Final factors analyzed in the rural settlements of Iran and measuring components of each factor are as the following: In continue, Summarized results of analysis of data are mentioned in accordance with the above mentioned factors:

4.1. Executive Efficiency With Respect To Workforce and Cost

The objective is the complete fulfillment of related necessities by developing the building and providing smooth trend of its implementation in a way that interfering factors do not stop or delay the work trend. In addition, the budget spent to develop the housing and the special procedures taken to reduce spending are the criteria for logicality of the cost with respect to priority needs of the household.

Components for measuring the factor are: speed of implementation [3, 610, 15, 16, 24, 26, 30, 31, 32], independency of expert forces in making [1, 3, 6, 16, 26, 29, 31, 32], repairing and maintaining [1, 5, 6, 19, 23, 24, 26, 29, 30, 31, 32], lack of need for sophisticated and unavailable equipments [1, 3, 4, 5, 11, 15, 16, 19, 23, 32], achieving gradual durability [1, 3, 4, 5, 11, 15, 16, 19, 23, 32], possibility to avoid flaws and mistakes [1, 3, 4, 5, 6, 7, 10, 11, 15, 16, 19, 23, 24], ease of implementation [11, 15, 16, 19, 26, 29, 32], smoothly accessed resources [11, 15, 16, 19, 23, 31, 32], low cost construction and maintenance [1, 10, 16, 19, 23, 24, 26, 32], fair balance between housing budget and credits allocated to it [23, 24, 30, 31, 32], compatibility with current economic conditions and lifestyle, increased self confidence, avoiding luxury [1, 3, 4, 5, 6, 7, 10, 11, 15, 24, 31, 32], desirability to work and to construct a house, convertibility to a capital commodity. [4, 5, 6, 7, 10, 11, 15, 16, 29, 30, 31, 32]

Field studies clearly show a decrease in the quality of concrete constructions. It might be a result of the following reasons:

1. Lack of required equipments and machineries
2. Difficulties in providing high quality material such as mixing water, fine aggregate, coarse aggregate and additives
3. Lack of expert labor in production process.

Also, increasing the cost and time of the construction process has been observed. This is the result of non-industrial process of construction. (Figure 1,2,3,5,6,7)
4.2. Climate adaptation with natural environment

The concern is physical accordance of the building and its construction scheme using climatic elements and employing proper mechanisms to provide comfort relative to the elements used for exploitation of potentials to counteract its negative effects.

- Components for measuring the factors: wind, rain, humidity, sunshine, and temperature. [3,11,15,16,18,20,26,29,30,31,32]

In humid zone, because of high heat capacity of concrete elements, undesirable effect in thermal adjustment, has been seen.

4.3. Physical Effectiveness

- The concern is physical accordance of building with requirements, needs and demands of the residents such that fulfilment of the said needs is predicted in the development method and that physic and spaces created in this method have adequate capability to provide for engineering issues, human factors and are in compliance with human behaviours.

Components for measuring the factor: observance of proportions and dimensional and space criteria from operational efficiency and visual desirability [3,4,6,30,31,32], hygiene [10,11,19,29,30,31,32], caring for tastes and demands of the residents[15,16,19,29,30], compliance with needs [10,11,15,16,19,29,30], avoid shortages [19,29,30,31,32], provide safety [3,4,6,10,11,15,16,19], establish proper relationship between open and closed spaces[3,4,6,10,16,19,29,32], capability to construct infrastructures and facility services.[11,15,16,19,29,30,31,32]

Using concrete increases length of span up to 7 m which improves architectural characteristics of renovated houses. Diversity in interior design and space planning is the result of such structural potentials. Physical effectiveness of renovated concrete structures improves the ability of architectural design in shouldering responsibility to the new style of life, hygienic needs and infrastructures.

4.4. Social Capabilities and Compatibilities

The intention is to create coordination with social under-layer and its expected social functions, such that in proportion with configurations, values, beliefs and traditions, the building would be able to offer a desirable function to the residents relative to other sects of the society.

- Components for measuring the factor: care for social values [1, 3, 4, 5, , 20, 23, 25, 26, 29, 32, 33], coordination with social functions [1, 3, 11, 13, 14, 15, 16, 19, 20, 23, 25, 26, 29], enhancing sense of cooperation and contribution, control and management of the work [6, 10, 11, 13, 14, 30, 31, 32, 33], self-sufficiency, improved quality of the dwellers' life style and culture. [16,19,20,23,25,30,31,32,33]

Vast use of concrete structures decreases capabilities and social compatibilities, because of its need to expert labours. It also results in low proficiency of houses and also, low innovations in the design process by habitants.
4.5. Stability
The intention is the resistance of building against usual, common, unconventional and temporary natural forces and erosion due to withstanding and enduring the environment and its effects and making use of the building all along its life cycle.

- Components for measuring the factor: resistance against earthquake, [6, 10, 24, 29, 30, 31, 32] natural and atmospheric conditions [1, 3, 4, 5, 6, 10, 24, 29, 30, 31, 32] construction quality[1, 3, 4, 5, 32], and durability. [1, 4, 5, 6, 10, 24]

Reinforced concrete structure has a good lateral and vertical stability and also desirable durability, if it has been implemented perfectly. But in rural construction, because of low quality of the process of producing reinforced concrete constructions -such as inappropriate designs, bad curing, missing components portion etc, stability of rural concrete structures could not be guaranteed.

4.6. Dynamism
Dynamism means growth, movement and possibility for optimization of the building based on dwellers' ideas and desires. A dynamic building is the one that is capable of forming a process and a continuous life.

- Components for measuring the factor: partial diversity [1, 3, 10, 11, 29, 30, 31, 32], changeability [4, 5, 6, 10, 11, 26], development possibility. [1, 3, 4, 5, 6, 24, 29, 30, 31, 32]

Lack of expert labours, materials and equipments of concrete construction is a serious limitation for future development and growth of houses. Also, low diversity in design process of building and detailing has been seen. (Figure 8)

4.7. Economic Development
The concern is to measure effects of the chosen approach on economic growth of the region and improvement of economical development factors in the region.

- Components for measuring the factor: expand the native industries and increase production capacity [,7,8,10,11,18,20,24] , improve regional commerce of the area [1,6,7,8,10] , reduce unemployment and train expert men[15,16]

Because of low economic ability of villagers, concrete constructions are not affordable in many rural areas of the country. On the other hand, industrialization of concrete production has had visible effects on growth of economic indicators such as.

4.8. Coordination and Protection of Natural Environment
The intention is to find out how loyal a building development style and the capabilities embedded in a long constructed building during its different utilization steps have been in keeping up with the rules and basics of conserving the environment and how much have they protected the natural resources from getting damaged. In addition, coordination with natural bed, meaning the proportionality of the development procedure, hidden capacities of the structure in its different utilization stages with current environmental conditions, optimum usage of environmental conditions and natural forces must be considered.
The most important problem of concrete buildings, according to this factor is recycling. The elements of buildings which have been constructed with concrete could not be recycled or reused in the other way in buildings. It has some visible effects on increasing pollution of closed environment of rural areas.

4.9. Cultural Continuity
The intention is the extent of notice given to the historical bed on which the building is constructed, in a way that the developed building is in logical accordance with construction traditions and that the resulting product does not contradict with the existing physical background.

New buildings which have been constructed by new materials and methods –such as concrete- often cannot provide acceptable architectural correlations to their existing context. Because of lack of skill in villagers to use these new methods of building and materials appropriately, the result is not in continuity with vernacular architecture. (Figure 4)
5. CONCLUSION
Concrete is one of the most important based materials for building construction. In spite of that, difficulties of its implementation in rural areas cause low efficiency. Most of the difficulties encountered can be outlined as the following:
- Difficulties in providing good quality materials, such as water, aggregates and appropriate type of cement.
- Low quality of particle size distribution.
- Lack of technical knowledge of implementation of reinforced concrete.
- Inappropriate curing of concrete.
- Inappropriate structural design.
- Missing standards in time of haulage.
Considering the above achievements, the solutions for increasing the efficiency of concrete usage are:
- Developing vernacular knowledge in concrete construction technology and methods.
- Providing and developing industrialization in concrete construction process and productions.
- Providing appropriate standards for implementation of concrete constructions in rural areas.
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A LABORATORY INVESTIGATION ON THE EFFECTIVE PARAMETERS OVER THE PENETRABILITY OF ROLLER COMPACTED CONCRETE

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ABSTRACT
Roller Compacted Concrete construction technique has been recently remarkably considered in Iranian academic centers and dam engineering industry. However, research studies can hardly be found in which physico-mechanical properties of RCC have been studied as in a real ongoing project. In this study, permeability of the mass of RCC mixture used in Zirdan RCC dam located in south of Iran, the second large RCC dam in Iran has been investigated. Influences of cementitious material content, water-cement ratio, pozzolan replacement ratio in cementitious material, delay in working time and age of concrete specimens on permeability coefficient have been studied. Moreover, effects of different types of pozzolan on permeability have been examined. Results showed that RCC has an equal or even lower permeability coefficient in comparison to an equivalent ordinary concrete. Effect of water content on permeability was considerable and in comparison to cementitious material content, showed a higher degree of importance. The rate of developing permeability coefficient (decreasing) was found faster than the rate of mechanical strength development (increasing). Delay in working time decreased the permeability of RCC. However, decreasing the water content below its optimum limit would result in an excessively high permeability coefficient. Finally, it was observed that silica fume had a significant effect on permeability coefficient.

