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# PREDICTION OF SEISMIC OVERSTRENGTH IN CONCRETE MOMENT RESISTING FRAMES USING INCREMENTAL STATIC AND DYNAMIC ANALYSES

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## SUMMARY

Structural overstrength in reinforced concrete frame structures was investigated through analytical research. A large number of multi-bay, multi-story buildings were designed and analyzed using both dynamic inelastic time history and static inelastic pushover analyses. The variables included the number of bays and stories. This resulted in the design of 25 reinforced concrete frame buildings. A large number of actual earthquake records were used in dynamic analysis. The results were assessed in terms of lateral drift ratios and base shear forces. It was shown that the number of bays did not affect overstrength significantly. Low-rise buildings had higher overstrength ratios, which, when the working stress level design of the Iranian Code (Standard No. 2800) [1] was considered, decreased from approximately 5 to 2.5 for buildings with 1-story to 4-stories, respectively. The overstrength ratio remained about the same for buildings in the range of 4 to 10 stories at an approximate value of 2.5. When ultimate stress levels were considered in design, to be consistent with North American Building Codes, the overstrength ratios translated into 3.3 to 1.7 for buildings with 1-story and 4–stories, respectively. For the latter case, the buildings showed an overstrength ratio of approximately 1.7 when the story height remained between 4 and 10. A close correlation was obtained between the results of static inelastic (pushover) and dynamic inelastic analyses, suggesting that it may be sufficient to use pushover analysis to establish structural overstrength.

## INTRODUCTION

Structures are routinely designed and built to have higher strengths than those required for service load conditions. Typically, members have larger sizes and higher material strengths than the minimum design requirements stipulated in building codes. The capacity design procedure implemented for seismic design also results in increased strengths. Furthermore, the redundancy in the structural system may result in redistribution of stresses and increased overall strength of the structure. While building codes address

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structural overstrength either implicitly or explicitly, it is often difficult to assess the level of overstrength with reasonable accuracy. This paper describes the use of static and dynamic analysis in predicting overstrength in reinforced concrete moment resisting frame structures. A total of 25 building frames were analyzed using incremental static inelastic analysis, commonly known as the pushover analysis, as well as dynamic inelastic response time history analysis with incrementally increasing peak ground acceleration until failure. A total of 10 earthquake records were used in dynamic analyses. Frame buildings with 1, 2, 4, 6, and 10 stories, having 1, 2, 3, 4, or 5 bays were designed using seismic force levels obtained from the Iranian Seismic Code [1] and proportioned using the ACI318-02 Building Code [2]. Structural overstrength was evaluated on the basis of maximum strength to design strength ratio.

## DESCRIPTION OF STRUCTURES CONSIDERED

Twenty five reinforced concrete moment resisting frames were designed and analyzed as part of the current research program. Figure 1 illustrates the elevation views of frames with one, two, four, six and ten stories, having one to five bay(s). The frames were labeled with the same reference code, consisting of two characters and three digits, as "Hixyz". "H" represents the location of structure which is assumed to be high seismic risk area according to the Iranian Standard [1] and designed to be highly ductile (R=10). The second character "i" stands for inelastic, and represents the type of analysis employed. The two digits "x" and "y" indicate the number of stories (01 to 10) and the last digit "z" indicates the number of bays (1 to 5). The frames were designed using the seismic load provisions of the Iranian Seismic Code [1] and were proportioned using ACI-318-02 [2].

