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DESIGN OF MASONRY INFILLED REINFORCED CONCRETE FRAMES IN DIFFERENT SEISMIC CODES

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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 conFigure urations, 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. INATURAL 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 1: Natural period in different codes

Structural type	**IS 2800 [6]**	**UBC 97[7]**	**ASCE-06 [8] & NEHRP 2003 [9]**	**EC8 [10]**
Steel moment-resisting frames	$0.08(H)^{0.75}$	$0.0853(h_n)^{0.75}$	$0.0724(h_n)^{0.8}$	$0.085(H)^{0.75}$
RC moment-resisting frames	$0.07(H)^{0.75}$	$0.0731(h_n)^{0.75}$	$0.0466(h_n)^{0.9}$	$0.075(H)^{0.75}$
For structures with MI walls	Steel moment frames: $0.8*.08(H)^{0.75}$ Concrete moment frames: $0.8*0.7(H)^{0.75}$	$0.0743(h_n)^{0.75}/\sqrt{A_c}$ (see note no.1)	-	$0.075(H)^{0.75}/\sqrt{A_c}$ (see note no.2)

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

A_c is the combined 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. De 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.

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

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

A_i is the effective cross-sectional area of the wall i in the first story of the building, in m^2. Lwi 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 2: Response reduction factor in different codes

Lateral Resisting System	Allowable Stress		Ultimate Strength		EC8 [10]
	IS 2800 [6]	UBC 94 [14]	UBC97 [7]	New American Codes [8,9 & 15]	
CSMF[1]	10	12	8.5	8	$4.5\alpha_u/\alpha_1$
CIMF[2]	7	8	5.5	5	$3\alpha_u/\alpha_1$
COMF[3]	4	5	3.5	3	-
CSMF[1] + MI Walls	10	8	5.5	SMW[4] 5.5 IMW[5] 4	2[8]
CIMF[2] + MI Walls	7[7]	7	4.2	IMW[5] 3.5 OMW[6] 3	2[8]
COMF[3] + MI Walls	-	6	4.2	-	-

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 η that is given by:

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

where q is the response reduction factor given in Table-2, ΔV_{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 η 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].

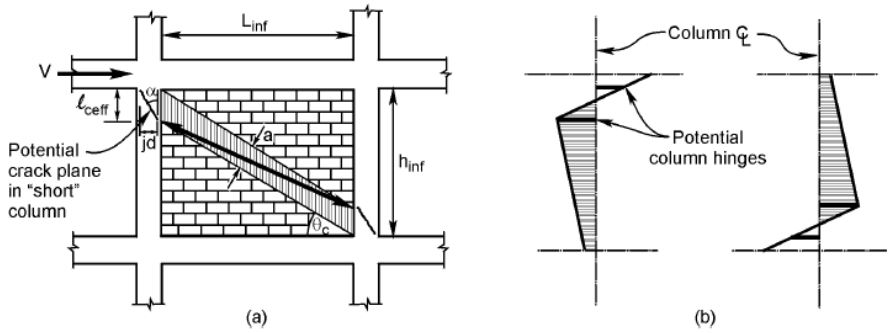

Figure 1. Modeling the adverse effect of an infill panel on the performance of the perimeter frame showing (a) the placement of the strut, and (b) the moment pattern on the columns

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 ρ_w is given as:

$$\rho_w = 1 - 2.5A_r,\ \rho_w \geq 0 \tag{2}$$

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 $\lambda_{opening}$ based on the width of opening measured across a horizontal plane $L_{opening}$ and given by Equation (3):

$$\lambda_{opening} = 1 - \frac{1.5L_{opening}}{L_{\inf}}, \lambda_{opening} \geq 0 \tag{3}$$

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 m^2 area .Longitudinal steel in the element shall not be less than 300 mm^2 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_{ceffi}) 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_{ceffi}, 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.

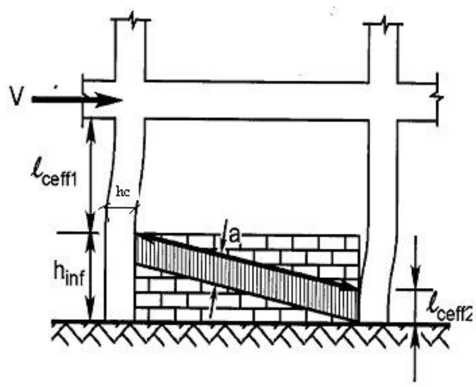

Figure 2. The effect of partial infills on frame performance

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.

$$V = \gamma_{Rd} ((M_{Rd,c1} + M_{Rd,c2}) / L_{cl}) \tag{4}$$

where L_{c1} is the contact length (L_{ceff} or L_{ceff1}), and γ_{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: Sesmic 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.

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