Keywords: RCC, permeability coefficient, pozzolan, silica fume, working time

1. INTRODUCTION
Permeability of RCC mass is one of the most important parameters in RCC dams, and also in Roller Compacted Concrete Pavements (RCCP). This is due to direct relationship between this parameter and problems such as water leakage through dam body, pore water pressure, stability in freezing and thawing cycles, and durability requirements. Water leakage may have deteriorating effect on hardened RCC strength by washing away cementitious materials. Also, freezing and thawing cycles would have undesirable effects on highly permeable concrete. On the other hand, design of mixtures with sufficient impermeability may lead to omission of several extensive works such as upstream
impermeable faces, resulting in positive effects on technical and economical aspects of project.

Two different aspects of RCC, concrete and soil aspects, have led to two different theories about RCC permeability. Considering concrete approach, permeability is related to the content of cementitious material in RCC mixture [1,2]. According to soil approach, on the other hand, increasing the amount of cementitious material will not result in significant change in permeability coefficient in the case designing a suitable RCC mixture proportioning. In fact, it is possible to achieve RCC permeability coefficient as low as a conventional concrete mixture's and using low content of cementitious material [3].

Although a few studies have been formerly conducted on RCC permeability [4,5], our understanding of this vital property of RCC remains far from adequate. In addition, pure theoretical researches cannot provide useful, adequate and practical ways to address RCC issues which are currently just the fruit of innovations of contractors and designing engineers.

Having these facts in mind, we were encouraged to investigate the influence of several factors on RCC mixture permeability of Zirdan RCC dam, the second large RCC dam in Iran considering technical, economical and constructional aspects. The dam's study phase test fills of this dam is going on and the placement works is about to start.

2. EXPERIMENTAL WORKS
2.1. Materials
As previously mentioned, materials used in this study were those ones currently used in Zirdan RCC dam. The details are as follows:

Zirdan riverbed materials have been used as aggregates. As it is currently employed in site, two classes of aggregates have been considered: 1- sand (0-4.75 mm) 2- gravel (4.75-37.5 mm), these two classes are mixed in a 40:60 proportion, respectively.

Cement Type II has been used in study. Results of its chemical analysis and physical properties are presented in Table [1]. The main applied pozzolan is Khash pozzolan. A type of silica fume, slag and another natural pozzolan type (Trass) have been employed for comparison. Pozzolan chemical analyses are presented in Table [2].

In order to respond to research questions, several mixture proportions have been applied.

Results obtained from all tested mixtures with their physical and mechanical properties are presented in Table [3].

<table>
<thead>
<tr>
<th>Table 1: Cement chemical composition</th>
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</tr>
<tr>
<td>-------</td>
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<tr>
<td>21.31</td>
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Table 2: Pozzolan composition.

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<th>K2O</th>
<th>SO3</th>
<th>MgO</th>
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<th>Fe2O3</th>
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<th>SiO2</th>
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<td>2.66</td>
<td>0.1</td>
<td>1.7</td>
<td>7.8</td>
<td>4.96</td>
<td>14.54</td>
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<td>0</td>
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<td>90</td>
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<tr>
<td>Trass</td>
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<td>67</td>
</tr>
<tr>
<td>Slag</td>
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<td>1.6</td>
<td>9.06</td>
<td>38</td>
<td>0.71</td>
<td>10.5</td>
<td>35.7</td>
</tr>
</tbody>
</table>

2.2. Test Procedure

Cylindrical specimens have been made according to ASTM C II 76-98 [6], using a modified Ve Be table; the only difference to the mentioned code is that the specimens are made in two 10 cm high layers. Therefore, their height is 20 cm. It means 10 cm shorter than the standard specimens. The specimens have been stored in 20 ± 2°C water.

The measurement of the permeability coefficient was performed by the method shown in Figure [1]. During 48 hours before the permeability test the cylindrical specimens were conditioned at 50 % HR and 20°C. Then, the 6 atm water pressure was applied for 24 hours on a circular concrete surface area of 7.5 cm in diameter. Surrounding this circular surface was water height by bituminous material. After 24 hours of sustained pressure, the specimen was split and the depth of penetration (d) was obtained. In this case, the permeability coefficient (K_v) was derived from the (Eq. [1]) developed by Valenta [7].

\[ K_v = \frac{d^2 V}{2ht} \]  

V is the volume voids filled by water in the penetrated zone, (determined by measuring gain), h is the head of water and t is the time to penetrate to depth (d).
3. RESULTS AND DISCUSSIONS

The permeability coefficients obtained are presented in Table [3]. Table [3] also includes compressive strength of specimens. Effects of different parameters on permeability are described below [8].

<table>
<thead>
<tr>
<th>ID</th>
<th>C+P (kg/m³)</th>
<th>P (kg/m³)</th>
<th>W (kg/m³)</th>
<th>W/(C+P)</th>
<th>Pozzolan Type</th>
<th>Consistency (sec)</th>
<th>Age (day)</th>
<th>K (cm/s)</th>
<th>fc (MPa)</th>
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<td>A1</td>
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<td>110</td>
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<tr>
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<td>28</td>
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</tr>
</tbody>
</table>

C: Cement  
P: Pozzolan  
W: Water  
K: Permeability coefficient  
f\(_c\): Compressive strength

3.1. Influence of Cementitious Material Content

As can be seen in Figure [2], an increase in cementitious material from 140 to 200 kg/m³ has not shown a significant decrease in permeability coefficient though having a remarkable positive effect on mechanical strength. According to this
Figure, mixtures having acceptable permeability coefficient would be obtained using low cementitious material content. Although increase of cementitious material would result in a better mechanical strength, no such significant effect is observed for permeability coefficient.

![Permeability coefficient Diferent cementitious material contents](image)

**Figure 2. Effect of cementitious material content.**

### 3.2. Influence of Water to Cement Ratio
According to Figure [3], optimum water to cement ratio is estimated 0.55. The diversity of results shows an implication of importance of water content in RCC mixture, instead of water to cement ratio as in ordinary concrete. Another conclusion obtained from Figure 5 is that while water content is increased from its optimum level, insignificant increase in permeability coefficient would be obtained, whereas decreasing water content from the optimum level, even very low content, leads to decrease the compaction factor, and shows an excessive increase in permeability coefficient.

![Permeability coefficient Diferent water to cement ratios](image)

**Figure 3. Effect of water to cement ratio.**

### 3.3. Influence of Pozzolan Content
According to Figure [4], the lowest permeability coefficient would be obtained using optimum replacement ratio of pozzolan (in this study, 30% for the main
applied pozzolan). Using higher replacement ratio (e.g. 50%), especially in short ages (up to 28 days) would increase permeability coefficient.

![Permeability coefficient Different pozzolan replacement ratios](image)

**Figure 4. Effect of pozzolan content**

3.4. Influences of Age in Working Time

As can be seen in Figure [5], rate of permeability coefficient development shows a different trend to that of for mechanical strength development. Mechanical strength of cemented mixtures as well as RCC increase significantly during the first 28 days, while RCC permeability coefficient do not shows remarkable decrease after 14 days in this study. This is in case that 40% of cementitious material is pozzolan.

![Permeability coefficient Different ages](image)

**Figure 5. Effect of age**

3.5. Influences of Delay in Working Time

Effect of delay in working time (compaction time) is shown in Figure [6]. Dasmeh et al. [9,10] reported that employing pozzolan in cementitious material would
extend allowable compaction time. In that study they showed that specimens containing pozzolan in their cementitious material and made after 120 min. delay in compaction time had shown an improve in mechanical strengths. The results obtained from present study on compaction time indicate that it is possible to achieve a lower permeability coefficient for specimens compacted after a delay of up to 110 min. However, the results obtained from specimens compacted with a delay longer than 110 min. illustrate a notable increase of permeability coefficient. It may be due to evaporation of water of mixture. Therefore, decreasing the water content below its optimum limit which would result in an excessively high permeability coefficient.

![Figure 6. Effect of delay on working time](image)

### 3.6. Influences of Type Ofpozzolan

The effect of use of silica fume in reduction of the permeability coefficient of Conventional Vibrated Concrete (CVC) has been shown by several authors [11,12]. For Roller Compacted Concrete (RCC), excellent effect of employing silica fume in cementitious material is presented in Figure [7]. In comparison to mixtures containing other types of pozzolan in their cementitious material, and also to mixture without any additive in its cementitious material, a remarkably lower permeability coefficient has been obtained when applying silica fume. The result is due to the fact that using silica fume in the mixtures would improve the "Transition Zone", change capillary pores to gel pores and seal their inter connections. It can also be seen that replacement of slag up to 50% of cement could lead to considerably good results. The main pozzolan applied in this study (obtained from Khash area) has also shown acceptable results regarding permeability coefficient as well as mechanical strength.
3.7. An Observation: Tortuousity Effect
Some of specimens in this study showed greater values for horizontal permeability coefficient than vertical permeability coefficient. This is an implication of fast evacuation of streamlines through side-walls. The difference between permeability coefficient of two horizontal and vertical directions can be justified as follow: Aggregates including high percentage of flat and elongated particles are allowed to be used in RCC mixtures (coarse aggregate used in this study contains 30% of flat or elongated particles). RCC compaction methods on the other hand, would arrange these particles in horizontal direction. This arrangement of particles is the reason for a phenomenon called "Tortuousity" which increases the length of streamlines in vertical direction. Consequently this increase would result in a lower vertical permeability coefficient and higher value in horizontal direction. This problem should be paid attention when evaluating RCC dam required permeability coefficient. The phenomenon "Tortuousity" is illustrated in Figure [8].

