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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.

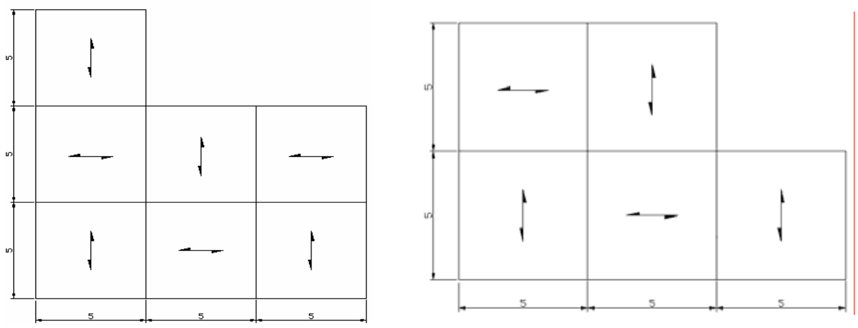
Keywords: pushover, concrete frame, Seismic assessment, irregularity, design codes

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 need 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].



(a) (b)

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

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/m^2 (due to self weight, finishes and permanent partitions) and to live loads equal to 200 kg/m^2. 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 1: Design lateral forces

Buildings	Loads (ton)
3-story	52.1
4-story	51.7
5-story	65.1

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

$$GR_1 = 1.1\ (Q_D + Q_L) \quad (1)$$

$$GR_2 = 0.9\ Q_D \quad (2)$$

$$GR_3 = Q_D + 0.2\, Q_L \qquad (3)$$

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) PX1 and –PX1 triangular distributions of lateral forces + GR_1
(b) PX2 and –PX2 triangular distributions of lateral forces + GR2
(c) PX3 and –PX3 triangular distributions of lateral forces + GR3
(d) PY1 and –PY1 triangular distributions of lateral forces + GR1
(e) PY2 and –PY2 triangular distributions of lateral forces + GR2
(f) PY3 and –PY3 triangular distributions of lateral forces + GR3
(g) FX1 and –FX1 rectangular distributions of lateral forces + GR1
(h) FX2 and –FX2 rectangular distributions of lateral forces + GR2
(i) FX3 and –FX3 rectangular distributions of lateral forces + GR3
(j) FY1 and –FY1 rectangular distributions of lateral forces + GR1
(k) FY2 and –FY2 rectangular distributions of lateral forces + GR2
(l) FY3 and –FY3 rectangular distributions of lateral forces + GR3

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 0f 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/cm^2 and steel having yield strength equal to 4000 kg/cm^2.

Table 2: Columns and beams cross-sections

Buildings		**3-storey**	**4-storey**	**5-storey**
Columns (dimensions in cm.)	First Floor	B= 35 8 Φ20	B= 40 12 Φ 20	B= 40 16 Φ 20
	Second Floor	B= 30 8 Φ 20	B= 35 8 Φ 20	B= 40 12 Φ 20
	Third Floor	B= 30 8 Φ 20	B= 30 8 Φ 20	B= 35 8 Φ 20
	Fourth Floor	-	B= 30 8 Φ 20	B= 30 8 Φ 20
	Fifth Floor	-	-	B= 30 8 Φ 20
Beams (dimensions in cm.)	All	b= 20 h=30	b=20 h=30	b=20 h=30

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.