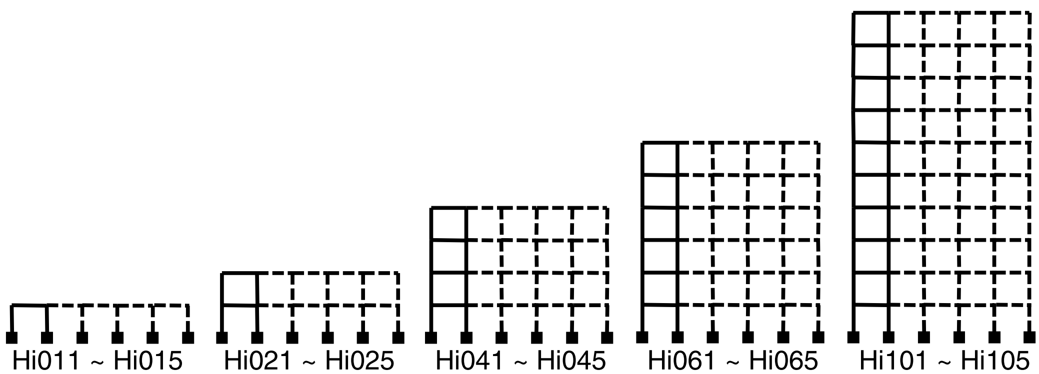


**Figure1: Schematic representation of the frames selected (broken lines represent the variation of the number of bays from 1 to 5)**

## MODELING OF STRUCTURES

Incremental static (pushover) and dynamic time history analyses with incrementally increasing peak ground acceleration (PGA) were conducted using computer program IDARC [3]. IDARC was developed for inelastic damage analysis of reinforced concrete structures. It utilizes macro-models for reinforced concrete structures and establishes their damageability under horizontal and vertical earthquake excitations. Some of the modeling features include:

- Flexibility approach in constructing the element stiffness matrix while allowing for the variation of point of contraflexure.
- General hysteretic model that is capable of accounting for the three main behavioral patterns in reinforced concrete elements, i.e., stiffness degradation, strength deterioration and pinching.
- A non-symmetric tri-linear envelope curve that distinguishes cracking and yielding points.
- Determination of the tri-linear envelope parameters based on empirical or mechanical models.

- Expression of response statistics using the Park-Ang damage index, enabling the interpretation of damage sustained by the structure.

In addition, the program can extract response information on selected sub-assemblages and output specified displacement, drift and story shear histories. It uses a generalized fiber model for the generation of moment-curvature envelopes based on cross-sectional data. P-Delta effects are included in the step-by-step analysis, and a single-step correction is applied to control unbalanced forces during event transition (stiffness changes during loading and unloading). The three-parameter Park hysteretic model [3], which is used for modeling members in IDARC, is shown in Figure 2.

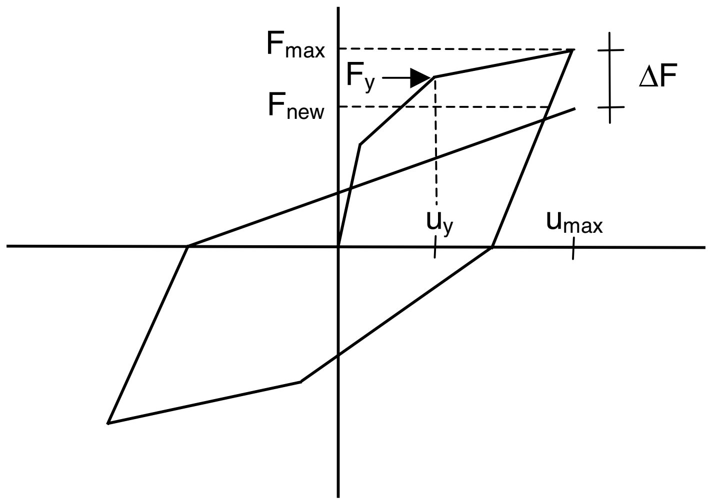


**Figure 2: Hysteretic model used in IDARC [3]**

## INPUT GROUND MOTIONS

Ten different ground acceleration records were considered (eight from Iranian provinces and two from U.S.A locations). Table 1 provides some relevant information for the records. The records were scaled to match the same maximum acceleration value (pseudo-accelerations are normalized relative to their values at zero period [4]) to eliminate the effects of PGA in comparing the effects of frequency content. Figure 3 depicts normalized 5% damped elastic response spectra for all records. In addition, the average spectrum and the design spectrum for subsoil class II of the Iranian Standard 2800 [1] are included in the same figure. As can be seen in Figure 3, the dispersion of spectral values is high. Also, the difference between the average and code spectra is considerable. Nevertheless, when the PGA is gradually increased, the spectra and their average cover all the vibration modes of interest.