4. CONCLUSION REMARKS
Brief summery of the findings is described below:
1) Influence of water content of RCC mixture on permeability was found vital and
in comparison to cementitious material content has a greater degree of importance.
2) Employing pozzolan up to its optimum replacement ratio in cementitious material decreased the permeability coefficient.
3) In spite of mechanical strengths, the rate of permeability coefficient improvement was considerable up to first 7 to 14 days.
4) Delay in compaction time would improve permeability coefficient of specimens containing pozzolan in their cementitious material unless it results in a decrease of water content below its optimum limit.
5) Effect of using silica fume in RCC mixture was excellent. Also employing slag in cementitious material had considerable positive effects on the reduction of permeability coefficient. Considering the low price of slag, it is recommended to replace it in great percentages in cementitious material.
6) The RCC permeability coefficient could be much less than 3x10^-9 cm/s, a typical value for conventional concrete permeability when using low cement content. In case of using flat and elongated aggregates, different values would be obtained for horizontal and vertical permeability coefficient.

REFERENCES
ارائه طرح اختلاط مناسب بنن غلاتکی سد زیردان از طریق روش آزمایشگاهی

چکیده

یکی از مسائل مهمی که در زمینه سدهای RCC مطرح است، تعیین طرح اختلاط بنن غلاتکی می‌باشد که به توجه به جدید بودن نسیب بنن RCC و عدم تجربه کافی در ایران، معمولاً وقت و هزینه‌زیادی را بر پا کرده‌اند. در این مقاله، هدف بررسی عوامل موثر بر خواص بنن غلاتکی و آزادی نحوه دستیابی به طرح اختلاط مناسب بنن غلاتکی با توجه به سه عامل مراقبت فشاری (کارایی مناسب و عدم جداشتدگی)، داشته‌اند. در این راستا به این روش عمل شده که تعداد زیادی طرح اختلاط با سنگانهای شکسته و قابل ساخته شد و از نظر معادن سیمان و پوژولان، نوع و دانه بندی صلاح سنگانه ای، نوع و مقدار مواد افزودنی و مقدار آب بر روی خواص بنن غلاتکی خصوصاً مراقبت فشاری و کارایی مورد بررسی قرار گرفته است. سنگانه‌های صخره‌ای در جهت حفظ طبقه بندي درآی باتریک در اندوز می‌باشد.

یافته‌ها: با نگاهی از آن است که میزان مصالح ریزدان در حدود 9 تا 11 درصد حجمی کل مصالح مناسب است. همچنین جداشتدگی سنگانه سببیت زیادی به تغییرات میزان آب دارد. بسیاری از پوژولان در حدود 30 درصد حجم بسی به خصوصیات بنن سخت شده می‌شود. همچنین با ثابت‌گیری مقدار بالاتری نسبت به طرح‌های دگر کسر کردن. همچنین با افزایش مقدار در حدود 0.8 درصد در شرایط بسیار، حدود 9 درصد افزایش می‌پذیرد و در دو نیز کاهشگر آب پیشرفته حاصل شده است. و با کاهش دمای حرارت در درجه حرارت به عکس شاهد کاهش در مراقبت سختی می‌باشد.

نتایج آزمایشات نشان می‌دهد که سنگانه‌هایی که دارای مقدار زیادی ریزدان هستند، مراقبت فشاری مطلوبی نتیجه ندهد و سنگانه‌هایی که قاب مقیاس ریزدان است از مرحله مراقبت فشاری خوبی برخوردارند و لی در آنها جداشتدگی بالا رخ می‌دهد. همچنین مقدار آب با زیاد باد به کاهش کلی می‌شود. استفاده از پوژولانی امکان جداشتدگی بالا را کاهش داده و موجب بالا رفتن میزان خمیرمی‌شود.

کلیدواژه‌ها: بنن، RCC سد، زیست، طرح اختلاط

1- مقدمه

بنن غلاتکی، بزرگترین پیشرفت در ساخت سدهای بنن طی ۳۰ سال گذشته می‌باشد و باعث اجرای سریعتر و کم
ماده بین چین، به‌طور معمول، دارای مقدار مختلفی از دو نوع مخلوط‌ها می‌باشد. در سال‌های تشبیه‌ای استفاده از این برای بهبود اندازه‌گیری‌های مناسب چندیکی می‌باشد. در این نواحی، مقدار مایع مناسب برای کاهش افق‌هایی که در جهان از آب و آتش مشاهده می‌شود، کمی از مواد مایع در مصرف می‌باشد. در جهان شرقی و آسیا که از پیشگامی سابقه دارد، مقدار بالایی از مواد گازی در نقاط مختلفی به همراه با مواد آب‌زدایی می‌باشد.

در جهت اطمینان از مواد مایعی که در سطح و در گرده و غبار موجود بوده‌اند، این نوع مخلوط‌ها حاوی مواد سیمانی کمتر از 100 کیلوگرم بر متر مکعب می‌باشد. بنابراین به همین عملیات، این نوع مخلوط‌ها دارای مواردی جهان است که در آن‌ها از مواد سیمانی بهره‌برداری شده است.

در مواردی که مایع مناسب برای کاهش افق‌هایی که در جهان از آب و آتش مشاهده می‌شود، کمی از مواد مایع در مصرف می‌باشد. در جهت اطمینان از مواد مایعی که در سطح و در گرده و غبار موجود بوده‌اند، این نوع مخلوط‌ها حاوی مواد سیمانی کمتر از 100 کیلوگرم بر متر مکعب می‌باشد. بنابراین به همین عملیات، این نوع مخلوط‌ها دارای مواردی جهان است که در آن‌ها از مواد سیمانی بهره‌برداری شده است.
مانند سیمانهای تیپ ۲، سیمان پرتنلد پوزولانی و سیمان پرتنلد با سریاره کوره اقتصادی استفاده می‌شود[۱۰]. سیمان مورد استفاده در ساخت طرح‌های اخلاق‌آور از کارخانه خاص تأمین شده و در مرحله مقدماتی و اول، سیمان

مصالح مصرفی در بنگلگی شامل سیمان (سیمان پرتنلد و پوزولان)، سیمکنش (شیشه و ماسه) و مواد زیرین می‌باشد.

۳-۲ مصالح سیمانی

مصالح سیمانی مورد استفاده در بنگلگی شامل سیمکنش (شیشه و ماسه) و مواد زیرین می‌باشد. نوع و مقدار سیمان مصرفی استفاده در بنگلگی شامل سیمان پرتنلد و پوزولان می‌باشد. سیمان مصرفی بستگی به ظرفیت محیطی و خواص مورد نیاز آن دارد. در سدهای RCC جهت کم کردن اثرات ناشی از حرارت هیدرولیس سیمان، معمولاً از سیمانهای با حرارت زاپی کمتر از سیمانهای معمولی (سیمانهای تیپ ۱) مانند سیمانهای تیپ ۲، سیمان پرتنلد پوزولانی و سیمان پرتنلد با سریاره کوره اقتصادی استفاده می‌شود[۱۰].

۳-۲-۱ مصالح سیمانی

می‌توان به این سیمکنش اضافه کرد که اجرا، افزایش دیفراکتی و سایر اکثر اثرات طرفین به نتیجه‌گیری و پیشش آن‌ها در طراحی و ساخت سازه‌ها با نظر به حالت طبیعی و محیطی سیمان انجام تریم می‌شود.
دیده می‌شود که افزودنی‌های دیگر، به خصوص آب، موجب افزایش میزان پایداری مصالح می‌شود. قواعدی برای استفاده از این افزودنی‌ها در مصالح پودر بیومیکرولرنی قرار گرفته است. 

4-3-2 افزودنی

در مبحث طرح‌های اختلاف از آب نوع افزودنی تیپ B مطالعات استاندارد ASTM C618 و یک نوع افزودنی تیپ D و پیک نمونه افزودنی تیپ C494 استفاده گردیده است. [12] همچنین طرح‌هایی نیز بدون افزودنی چهت مقایسه ساخته شده که در پایان افزودنی تیپ D در طرح مثبت و نگرانی استفاده قرار گرفت.

4-3-3 مصالح درشت دانه

یکی از پارامترهای اساسی در پودر بیومیکرولرنی (M.S.A) می‌باشد. به کار بردن نانووزن از 25 میلی متر باعث می‌گردد که هم مشکل‌دوزشی و هم ورودی داشته باشد. همین طرح‌های اختلاف میلی متر در نظر گرفته شده است. در مختل‌هایی که با موادی به‌دست آمده ساخته می‌شوند مصالح شامل ماسه‌های 0-3 و 5-15 می‌باشد.

5-3-2 مصالح ریزدانه

مصالح ریزدانه با ماسه بیش از 4 سنتیگرام ریزدانه مداخله 4 سنتیگرام ریزدانه (M.S.A) می‌باشد. به کار بردن مصالح با نانووزن از 25 میلی متر قطعه می‌شود.

در تولید ماسه 5-00 در حالت شسته و نشسته برسی شد، به جهت به دست آوردن بهترین نسبت اختلاف، اقدام به انجام آزمایش و مصالح آزاد میله خوده مصالح گردد. برای نظر گرفتن پارامترهای حداکثر وزن میله خوده و کنترل پدیده جداسازی نسبت اختلاف 2 به 1 (به ترتیب در شن 20-5 و 50-25 نهایی شد. در ماسه نیز پارامترهای دیگر مربوط به پونش داشته بندی مصالح مخلوط، منجر به نسبت اختلاف 1 به 2 در حال زمان 3-0 و 5-0 گردید. نسبت‌های اختلاف کلی
مصالح نیز با در نظر گرفتن کنترل یدیده جداسازی ورن مخصوص میله خورده حداکثر، هدایت میزان خمیر لازم انتخاب گردد.