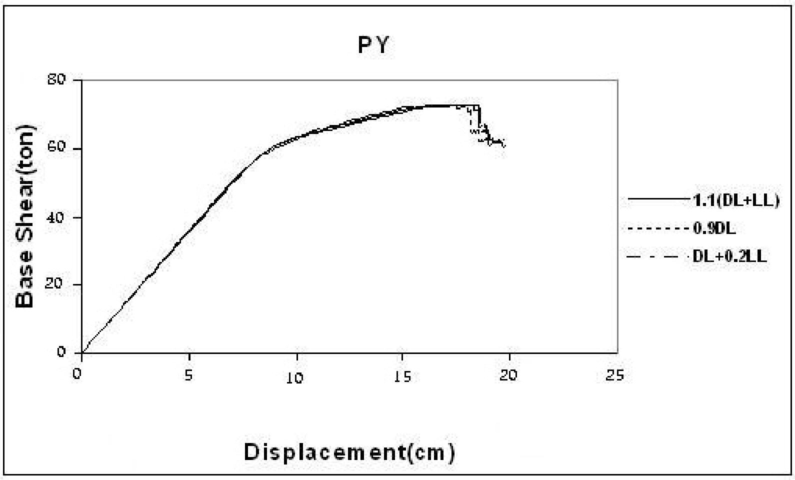
A sample of the displacements corresponding to the yield points for different vertical and lateral loads for the 5-story is given in Table 3. Same calculations are repeated for pushover curves in the negative direction and the results are similar to those given in Table 3.

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

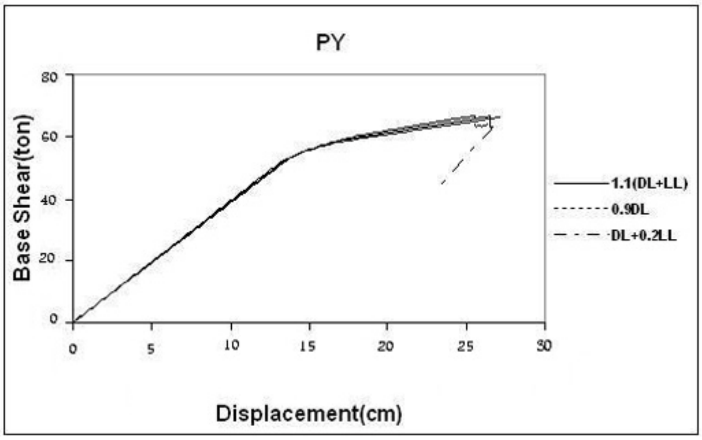
Loads	F_x	P_x	F_y	P_y
GR1	17	17	18	18
GR2	17	17	18	18
GR3	17	17	18	18

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).



(a)



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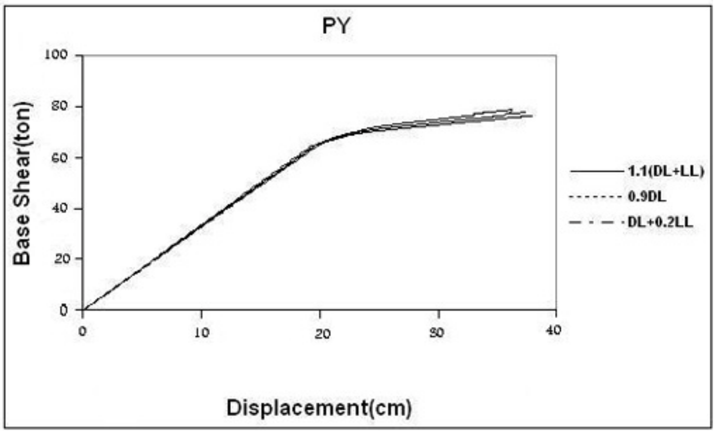


(c)

Figure 2. The performance curves for (a) 3-story building (b) 4-story building (c) 5-story building