## PERFORMANCE CRITERIA

Performance criteria must be defined for structures or structural components to monitor response during analysis. These criteria also help define yield and collapse states of a structure. In this research, the following performance levels were used to identify the limiting conditions.

a) The interstory drift ratio is limited to 3% in nonlinear analysis. This is consistent with the limit specified in the Iranian Standard 2800 [1] and close to the limits specified in other codes of practice which vary between 2% and 3%.

b) Structural instability is based either on plastic hinge formation or conversion of structure to a mechanism.

c) The limiting value for damage index is 1, as per Park-Ang (Beyond repair and prior to the loss of building).
d) The stability index is limited to 0.25, as per the Iranian Standard 2800[1].
e) Curvature is limited to the ultimate curvature.

The structure is assumed to have failed when the structure meets one or more of the above criteria. Table 2 provides a summary of the limiting performance criteria outlined above.

**Table 1: Earthquake ground motions used in analyses**

| Earthquake | Station | Year | PGA (g) |
|---|---|---|---|
| Abbar | Iran | 1990 | 0.526 |
| El Centro (1) | USA | 1940 | 0.290 |
| El Centro (2) | USA | 1940 | 0.319 |
| Ghaen | Iran | 1976 | 0.168 |
| Golbaf | Iran | 1981 | 0.222 |
| Lahijan | Iran | 1990 | 0.176 |
| Naghan | Iran | 1977 | 0.723 |
| Tabas | Iran | 1978 | 0.933 |
| Zanjan | Iran | 1990 | 0.127 |
| Zanjiran | Iran | 1994 | 1.006 |