۲-۳-۶ دانه‌بندی کلی مصالح سنگی

یا توجه به میزان خمیر، در طرح‌های حاوی ۱۲۰ تا ۱۵۰ کیلوگرم بر متر مکعب مواد سیمانی درصد ماسه، بالاتر از ۱۵۰ کیلوگرم بر متر مکعب ۲۸ درصد و پایین‌تر از ۱۲۰ کیلوگرم بر متر مکعب مواد سیمانی نیز ۴۲ درصد ماسه مناسب بود. در شکل ۱ منحنی دانه بندی کلی مصالح سنگی طرح آمده است.

شکل ۱- دانه بندی کلی مصالح سنگی بنفشه

۲-۴ معیارهای طرح پیشنهادی

جهت دستیابی به طرح اخلاقه پیشنهادی می‌توان زیر در نظر گرفتند:
الف- معیار مقاومت فشاری: برای مقاومت فشاری می‌توان اصلی رساند به مقاومت فشاری ۱۵ کیلوگرم بر متر مربع درصد ماسه میزان ویژه مربوط به مولکول‌های معادل باشد.
ب- معیار کارایی: برای اینکه مخلوط کارایی مناسبی داشته و بوی‌رسوب‌هایی مطلوب بین لایه‌های مناسبی ایجاد گردد، معیار زمان ویژه می‌تواند میان‌الین ۱۴ تا ۲۴ ثانیه در نظر گرفته شده است.
ب- معیار عدم جداسازی دانه‌ها: جداسازی دانه‌ها عمدا بعلت درصد آب خیلی کم و یا خیلی زیاد، کم بودن مقدار رزدانه و عدم تبادل بین دانه‌های با اندازه‌های مختلف به وجود می‌آید. در طرح پیشنهاد گذارشگردی دانه‌های دیده تا ضمین حفظ یکنواختی، مقاومت و درمان مطلوب تبیجه شود.

۳- یافته‌ها

۳-۱ مرحله اول

در این مرحله اقدام به ساخت طرح‌های تکمیلی شد که نمونه‌ها آن طرح اخلاقه‌ها در جدول ۱ آورده شده است.
158

همچنین نتایج آزمایشات بر روی نمونه‌های ساخت شده به شرح زیر می‌باشند:

3-1-1 

در شکل 2 رابطه بین نسبت آب به سیمان و مقاومت فشاری 78، 90 و 180 روزه کلیه طرح‌های این مرحله نشان داده شده است.

جدول 1: نمونه‌ای از طرح اخلاط‌های ساخت شده

<table>
<thead>
<tr>
<th>شماره</th>
<th>طرح</th>
<th>نام طرح</th>
<th>مقدار فشاری kg/cm²</th>
<th>زمان روشه</th>
<th>نسبت آب به سیمان</th>
<th>میزان ماده افزوده</th>
<th>افزوده</th>
<th>افزوده</th>
<th>افزوده</th>
<th>افزوده</th>
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</thead>
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<td>1,670</td>
<td>180</td>
<td>90</td>
<td>28</td>
<td>0.07</td>
<td>63</td>
<td>7</td>
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<td>411</td>
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<tr>
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<td>1,850</td>
<td>90</td>
<td>72</td>
<td>8</td>
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<td>858</td>
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<td>417</td>
<td>857</td>
<td>364</td>
<td>545</td>
</tr>
</tbody>
</table>

ازمایشات نشان داد که مقاومت فشاری طرح‌های ساخت شده با سنگدانه‌های نسبت طبیعی مطلوب نیست.

همچنین مقدار آب مناسب برای این نوع سنگدانه‌ها زیاد است. علت این امر یافتن مناسب مواد رمزانه خصوصاً دراه
3-1-3 موارد افزودنی

در طرح اختلاط سد بتن غلتکی زیردان از موارد افزودنی Conplast RP264M و ChrysoPlast CER استفاده شد که طرح‌های جلوی افزودنی منطبق (نوع D مطابق استاندارد ASTM C494) حدود 12 درصد مقاومت فشاری بالاتری نسبت به طرح‌های دیگر کسب کردند. در شکل 2 رابطه بین میزان مصالح سیمان و مقاومت فشاری 180 روزه طرح‌ها با افزودنی‌های مختلف نشان داده شده است. همچنین در مورد مخلوط‌های کم سیمان، کاربرد موارد افزودنی روی ساز باعث کاهش در مقدار آب (یا زمان Vebe ثابت) می‌شود ولی با وجود کاهش در نسبت آب به سیمان، مقاومت فشاری به طور محسوس افزایش نمی‌یابد. لذا در این حالت دلیل عده برای استفاده از روی‌های ساز، می‌تواند افزایش روی‌های جهت نیل به زمان Vebe مناسب و بهبود یک‌نواختی بتن باشد.
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شکل ۴- مقاومت فشاری طرح‌های با افزودنی‌های متفاوت

با توجه به اینکه حداکثر میزان توشیب‌های پلیمر این مواد حدود ۸۰ درصد و زمان میزان‌های میان‌برد، علاوه بر دو دوچرخه، سه طرح با دوچرخه ۸ نیز کار شد که برای مقایسه آنها با یکدیگر، ۴ طرح که زمان‌های ویژه

اند. ترتیب که توزیعی به یکدیگر متفاوت است بایستی با توجه به طرح که زمان ویژه دارد، در جدول ۴ آورده شده است.

جدول ۳: طرح‌های اخلاق با دوره مختلف مواد افزودنی

<table>
<thead>
<tr>
<th>نام طرح</th>
<th>افزودنی</th>
<th>زمان ویژه (ثانیه)</th>
<th>میزان دانشه (kg/m3)</th>
<th>مقاومت فشاری (kg/cm2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>R160-30-11</td>
<td>R160-30-12</td>
<td>0.6 Chrysoplast CER</td>
<td>16 0.742 46.0 84.5 2508 15</td>
<td></td>
</tr>
<tr>
<td>R160-30-12</td>
<td>R160-30-13</td>
<td>0.6 Chrysoplast CER</td>
<td>20 0.750 46.2 97.6 2501 10~15</td>
<td></td>
</tr>
<tr>
<td>R160-30-13</td>
<td>R160-30-14</td>
<td>0.8 Chrysoplast CER</td>
<td>18 0.728 45.8 106.0 2507 10~12</td>
<td></td>
</tr>
<tr>
<td>R160-30-14</td>
<td>R160-30-15</td>
<td>0.8 Chrysoplast CER</td>
<td>45 0.714 45.5 108.9 2517 27~30</td>
<td></td>
</tr>
</tbody>
</table>

چنانچه مشاهده می‌شود با افزایش دوچرخه ۱۲ و ۱۳ در Chrysoplast از ۸ به ۱۸ درصد (مقاومت طرح‌های ۱۲ و ۱۳) در شرایط یکسان، حدود ۹ درصد افزایش مقاومت و ۳ درصد نیز کاهشی آپیشتر حاصل شده است. ولی با

افزایش دوچرخه نیز کاهش گسترده در مقاومت ویژه، نتیجه‌گیری که توجه قابل توجه مقاومت یکسان در دو طرح Conplast بر عهکس شاده کاهش در مقاومت ویژه در نتیجه، توانایی افزایش مقاومت با الار بی‌شک دارد. در شرایط یکسان ۱۲ با زمان‌های ویژه مقاومت است که می‌توانیم مقاومت بالاتر افزودنی Chrysoplast در شرایط یکسان می‌باشد که این مسئله در طرح‌های ۱۲ و ۱۸ درصد مواد مزکر نیز به وضوح دیده شد.
۳-۱-۵ مقادیر مختلف پوزولان

با تابی بودن مجموع مواد سیمانی، افزایش پوزولان کارآی را با بالایه برده. بدین منظور علاوه بر طرح‌های حاوی ۳۰ درصد پوزولان، طرح‌هایی نیز با ۳۵ و ۴۰ درصد پوزولان ساخته شد. رابطه بین مواد سیمانی با مقاومت فشاری (شکل ۵) و نسبت آب به سیمان و مواد سیمانی با مقاومت (شکل ۶) بیانگر مناسبی بودن عملکرد طرح‌های حاوی ۳۰ درصد پوزولان است.

شکل ۵ - رابطه بین مواد سیمانی با مقاومت فشاری

شکل ۶ - نسبت آب به سیمان و مواد سیمانی با مقاومت فشاری

۳-۱-۶ مقاومت کششی غیرمستقیم

در زمان ساخت نمونه‌های استوانه‌ای جهت انجام آزمایشات مقاومت فشاری نمونه‌هایی نیز به منظور بررسی مقاومت کششی طرح‌های اختلاف نیز تیزگرفته شد که مقادیر مقاومت کششی مربوطی به نمونه‌هایی با میزان مصالح سیمانی بیشتر از ۱۵۰ کیلوگرم بر متر مکعب در سینه ۷، ۸، ۲۸، ۹۰ و ۱۸۰ روز آنها در شکل ۷ نشان داده شده است. نتایج نشان دهنده این است که مقاومت کششی غیرمستقیم در طرح‌های مختلف حدود ۱۰ تا ۱۱ درصد
مقاومت فشاری می‌باشد.