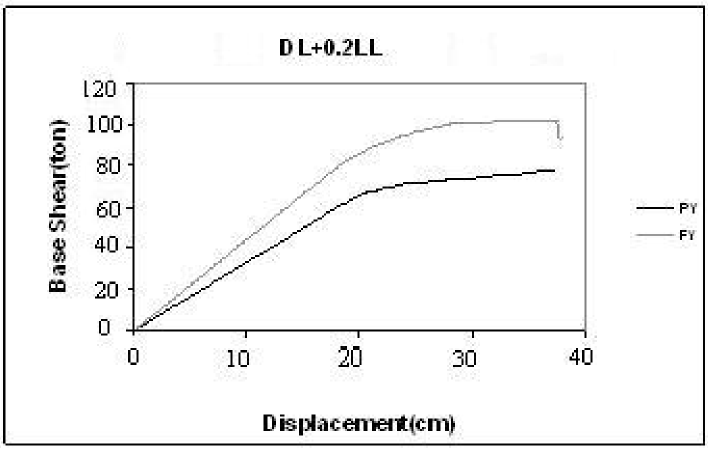


Figure 3. The performance curves for 5-story building for GR3

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

Lateral Loads	Displacements at the Performance Point (cm)	Forces at the Performance Point (ton)	Lateral Loads	Displacements at the Performance Point (cm)	Forces at the Performance Point (ton)
PX_1	19.73	69.29	$-PX_1$	-19.87	-69.27
PX_2	19.34	68.23	$-PX_2$	-19.46	-68.26
PX_3	19.53	68.84	$-PX_3$	-19.67	-68.80
FX_1	17.73	84.57	$-FX_1$	-17.87	-84.54
FX_2	17.32	83.78	$-FX_2$	-17.44	-83.83
FX_3	17.33	83.50	$-FX_3$	-17.67	-84.20

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

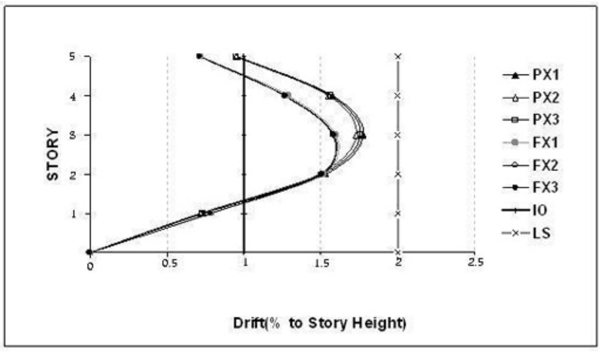
Lateral Loads	Displacements at the Performance Point (cm)	Forces at the Performance Point (ton)	Lateral Loads	Displacements at the Performance Point (cm)	Forces at the Performance Point (ton)
PY_1	20.97	66.98	$-PY_1$	-21.03	-66.98
PY_2	20.58	66.89	$-PY_2$	-20.62	-66.13
PY_3	20.77	67.00	$-PY_3$	-20.83	-66.97
FY_1	18.77	81.23	$-FY_1$	-18.83	-81.24
FY_2	18.37	81.27	$-FY_2$	-18.37	-81.27
FY_3	18.57	81.25	$-FY_3$	-18.63	-81.25

3.4. Maximum Displacement

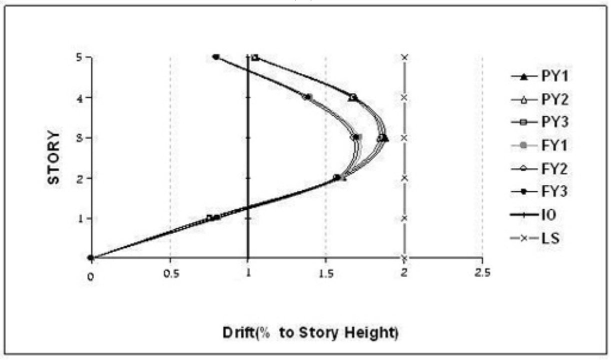
Considering the maximum roof displacement of the buildings, the results obtained denote that all of the frames in both directions are within the life safety performance level.

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.



(a)



(b)

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].

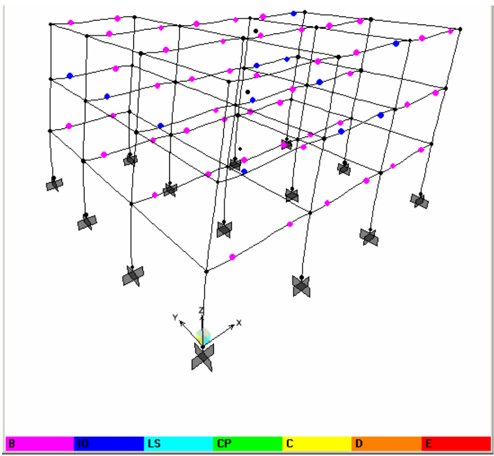


Figure 5. Plastic hinges in the 3-story building under PX_1

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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