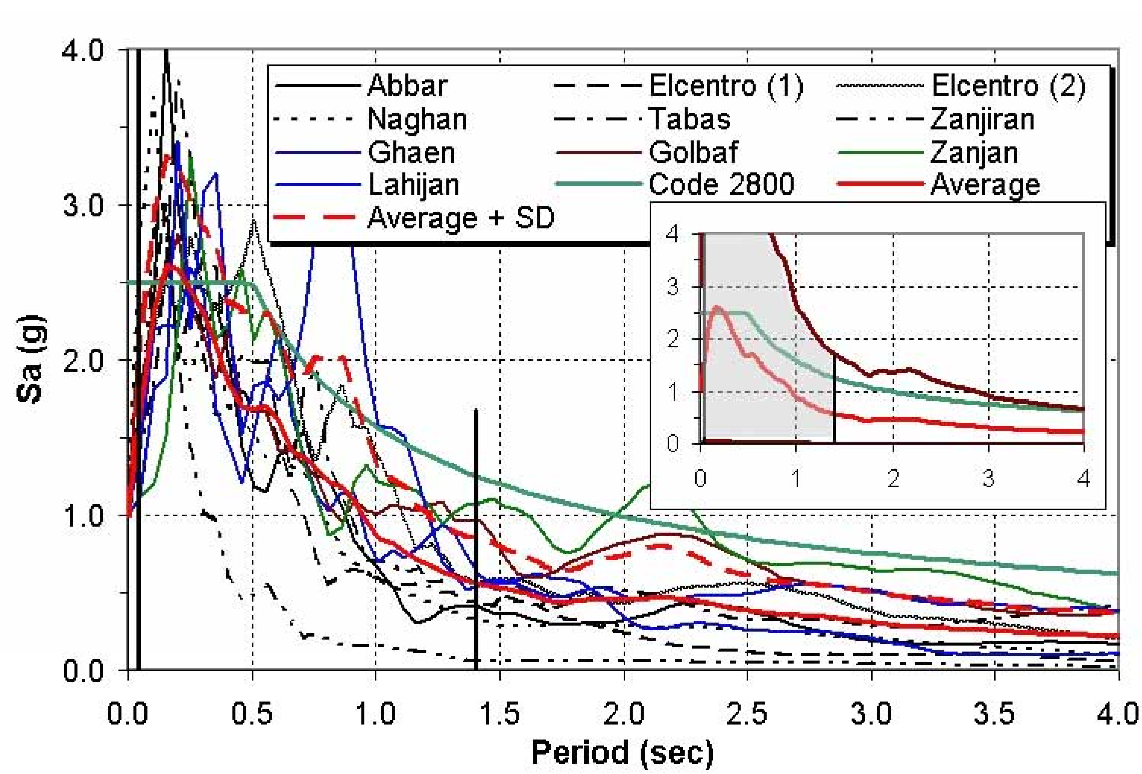


**Figure 3: Response spectra for earthquake records, their average and design code**

**Table 2: Response criteria for structures**

| Parameter | Description | Limitation |
|---|---|---|
| ID | Interstory drift ratio | = 3% |
| - | Stability | Mechanism |
| DI | Overall Park-Ang damage index | = 1 |
| $\theta$ | Stability Index = interstory drift × vertical loads / story shear | = 0.25 |
| $\phi/\phi_u$ | Curvature control | = 1 |

## INCREMENTAL STATIC (PUSHOVER) ANALYSIS

The structures were analyzed under incrementally increasing lateral loads with inverted triangular distribution and constant gravity loads, to estimate overstrength under static inelastic load conditions. This pushover analysis was conducted using program IDARC. The response curves were obtained in terms of overall drift versus base shear coefficient ($C_b$), where the base shear coefficient was defined as the ratio of base shear to structure's weight. A sample response of pushover analyses is illustrated in Figure 4 for the 5-bay, 10-story frame building (Hi105). Bi-linear idealization of the curve is also plotted in Figure 4, following the recommendations of Park [5] for reinforced concrete members. Accordingly, the effective elastic stiffness is obtained as the slope of the line connecting the origin to either the point of first yield or 75% of the ultimate load, whichever is less.

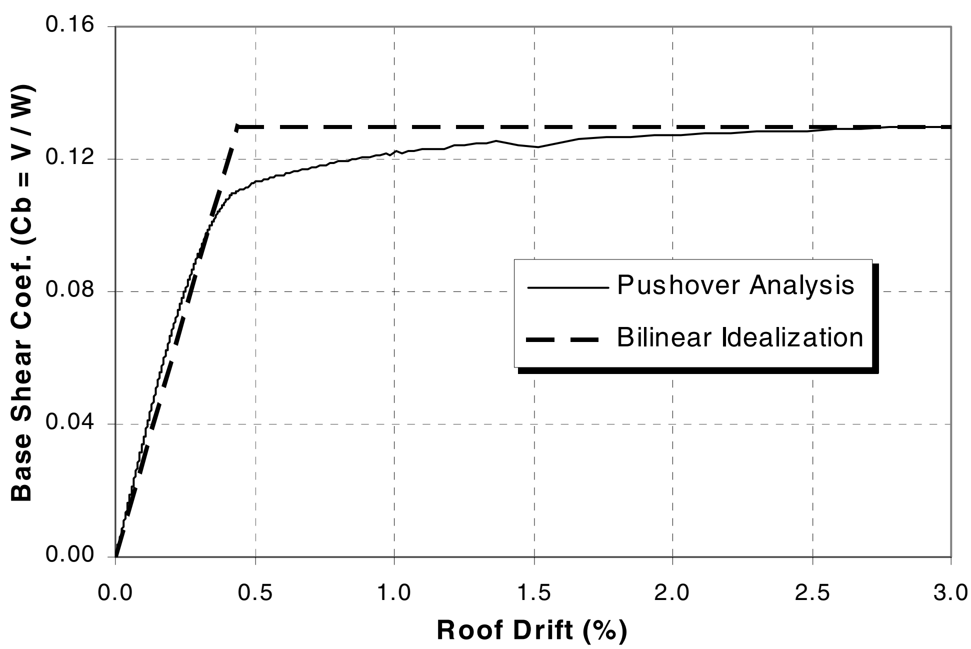


**Figure 4: The results of incremental static (pushover) analysis and the idealized bilinear elasto-plastic response for structure Hi105**