**شکل 7 - مقاومت کنشی غیر مستقیم طرح‌های اکلاته**

2-3 مرحله دوم

پس از اخذ نتایج طرح‌های مرحله اول ساخت طرح‌های نهایی با زمان ویژه واقعی 14 تا 24 ثانیه آغاز شد. همچنین از طرح‌های مرحله اول برای مقایسه نمونه‌ها و نشانی، افزودنی‌ها و درصد‌های مختلف پوزولان استفاده شده است. در شکل 8 ضریب بارزه نمونه‌های مختلف ارائه شده است.

**شکل 8 - ضریب بارزه طرح‌های مرحله دوم**

همچنین در شکل 9 رابطه بین مقدار مصالح سیمانی با توجه به مقاومت 7 و 28 روزه طرح‌ها آورده شده است.

**شکل 9 - رابطه نسبت آب به سیمان و مقاومت فشاری**
### 4- نتیجه گیری و انتخاب طرح اختجاع بهینه

1- با توجه به نتایج تحقیق برای مشخصه طرح (125 کیلوگرم بر متر سنتیمتر مربع در سن 180 روز) طرح اختجاع با مشخصات زیر انتخاب گردید.

<table>
<thead>
<tr>
<th>میزان</th>
<th>مصالح سیمان</th>
<th>حجم خشخاش سیمان</th>
<th>حجم خشخاش سیمان</th>
<th>نسبت</th>
<th>وزن</th>
<th>مقادیر طرحی kg/m²</th>
</tr>
</thead>
<tbody>
<tr>
<td>میزان</td>
<td>kg/m³</td>
<td>نسبت حجمی (درصد)</td>
<td>%</td>
<td>تنها</td>
<td>وزنی</td>
<td>7وزنی</td>
</tr>
<tr>
<td>میزان</td>
<td>120</td>
<td>120</td>
<td>0.99</td>
<td>0.5</td>
<td>1.5</td>
<td>15</td>
</tr>
</tbody>
</table>

### تکرک و قدردانی

بِدینسیلیه از هنگام محسن جعفریگلیو و تکسیم‌های آزمایشگاه بین شرکت چهان کوثر (پیمانکار سد زیردران) که در کلیه مراحل ساخت مخلوط‌های بین و انجام آزمایش‌ها، صمیمانی همکاری فراوانی و الهام نیافتند.
مراجع

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CD11
Nano Technology
INVESTIGATION OF MECHANICAL AND PHYSICAL PROPERTIES OF MORTARS CONTAINING SILICA FUME AND NANO-SiO$_2$

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ABSTRACT
It has been found that physical properties of concrete, particularly strength and permeability significantly depend on its pore structure. Ultra fine particles of nano-SiO$_2$ fill the voids of CSH structure and provide more homogenous distribution of hydrated products. This effect of nano-SiO$_2$ enhances the durability of cement composites as well as the strength. In this paper, influence of nano-SiO$_2$ on different properties of cement mortar was investigated in comparison with silica fume (SF) as a well-known active pozzolan. Different amounts of nano-SiO$_2$ (0, 1%, 3%, 5%, 7% and 9%) were incorporated into the ordinary cement mortar. Mechanical properties, shrinkage, water absorption of the specimens were determined. Results showed that the optimal content of Nano-SiO$_2$ in plain cement mortar was around 7%. Nano-SiO$_2$ particles were more effective in developing higher mechanical strength and lower water absorption than that of SF. Yet the mortar containing nano-SiO$_2$ experienced higher shrinkage than that of SF mortar.

Keywords: cement mortar, mechanical properties, nano-SiO$_2$, silica fume, shrinkage

1. INTRODUCTION
High strength concrete and mortar with high strength and durability properties offer many advantages. They have been gradually replacing normal strength concrete due to their improved mechanical characteristics and low permeability. With such outstanding characteristics they can be utilized in structure, exposed to severe loading or influenced by environmental conditions, for instance large bridges and offshore constructions [1,2].

Silica fume has been widely used as a supplementary cementing material for producing high performance concrete. It is used to enhance the strength and durability of concrete. It has been reported that use of SF as a cement replacement increased sulfate and acid resistance and decreased chloride permeability of concrete. When SF is added to cement/concrete, it acts as a filler to fill the gaps between cement particles resulting in finer pore structure. Also more CSH gel can be formed in SF concrete due to the reaction that occurs between the silica in SF...
and the Ca(OH)\textsubscript{2} in hydrating cement (pozzolanic reaction)[3,4]. Recently with the help of advanced nanotechnology developments, nano-SiO\textsubscript{2} with finer particles size and higher pozzolanic activity has been introduced. Studies have shown that incorporating nano-SiO\textsubscript{2} into cement based materials improved mechanical properties of the products. Qing Ye [5] reported that nano-SiO\textsubscript{2} improved the bond strength of paste-aggregate interface. Additional studies have also concluded that pozzolanic activity of nano-SiO\textsubscript{2} was much greater than that of silica fume [6]. The abrasion resistance of concrete containing nano-SiO\textsubscript{2} was studied by Hui Li [7]. He suggested that nano-SiO\textsubscript{2} was valuable for enhancing abrasion resistance of pavement. Gengig Li [8] showed that nano-SiO\textsubscript{2} added to high-volume fly ash high-strength concrete could improve short and long term strengths. K. Lin [9] reported that nano-SiO\textsubscript{2} particles could potentially improve the negative influences caused by sewage sludge ash (SSA) replacement mortar. It has been found that when nano-SiO\textsubscript{2} particles are uniformly dispersed in cement paste they will accelerate cement hydration due to their high activity [10]. Owing to the unique properties of nano-SiO\textsubscript{2} it seems that it has a potential to be utilized in production of high strength concrete. Hence more assessments are necessary to ensure usage possibility of nano-SiO\textsubscript{2} in cement based materials. Accordingly, in the present experimental study, Nano-SiO\textsubscript{2} particles were incorporated into ordinary cement mortar and compressive strength, flexural behavior, water absorption and shrinkage of these composites were investigated. Results were compared with mortar containing 10% silica fume.

2. MATERIALS AND METHOD

**Materials:** In this study, ordinary Portland cement type I, silica fume, nano-SiO\textsubscript{2} particles and tap water were used. SF used in this experiment contained 91.1% SiO\textsubscript{2} with average particles size of 7.38 µm. The chemical compositions of SF and cement were analyzed using an X-ray microprobe analyzer and listed in Table 1. In order to achieve the desire fluidity and better dispersion of nano particles, a polycarboxylate ether based superplastizer was incorporated into all mixes. Natural river sand was used with the fraction of sand passing through 1.18 mm sieve and retaining on 0.2mm. The specific gravity of sand was 2.51 gr/cm\textsuperscript{3}. Basic material properties of nano-SiO\textsubscript{2} are given in Table 2.

<table>
<thead>
<tr>
<th>Chemical compositions (%):</th>
<th>O.P.C.</th>
<th>S.F.</th>
</tr>
</thead>
<tbody>
<tr>
<td>L.O.I</td>
<td>SO\textsubscript{4}</td>
<td>MgO</td>
</tr>
<tr>
<td>1.35</td>
<td>4.8</td>
<td>61.5</td>
</tr>
<tr>
<td>2.1</td>
<td>0.45</td>
<td>.6</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Item</th>
<th>Diameter (nm)</th>
<th>PH value</th>
<th>Composition (mass%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Target</td>
<td>50</td>
<td>10</td>
<td>SiO\textsubscript{2}(30%) + H\textsubscript{2}O(70%)</td>
</tr>
</tbody>
</table>
Table 4: Mix proportion of the specimens

<table>
<thead>
<tr>
<th>No</th>
<th>Sand/Binder</th>
<th>Water/Binder</th>
<th>% Content (by weight)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>O.P.C.</td>
</tr>
<tr>
<td>NS0</td>
<td>2.75</td>
<td>0.5</td>
<td>100</td>
</tr>
<tr>
<td>NS1</td>
<td>2.75</td>
<td>0.5</td>
<td>99</td>
</tr>
<tr>
<td>NS3</td>
<td>2.75</td>
<td>0.5</td>
<td>97</td>
</tr>
<tr>
<td>NS5</td>
<td>2.75</td>
<td>0.5</td>
<td>95</td>
</tr>
<tr>
<td>NS7</td>
<td>2.75</td>
<td>0.5</td>
<td>93</td>
</tr>
<tr>
<td>NS9</td>
<td>2.75</td>
<td>0.5</td>
<td>91</td>
</tr>
<tr>
<td>SF10</td>
<td>2.75</td>
<td>0.5</td>
<td>90</td>
</tr>
</tbody>
</table>

Methods: Cement mortars containing SF or nano-SiO₂ were prepared with the same flowing capacity through the adjustment of superplastisizer (Table 3). The water/binder and sand/binder ratios of all mixtures were 0.5 and 2.75 respectively, where the binder weight is the total weight of cement, SF or nano-SiO₂. The amount of SF replacement in mortar (SF mortar) was fixed at 10% which is an acceptable range and is used most often. In order to achieve the desired properties, it is essential to disperse nano particles uniformly. Accordingly, mixing was carried out in a rotary mixer as follows:

1. The nano-SiO₂ particles were stirred with 90% of mixing water at high speed and for about 1 min.
2. The Cement and SF were premixed for 30 s. Then dry mixed cement and SF were added to the mixture. After adding, the mixer was allowed to run for 1 min at medium speed.
3. The sand was gradually added at 30s while the mixer was running at medium speed.
4. The superplastisizer and remaining water were added and stirred at high speed for 30s.
5. The mixture was allowed to rest for 90s. Then mixing was continued for 2 min at high speed.