## INCREMENTAL DYNAMIC COLLAPSE ANALYSIS

A large number of nonlinear inelastic time-history analyses were carried out using the incremental dynamic collapse analysis technique. The performance of 25 frame structures was assessed under 10 earthquake records, as PGA was increased incrementally in each analysis. Accordingly, the PGA was increased in increments of 0.02g, starting from 0.02g, until the structural failure was encountered on the basis of the performance criteria outlined earlier and summarized in Table 2. The initial value of 0.02g was consistent with the design procedure employed, which called for the first yield to occur at a PGA greater than 0.35g/R. This would result in 0.035g for the structures considered in the current study.

Structural response was monitored during analysis as PGA was increased. When first yielding was attained, the base shear coefficient and the corresponding PGA were noted. Similarly, when failure was encountered, the corresponding base shear coefficient and PGA were recorded to be used in overstrength calculations.

## RESULTS OF ANALYSES

The results of static push over and dynamic time history analyses were evaluated to establish overstrength in structures. The overstrength was estimated by following the standard definitions also used by other researchers [6, 7]. Accordingly, it was defined as;

$$R_o(size,\phi) = \frac{C_y}{C_d} \tag{1}$$

$$R_o(redun,sth) = \frac{C_m}{C_y} \tag{2}$$

$$R_o = R_o(size,\phi)\cdot R_o(redun,sth) = \frac{C_m}{C_d} \tag{3}$$

Where;

| | |
|---|---|
| $C_d$ | : Design base shear coefficient. |
| $C_y$ | : Base shear coefficient at yield. |
| $C_m$ | : Base shear coefficient at failure or maximum attained during dynamic analysis as governed by the failure criteria. |
| $R_o(size,\ \phi)$ | : Overstrength arising from restricted choices for member sizes, rounding up of sizes and dimensions, and differences between nominal and factored resistances. |
| $R_o(redun,sth)$ | : Overstrength arising from redundancy (until a collapse mechanism is formed) and steel strain hardening. |
| $R_o$ | : Total overstrength. |

The same approach was used for both incremental static and incremental dynamic collapse analyses. The Iranian seismic design code is based on working stress. The $C_d$ values were obtained from the Code expression (C = ABI/R), and were used in design. To facilitate comparison with the ultimate stress level employed in most codes in the world the overstrength arising from size and factored resistance $R_o(size,\ \phi)$ should be divided by approximately 1.4 to 1.5 [8].

The results obtained for incremental dynamic collapse analyses are shown in Table 3. The table presents the components of overstrength and their standard deviation (SD). The values of $R_o(size,\phi)$ and $R_o(redun,sth)$ do not appear to be affected significantly by the number of bays. A similar observation was made for the results of pushover analyses. On the other hand, they do show a substantial variation with the number of stories. Therefore, the results are presented in Tables 4 and 5 in terms of average values of $R_o$ over the number of bays, for incremental static and dynamic collapse methods, respectively. These tables also indicate the coefficient of variation (COV), i.e., the ratio of SD to mean value. The examination of the standard deviations, coefficients of variations and mean values given in Tables 4 and 5 confirms the rationale used in finding the average values of $R_o$ over 3 to 5 bay structures in each group. This approach embarks on conservative values of $R_o$ for one and two bay structures, which are not widely used in actual construction practice. The results of incremental static and incremental dynamic collapse analysis methods are compared in Table 6 and Figure 5. These comparisons are based on working stress values ($R_w$ used in design code) and should be divided by 1.4 to 1.5 to obtain corresponding values at the ultimate stress level (R used in design code) [8].