After mixing, the samples were cast into the 50×50×50 mm cubes for compressive and water absorption tests and 50×50×200 beams for flexural and shrinkage tests. The compressive samples were placed in two layers. Each layer was tamped 32 times in about 10s using a hard rubber mallet following the procedure of ASTM C-109 [11]. The flexural samples also were placed in two layers and each layer was tamped 12 times in 4 rounds as per ASTM C-348 [12]. After 24 hours the specimens were removed from the molds and cured in water at 23±2 °C for 7, 28, 60 and 90 days. The specimens were tested using a hydraulic testing machine under load control at 1350 N/s for compressive test and 44N/s for flexural test. The absorption test was carried out on two 50 mm cubes. Saturated surface dry specimens were kept in an oven at 110°C for 72 h. After measuring the initial weight, specimens were immersed in water for 72h. Then the final weight was measured and the absorption was reported to assess the mortar permeability of mortar in this study.
3. RESULTS AND DISCUSSION

**Compressive strength:** Figure 1 shows the variation in the compressive strength of mortars. It can be seen that the compressive strength of cement mortar with nano-SiO₂ is higher than that of plain cement mortar and gradually increases with an increase in the amount of nano-SiO₂. It is obvious that increase in the nano-SiO₂ content beyond 7% did not change the compressive strength significantly. It is found that large amounts of nano-SiO₂ decrease the compressive strength of the composites instead of improving it. Because when the content of nano-SiO₂ is large, nano particles are difficult to disperse uniformly. Therefore, they create a weak zone in the form of voids, consequently the homogeneous hydrated microstructure can not be formed and a lower strength will be probable [13]. Also it can be observed that the compressive strength of the specimens containing 5%, 7% and 9% nano-SiO₂ (Mixtures NS5, NS7, NS9) are higher than that of the SF mortar. This indicates that nano-SiO₂ has a higher pozzolanic activity and is more valuable in reinforcement of mortar than that of SF.

![Figure 1. Compressive strength of different cement mortars](image)

**Flexural strength:** Flexural strength test results are shown in Figure 2. Results show that nano-SiO₂ is more effective in developing flexural strength than that of SF. From the results it can be concluded that the optimum nano-SiO₂ content in ordinary cement mortar ranges between 5% and 7%.

Two fundamental mechanisms can be deduced for strength enhancement by nano-SiO₂:

1) Strength enhancement by matrix densification and paste-aggregate interfacial zone refinement

2) Strength enhancement by reduction in the content of Ca(OH)₂.

The first strengthening mechanism is called the filler effect. The micro filling effect of nano-SiO₂ is one of the important factors for the development of dense concrete/mortar with very high strength, because small amount of air content significantly decreases the strength of the mortar. Nano-SiO₂ particles, due to their
small size act as a filler to fill into the interstitial spaces inside the skeleton of hardened microstructure of cement paste to increase its density as well as the strength [14]. The filling effect of nano-SiO$_2$ is also valuable for generating strong transition zone. It has been reported that the microstructure of the transition zone between cement paste and aggregates strongly influences the strength and durability of concrete [15]. Nano-SiO$_2$ particles reduce the wall effect in the transition zone between the paste and the aggregate and strengthen this weaker zone due to the higher bond between those two phases. It should be mentioned that the silica fume causes reduction in the volume of large pores and increases the mortar strength too, but as the size ratio between filler and the aggregates is one of the main parameters that strongly affects the strengthening caused by filling effect, and thanks to the high size ratio between nano-SiO$_2$ and cement grains, the filling effect of nano-SiO$_2$ particles is more obvious. The second strengthening mechanism is the pozzolanic activity. Pozzolans are defined as siliceous or siliceous and aluminous materials that in themselves possess little or no cementing property but in finely dispersed form in the presence of moisture chemically react with calcium hydroxide at ordinary temperature to form compound possessing cementitious properties. Two major products of cement hydration are calcium silicate hydrate (CSH) and calcium hydroxide (CH) respectively. Calcium silicate hydrate which is produced by hydration of C$_3$S and C$_2$S plays a vital role in mechanical characteristics of cement paste. Whereas calcium hydrate which is also formed by hydration of cement does not have any cementing property. It contains about 20-25% of the volume of the hydration products. Calcium hydrates due to their morphology are relatively weak and brittle. Cracks can easily propagate through regions populated by them, especially at the aggregate cement paste matrix interface [16]. Nano-SiO$_2$ reacts with Calcium hydrates formed during hydration of cement rapidly and produces calcium silicate hydrate with cementitious properties which is beneficial for enhancement of strength in concrete/mortar.

![Figure 2. Flexural strength of different cement mortars](image-url)
Both nano-SiO$_2$ and SF belong to pozzolanic materials, however results showed that the pozzolanic activity of nano-SiO$_2$ was much greater than that of SF. A possible reason for this observation could be nucleation effect. Nano-SiO$_2$ due to its high specific surface serves additional nucleation sites for precipitation of the hydration products whereby chemical reactions are accelerated [17]. Moreover it has been suggested that the surface of pozzolan can adsorb many Ca$^{2+}$ ions and that lowering of the concentration of the calcium ions accelerates the rate of dissolution of C$_3$S that increases the rate of hydration [18]. Owing to the higher specific surface of nano particles they adsorb more Ca$^{2+}$ ions and accelerate the rate of hydration more effectively.

Shrinkage: Shrinkage is a common phenomenon generally encountered in almost every cementitious product due to contraction of total mass upon loss of moisture. It is sometimes accompanied by development of cracks specially in such members whose surface area to volume ratio is large [19,20]. These cracks serve as conduits for salt and water. The saline solution comes in contact with reinforcing steel and promotes corrosion. Corrosion causes expansion of steel and inevitably pop-outs occur in the concrete cover, Thereby reducing the strength and service life of the concrete [21]. In view of the importance of the volume changes due to shrinkage this section is devoted to the study of the influence of nano-SiO$_2$ on the drying shrinkage of ordinary mortars. Prismatic specimens with 50×50×200 mm dimensions were prepared. The first measurement was taken using a length comparator with a precision of 2µm after 24 h of mixing, while the rest of measurements were taken at different ages of 3, 7, 14, 21, 28, 35, 42 days. The specimens were cured in the laboratory environment. The average temperature in the laboratory was 27±3 °C. The shrinkage behavior of mortars containing different amounts of nano-SiO$_2$ in comparison with SF mortar is presented in Figure 3. Results showed that both SF mortar and mortar containing nano-SiO$_2$ experienced higher shrinkage than that of ordinary cement mortar. Moreover it can be seen that the drying shrinkage of mortars with nano-SiO$_2$ is higher than that of SF mortar.
and increases with increasing nano-SiO$_2$ content. The increase in the drying shrinkage of mortar containing nano-SiO$_2$ might be due mainly to refinement of pore size and increase of mesopores volume which is directly related with the shrinkage due to self desiccation. Moreover nano-SiO$_2$ particles act as an Activator to accelerate cement hydration, therefore the degree of hydration increases as the amount of nano-SiO$_2$ increases and the autogenous shrinkage related to chemical shrinkage also increases [22].

**Water absorption:** The absorption characteristics indirectly represent the porosity through an understanding of the permeable pore volume and its connectivity [23]. In order to investigate the effect of nano-SiO$_2$ particles on cement mortar permeability, water absorption test was carried out at the curing age of 28 days. The absorption values of mortars are listed in Table 4. It is clear that presence of pozzolanic material in cement mortar decreased the water absorption value. Nano-SiO$_2$ was more effective in reduction of permeability than that of SF. The increase of impermeability caused by nano-SiO$_2$ can be attributed to two concomitant phenomena:

i. Nano-SiO$_2$ particles generate a large number of nucleation sites for the hydration products and induce a more homogenous distribution of CSH and hence less pore structure [24].

ii. Nano-SiO$_2$ particles block the passages connecting capillary pores and water channels in cement paste [25].

### Table 5: Water absorption of mortars

<table>
<thead>
<tr>
<th>Mixture</th>
<th>Water absorption (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>NS0</td>
<td>6.12</td>
</tr>
<tr>
<td>NS3</td>
<td>5.11</td>
</tr>
<tr>
<td>NS5</td>
<td>4.35</td>
</tr>
<tr>
<td>NS7</td>
<td>4.23</td>
</tr>
<tr>
<td>SF10</td>
<td>5.18</td>
</tr>
</tbody>
</table>

4. CONCLUSIONS

An experimental study was carried out to investigate the effect of nano-SiO$_2$ on the physical and mechanical properties of mortar. Based on the experimental results, following conclusions can be drawn:

1. Noticeable increase was observed in compressive and flexural strength of ordinary cement mortar upon adding nano-SiO$_2$, however high amounts of nano-SiO$_2$ had a negative effect on mechanical properties especially flexural strength. From the results it can be concluded that the optimum nano-SiO$_2$ content in ordinary cement mortar ranges between 5% and 7%.

2. The effect of nano-SiO$_2$ on drying shrinkage of mortar was significant. The mortar samples containing nano-SiO$_2$ experienced higher values of drying shrinkage compared to reference mortars. This effect was more prominent for larger amounts of nano-SiO$_2$.

3. The absorption characteristics which indirectly reflect the porosity showed that nano-SiO$_2$ particles decreased the water absorption of cement composites by
pore filling and pozzolanic effects. Also it was observed that nano-SiO\textsubscript{2} particles were more effective in the reduction of permeability than that of SF.

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EFFECT OF POLYPROPYLENE FIBERS ON MECHANICAL AND PHYSICAL PROPERTIES OF MORTARS CONTAINING NANO-SiO$_2$

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ABSTRACT
It has been demonstrated that the fiber-matrix bond strongly affects the ability of fibers to stabilize crack propagation in the matrix. As the bond between fiber and matrix is mainly mechanical, it seems that incorporating nano-SiO$_2$ (NS) into fiber reinforced cement composites provides better bond with matrix through pore refinement and better distribution of the hydrated products. Hence in this paper an effort was made to study the effect of polypropylene (PP) fibers on mechanical properties and shrinkage of mortar incorporating NS. Three fiber volume fractions, 0.1%, 0.3% and 0.5% were considered. Compressive and flexural strength, water absorption and shrinkage of mortars were reported. Results showed that NS improved mechanical and water absorption characteristics of mortars significantly. It has been observed that the addition of NS fairly enhanced the fibers effectiveness in improving the mechanical strength of mortars.

Keywords: mortar, nano-SiO$_2$, compressive strength, flexural strength, shrinkage

1. INTRODUCTION
Nano-particles possess unique physical and chemical properties that can improve the function and properties of many types of materials. Among the nano-particles, nano silica have been used to improve the properties of cement based materials and some efforts on excellent mechanical properties and microstructure of cement composite with NS have been also reported. Studies have shown that application of NS into the production of mortar and concrete can lead to improvement in compressive strength, flexural behavior and abrasion resistance [1-4]. Therefore NS can be applied in production of high performance concrete (HPC) which has been gradually replacing normal strength concrete. As the rate of pozzolanic reaction is proportional to the amount of surface available for reaction and owing to the high specific surface of nano particles, they possess high pozzolanic activity that consume calcium hydroxide (CH) which arrays in the interfacial transition zone between hardened cement paste and aggregates and produce hydrated calcium silicate (CSH) which enhances the strength of cement paste [5]. In addition, due to nano scale size of particles, NS can fill the ultra fine pores in cement matrix. This physical effect of the finer grains leads to reduction in porosity of transition zone in
the fresh concrete. This mechanism strengthens the bond between the matrix and the aggregates and improves the cement microstructure and properties. Furthermore, it has been found that when the small particles of NS uniformly disperse in the paste due to their high activity, they generate a large number of nucleation sites for the precipitation of the hydration products which accelerates cement hydration [6]. Polypropylene fibers have been widely used for the reinforcement of cementitious materials to improve the toughness and energy absorption capability of matrix [7]. They were found to be extremely effective in reducing free plastic shrinkage, in retarding first crack appearance and in controlling crack development [8]. Although effectiveness of PP fibers in shrinkage cracking, impact resistance and ductility of cement matrices has been proved by many researchers, effect of PP fibers on compressive and flexural strength is not quit clear [9]. Studies have shown that there can be little or no chemical adhesion between the fiber and matrix as a result of their chemical inertness [10]. It seems that smooth surface of PP fibers intensifies this effect. Moreover, it has been suggested that the presence of PP fibers in cement paste results in the formation of a water film at the interface of fiber and matrix called wall effect. Due to greater mobility of calcium ions in a water environment, portlandite (calcium hydroxide) macro crystals can easily grow and make the transition zone more pores [11]. This phenomenon has a negative impact on the bond between fiber and matrix. It is clear that in order to utilize the maximum strength of the fiber and improve the composite properties, it is essential to enhance the interfacial bond of PP fibers. It seems that the physical and chemical effects of nano particles can be useful in reduction of wall effect between fiber and matrix. Accordingly, present study focuses on the effect of NS on mechanical and physical properties of fiber reinforced cement composite mortar.

2. EXPERIMENTAL PROCEDURE

Materials and mix proportions: The cement used in all mortar mixes was ordinary Portland cement which corresponds to ASTM type 1. The chemical analysis of Portland cement is shown in Table 1. NS in liquid form with the average particles size of 50 nm was used in this study. In order to achieve desire fluidity and better dispersion of nano particles, a polycarboxylate ether based superplastisizer was utilized. The content of superplastisizer was adjusted for each mixture to keep constant the fluidity of mortars. Ottawa sand conforming to ASTM-C778 [12] was used for mortar preparation. Table 2 reveals the physical properties of PP fibers. All specimens were fabricated with the water/binder and sand/binder ratios of 0.5 and 2.75 respectively. The weight of binder was considered equal to the sum of the weight of cement and NS.

In the initial stage of the present study a total of 6 batches of mortars were prepared to find the optimum amount of NS in ordinary cement mortar (Table 3). According to the initial stage results, in the second stage 0.1%, 0.3%, and 0.5% PP fiber (Compared with the total mortar volume) were added to the ordinary and the optimum mixtures selected in the initial stage with the purpose of evaluating the influence of the PP fibers on the strength and shrinkage properties (Table 4). In all
the tests, specimens without fiber were considered reference materials.

<table>
<thead>
<tr>
<th>Table 6: Chemical compositions of cement</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Items</td>
<td>Chemical compositions (%)</td>
</tr>
<tr>
<td>SiO₂</td>
<td>21.5</td>
</tr>
<tr>
<td>AL₂O₃</td>
<td>3.68</td>
</tr>
<tr>
<td>Fe₂O₃</td>
<td>2.76</td>
</tr>
<tr>
<td>CaO</td>
<td>61.5</td>
</tr>
<tr>
<td>MgO</td>
<td>4.8</td>
</tr>
<tr>
<td>SO₃</td>
<td>-</td>
</tr>
<tr>
<td>L.O.I</td>
<td>1.35</td>
</tr>
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</table>

<table>
<thead>
<tr>
<th>Table 2: Properties of polypropylene fiber</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Property</td>
<td>Polypropylene</td>
</tr>
<tr>
<td>Unit weight (gr/cm³)</td>
<td>0.9-0.91</td>
</tr>
<tr>
<td>Reaction with water</td>
<td>Hydrophobic</td>
</tr>
<tr>
<td>Tensile strength (MPa)</td>
<td>300-400</td>
</tr>
<tr>
<td>Elongation at break (%)</td>
<td>100-600</td>
</tr>
<tr>
<td>Melting point</td>
<td>175</td>
</tr>
<tr>
<td>Thermal conductivity</td>
<td>0.12</td>
</tr>
<tr>
<td>(W/m/K)</td>
<td></td>
</tr>
<tr>
<td>Length (mm)</td>
<td>6</td>
</tr>
</tbody>
</table>

**Test method:** In order to achieve desire properties, it is essential to disperse NS and PP fibers uniformly. Accordingly, mixing was carried out in a rotary mixer as follows:

1. The NS particles were stirred with 90% of mixing water at high speed and for about 1 min.
2. The specified amount of fiber was added and mixed for 2 min at medium speed.
3. The cement was added and the mixer was allowed to run for 1 min at medium speed.
4. The sand was gradually added at 30s while the mixer was running at medium speed.
5. The superplastisizer and remaining water were added and stirred at high speed for 30s.
6. The mixture was allowed to rest for 90s. Then mixing was continued for 2 min at high speed.

Fresh mortar was cast into 50×50×50 mm cubes for compressive and water absorption tests and 50×50×200 steel molds for flexural and shrinkage tests. The specimens were tamped using a hard mallet to decrease the amount of the air bulbs. After the feeding operation, each of the specimens was allowed to stand for 24 h. Then the specimens were demolded and kept in water at 23±3 °C until they were tested.

Compressive strength test was conducted in accordance with ASTM-C109 [13] using a hydraulic testing machine under load control at 1350N/s. The three-point
(i.e. center-point) loading flexural test was carried out with the span of 180mm and at a loading rate of 44N/s. The flexural and compressive strength were determined at 7, 28, 60 and 90 days of curing. Shrinkage test samples were cured in the laboratory environment at 27±3 °C. Changes in the length of the mortar samples were measured using a length comparator with the precision of 0.002mm. The first measurement was taken after 24h of mixing, while the rest of the measurements were taken at the ages of 3, 7, 14, 21, 28, 35 and 42 days. The water absorption test was carried out at 28 days as follows: Saturated surface dry specimens were kept in an oven at 110 °C for 72 h. After measuring the initial weight, specimens were immersed in water for 72h. Then the final weight was measured and the final absorption was reported to assess the mortar permeability.

Table 3: Mix proportion of the specimens (initial stage)

<table>
<thead>
<tr>
<th>Batch No</th>
<th>Sand/Binder</th>
<th>Water/Binder</th>
<th>% Content (by weight)</th>
<th>O.P.C.</th>
<th>N.S.</th>
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<tr>
<td>NS0</td>
<td>2.75</td>
<td>0.5</td>
<td>100</td>
<td>100</td>
<td>0</td>
</tr>
<tr>
<td>NS1</td>
<td>2.75</td>
<td>0.5</td>
<td>99</td>
<td>99</td>
<td>1</td>
</tr>
<tr>
<td>NS3</td>
<td>2.75</td>
<td>0.5</td>
<td>97</td>
<td>97</td>
<td>3</td>
</tr>
<tr>
<td>NS5</td>
<td>2.75</td>
<td>0.5</td>
<td>95</td>
<td>95</td>
<td>5</td>
</tr>
<tr>
<td>NS7</td>
<td>2.75</td>
<td>0.5</td>
<td>93</td>
<td>93</td>
<td>7</td>
</tr>
<tr>
<td>NS9</td>
<td>2.75</td>
<td>0.5</td>
<td>91</td>
<td>91</td>
<td>9</td>
</tr>
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</table>

Table 4: Mix proportion of the specimens (second stage)