**Table 3: Results of incremental dynamic collapse analyses**

| Ref. Code | First Yield | | Failure | | Design | $R_o(size,\phi)$ | $R_o(redun,sth)$ | $R_o$ | | $R_o/1.5$ |
|---|---|---|---|---|---|---|---|---|---|---|
| | $C_s$ | SD | $C_y$ | SD | $C_d$ | $C_y/C_d$ | $C_m/C_y$ | $C_m/C_d$ | SD | |
| Hi011 | 0.44 | 0.02 | 0.52 | 0.07 | 0.088 | 4.96 | 1.20 | 5.96 | 0.83 | 3.97 |
| Hi012 | 0.40 | 0.01 | 0.51 | 0.03 | 0.088 | 4.58 | 1.27 | 5.79 | 0.36 | 3.86 |
| Hi013 | 0.38 | 0.01 | 0.47 | 0.03 | 0.088 | 4.32 | 1.24 | 5.34 | 0.29 | 3.56 |
| Hi014 | 0.36 | 0.01 | 0.45 | 0.02 | 0.088 | 4.13 | 1.24 | 5.10 | 0.27 | 3.40 |
| Hi015 | 0.35 | 0.02 | 0.44 | 0.03 | 0.088 | 3.99 | 1.25 | 4.97 | 0.34 | 3.31 |
| Hi021 | 0.25 | 0.01 | 0.33 | 0.03 | 0.088 | 2.79 | 1.34 | 3.74 | 0.29 | 2.49 |
| Hi022 | 0.25 | 0.01 | 0.31 | 0.02 | 0.088 | 2.80 | 1.25 | 3.48 | 0.20 | 2.32 |
| Hi023 | 0.25 | 0.01 | 0.30 | 0.01 | 0.088 | 2.85 | 1.18 | 3.36 | 0.15 | 2.24 |
| Hi024 | 0.24 | 0.01 | 0.29 | 0.02 | 0.088 | 2.76 | 1.21 | 3.35 | 0.17 | 2.23 |
| Hi025 | 0.24 | 0.01 | 0.29 | 0.01 | 0.088 | 2.76 | 1.18 | 3.26 | 0.16 | 2.18 |
| Hi041 | 0.17 | 0.01 | 0.24 | 0.03 | 0.088 | 1.98 | 1.37 | 2.70 | 0.32 | 1.80 |
| Hi042 | 0.18 | 0.01 | 0.22 | 0.02 | 0.088 | 2.03 | 1.25 | 2.53 | 0.18 | 1.69 |
| Hi043 | 0.17 | 0.01 | 0.22 | 0.01 | 0.088 | 1.96 | 1.27 | 2.49 | 0.16 | 1.66 |
| Hi044 | 0.18 | 0.01 | 0.21 | 0.01 | 0.088 | 2.01 | 1.18 | 2.37 | 0.12 | 1.58 |
| Hi045 | 0.18 | 0.01 | 0.21 | 0.01 | 0.088 | 2.03 | 1.20 | 2.42 | 0.12 | 1.61 |
| Hi061 | 0.15 | 0.01 | 0.22 | 0.03 | 0.077 | 1.90 | 1.49 | 2.83 | 0.43 | 1.89 |
| Hi062 | 0.14 | 0.01 | 0.20 | 0.02 | 0.077 | 1.87 | 1.41 | 2.64 | 0.29 | 1.76 |
| Hi063 | 0.15 | 0.01 | 0.19 | 0.03 | 0.077 | 1.93 | 1.28 | 2.46 | 0.35 | 1.64 |
| Hi064 | 0.15 | 0.01 | 0.19 | 0.02 | 0.077 | 1.90 | 1.29 | 2.46 | 0.28 | 1.64 |
| Hi065 | 0.15 | 0.01 | 0.18 | 0.02 | 0.077 | 1.92 | 1.24 | 2.38 | 0.30 | 1.59 |
| Hi101 | 0.11 | 0.02 | 0.18 | 0.03 | 0.059 | 1.93 | 1.63 | 3.12 | 0.53 | 2.08 |
| Hi102 | 0.11 | 0.02 | 0.15 | 0.03 | 0.059 | 1.91 | 1.37 | 2.61 | 0.49 | 1.74 |
| Hi103 | 0.11 | 0.02 | 0.15 | 0.03 | 0.059 | 1.88 | 1.37 | 2.59 | 0.48 | 1.73 |
| Hi104 | 0.11 | 0.01 | 0.17 | 0.04 | 0.059 | 1.88 | 1.50 | 2.82 | 0.66 | 1.88 |
| Hi105 | 0.11 | 0.01 | 0.16 | 0.03 | 0.059 | 1.87 | 1.44 | 2.69 | 0.47 | 1.79 |