<table>
<thead>
<tr>
<th>No</th>
<th>S/B</th>
<th>S/B</th>
<th>%P</th>
<th>%P</th>
<th>S/B</th>
<th>S/B</th>
<th>%P</th>
<th>%P</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>2.75</td>
<td>0.5</td>
<td>100</td>
<td>0</td>
<td>5</td>
<td>2.75</td>
<td>0.5</td>
<td>100</td>
</tr>
<tr>
<td>2</td>
<td>2.75</td>
<td>0.5</td>
<td>93</td>
<td></td>
<td>6</td>
<td>2.75</td>
<td>0.5</td>
<td>93</td>
</tr>
<tr>
<td>3</td>
<td>2.75</td>
<td>0.5</td>
<td>100</td>
<td>0</td>
<td>7</td>
<td>2.75</td>
<td>0.5</td>
<td>100</td>
</tr>
<tr>
<td>4</td>
<td>2.75</td>
<td>0.5</td>
<td>93</td>
<td>0</td>
<td>8</td>
<td>2.75</td>
<td>0.5</td>
<td>93</td>
</tr>
</tbody>
</table>

\[\text{a: Sand} \quad \text{b: Binder (Cement +Nano-SiO}_2\text{)} \quad \text{c: Water}\]

3. EXPERIMENTAL INVESTIGATION AND RESULTS

Compressive strength: The compressive strength of cement mortars with different dosages of NS at four ages are given in figure 1. It is clear that the compressive strength of ordinary cement mortar increases with an increase in the amount of NS. It can be seen that increasing the NS content from 7% to 9% didn’t improve the compressive strength significantly. It seems that a large amount of NS even decreases the strength. According to Hui Li [14] homogeneous hydrated microstructure which is essential for the strength of cement matrix can not be formed because nano particles can not be well dispersed. Strength enhancement of NS can be attributed to reduction in the content of Ca(OH)\textsubscript{2} which does not have any cementing property and production of hydrated calcium silicate (CSH) that plays a vital role in mechanical characteristics of cement paste [15,16]. NS
particles also generate a large number of nucleation sites for the cement hydration products making the paste microstructure more homogenous and improve its strength and permeability [17]. In the view of the results above, cement mortar with substitution of cement by 7% NS was selected as the optimum mixture. Figure 2 shows the compressive strength of fiber reinforced mortars. Results appearing in this Figure indicate that PP fibers induce a slight modification in the compressive strength. The compressive strength of mortar increased gradually at first with the increase of fiber content but then decreased with the further increasing of fiber content. Almost all the specimens containing 0.1% pp fiber by volume exhibited an increase in compressive strength compared to the target specimens. A possible reason for this may be that PP fibers act as crack arresters.

![Figure 1. Compressive strength of ordinary mortars at different contents of nano-SiO2](image1)

![Figure 2. Compressive strength of different mortar mixtures according to the PP content](image2)

The uniformly distributed PP fibers reinforce the mortar against disintegration by resisting further opening of initial cracks and disallowing the microcracks from growing into macro cracks [18].

The strength development at 0.1% pp fiber addition varied depending upon the nature of mixtures. The mortar containing 7% NS showed greater average enhancement by 6.49% compared to plain cement mortar by 3.1%. At 0.3% fiber addition, the compressive strength of plain cement mortar decreased contrary to mortar containing 7% NS that still increased. It is obvious that increase in pp
dosage beyond 0.3% decreases the compressive strength. This is understandable because large contents of pp fibers are more difficult to disperse uniformly. Therefore fibers form clusters and create more micro-defects in cement matrix which inevitably reduces the compressive strength of mortar.

**Figure 3. Flexural strength of different mortar mixtures according to the PP content**

**Flexural strength:** The flexural strength of mortar specimens are presented in Figure 3. Comparing the flexural strength of nonfibrous specimens revealed that NS effectively increased the flexural strength of mortar. Results of fiber reinforced specimens showed that the flexural strength in fiber reinforced mortars was slightly higher than that of mortars without fibers. The values of flexural strength of cement composites increased with increasing the fiber content until it reached an optimal amount of 0.3% and then dropped to some lower value at 0.5%, however for mortar containing NS a slight increase of flexural strength was observed beyond 0.3%. It should be noticed that presence of NS in cement matrix improved the effectiveness of fibers in reinforcement of cement mortar. The microstructure of cement paste at the interfacial between fiber and matrix is the most important region influences of the fibers effectiveness. The addition of NS strengthens this weak region through reduction of the internal porosity especially in the transition layer by consumption of porous portlandite crystals which array in the interfacial between fiber and matrix. Therefore, fiber/matrix contact area increases and higher friction can be formed between the two. Typical flexural load deflection response of different mixtures containing 0%, 0.1%, 0.3% and 0.5% PP fibers at 90 days are represented in Figure 4. The test was controlled automatically by computer with a constant cross head movement of 1mm/min. It was found from the figures that for the unreinforced mortar, the materials demonstrated brittle behavior. The samples fully fractured with increase of mid span deflection after peak load while fiber reinforced mortar exhibited some what ductile behavior. A study of the load-deflection graphs showed that mortar containing NS was obviously more brittle than that of plain mortar, however integrating PP fibers somewhat compensated for this shortage. A small effect was noted upon fiber volume fraction of 0.1% and a relatively bigger increase was observed while increasing fiber content to 0.5%. When cracks occur and propagate, fibers are able to bridge across the surface of the cracks and prevent the crack face separation in the tension half of the reinforced beam. The fibers sustain the load until they pullout from the matrix. This
mechanism provides an additional energy-absorbing which leads to a stable fracture process and higher fracture energy. The presence of NS enhanced the efficiency of transforming load from matrix to fiber by increasing the friction coefficient between fiber and composite matrix. Hence effect of pp fibers on post-peak resistance was more obvious for mortars containing NS.

**Water absorption:** The water absorption of specimens is shown in Table 5. A study of the water absorption values of the unreinforced specimens revealed that incorporating NS into cement mortar improved the water absorption properties of the products. The reason for this observation is that the fine particles of pozzolan block the channels connecting capillary pores in cement paste and generate more homogenous distribution of CSH gel resulting in less pore structure and permeable voids [19].

Adding PP fibers changed the water absorption properties. The water absorption values of the mixtures decreased at 0.1% fiber content. It was observed that increasing the fiber percentage increased the water absorption of cement mortars. The reason behind this observation could be the poor dispersion of PP fibers in mortar that consequently increases the pore volume of cement matrix, for plain
cement mortar water absorption started to rise up at 0.3% fiber content, while in mortars containing NS at 0.5%. This means that presence of NS in cement matrix provided better fiber dispersion. The reason may be due to an increase in the cohesiveness of the cementitious matrix by NS which is beneficial for better dispersion of PP fibers [20].

Table 5: Water absorption of different mixes.

<table>
<thead>
<tr>
<th>Batch No</th>
<th>Absorption (%)</th>
<th>Batch No</th>
<th>Absorption (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>6.120</td>
<td>5</td>
<td>6.45</td>
</tr>
<tr>
<td>2</td>
<td>4.230</td>
<td>6</td>
<td>4.187</td>
</tr>
<tr>
<td>3</td>
<td>6.040</td>
<td>7</td>
<td>7.091</td>
</tr>
<tr>
<td>4</td>
<td>4.204</td>
<td>8</td>
<td>4.211</td>
</tr>
</tbody>
</table>

Shrinkage behavior: The shrinkage behavior of mortars is presented in figures 5 and 6. From the results it can be concluded that presence of NS in mortar increased the drying shrinkage apparently. It may be due to self desiccation caused by pore size refinement of NS [21]. Moreover, from the data presented by the previous researchers it is seen that NS particles act as an activator to accelerate cement hydration [22]. Therefore, the autogenous shrinkage related to chemical shrinkage can be increased.

Results of fiber reinforced specimens demonstrated that small amounts of fiber could contribute positively to moderate the length change caused by drying shrinkage. All the mortars reinforced with 0.1% pp fiber, provided better improvement for shrinkage. Obviously, using higher content of PP fibers (beyond 0.3%) did not work for moderating shrinkage strain. At 0.5% PP content, drying shrinkage of all specimens increased even more than reference mortars. More investigations are needed to explain this effect.

Figure 5. Effect of PP fibers on shrinkage of plain cement mortar

Figure 6. Effect of PP fibers on shrinkage of cement mortar containing 7% nano-SiO$_2$

4. CONCLUSIONS

A comprehensive experimental investigation was carried out to evaluate the influence of nano-SiO$_2$ on properties of fiber reinforced cement composite mortars. Based on the test and analysis results the following preliminary conclusions are obtained.
Utilizing polypropylene fibers in cement matrix caused a slight enhancement in compressive and flexural strength. The contribution of further increase of the fiber content to mechanical strength was not positive. A possible reason for this observation could be the poor dispersion of PP fibers in mortar that increases pore volume and creates more micro defects in cement matrix.

The fiber reinforced mortar demonstrated higher post-peak flexural strength compared with reference mortars. This effect was more obvious at larger contents of fibers. The effectiveness of the fiber reinforcement on mechanical strength somewhat improved with the incorporation of nano-SiO₂ particles. This can be due to reduction of the internal porosity especially in fiber/matrix transition zone that provides higher contact surface and hence friction between the two.

Water absorption of ordinary mortar decreased by incorporating nano-SiO₂. Adding small amount of pp fibers resulted in an improvement in water absorption characteristics, however higher amounts of fiber especially in ordinary cement mortar did not have any positive effect.

Presence of nano-SiO₂ in cement matrix increased the dying shrinkage of mortars. The inclusion of fiber reinforcement within composite cement mortar could moderate this effect. However, utilizing high contents of fiber (beyond 0.3%) didn’t have any positive impact on shrinkage strain.

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