**Table 4: Results of $R_o$ obtained from incremental static analyses (average over bays)**

| No. of Stories | 1 to 5 Bay Structures | | | 2 to 5 Bay Structures | | | 3 to 5 Bay Structures | | |
|---|---|---|---|---|---|---|---|---|---|
| | Mean | SD | COV (%) | Mean | SD | COV (%) | Mean | SD | COV (%) |
| 1 | 5.79 | 0.38 | 6.6 | 5.69 | 0.35 | 6.1 | 5.53 | 0.18 | 3.2 |
| 2 | 3.65 | 0.26 | 7.0 | 3.55 | 0.10 | 3.0 | 3.50 | 0.07 | 1.9 |
| 4 | 2.38 | 0.05 | 2.2 | 2.36 | 0.03 | 1.2 | 2.35 | 0.03 | 1.1 |
| 6 | 2.26 | 0.02 | 0.8 | 2.26 | 0.02 | 0.8 | 2.25 | 0.02 | 0.8 |
| 10 | 2.23 | 0.04 | 1.8 | 2.24 | 0.04 | 1.8 | 2.22 | 0.03 | 1.2 |

**Table 5: Results of $R_o$ obtained from incremental dynamic collapse analyses (average over bays)**

| No. of Stories | 1 to 5 Bay Structures | | | 2 to 5 Bay Structures | | | 3 to 5 Bay Structures | | |
|---|---|---|---|---|---|---|---|---|---|
| | Mean | SD | COV (%) | Mean | SD | COV (%) | Mean | SD | COV (%) |
| 1 | 5.43 | 0.43 | 7.9 | 5.30 | 0.36 | 6.8 | 5.14 | 0.19 | 3.7 |
| 2 | 3.44 | 0.19 | 5.4 | 3.36 | 0.09 | 2.7 | 3.32 | 0.05 | 1.6 |
| 4 | 2.50 | 0.13 | 5.1 | 2.45 | 0.07 | 2.9 | 2.43 | 0.06 | 2.4 |
| 6 | 2.55 | 0.18 | 7.1 | 2.49 | 0.11 | 4.5 | 2.43 | 0.04 | 1.8 |
| 10 | 2.77 | 0.22 | 7.9 | 2.68 | 0.10 | 3.8 | 2.70 | 0.11 | 4.2 |

**Table 6: Calculated overstrength from incremental static and incremental dynamic collapse analysis methods (average over bays)**

| No. of Stories | Incremental Static Method | | Incremental Dynamic Collapse Method | |
|---|---|---|---|---|
| | Overstrength | Standard Deviation | Overstrength | Standard Deviation |
| 1 | 5.53 | 0.18 | 5.14 | 0.19 |
| 2 | 3.50 | 0.07 | 3.32 | 0.05 |
| 4 | 2.35 | 0.03 | 2.43 | 0.06 |
| 6 | 2.25 | 0.02 | 2.43 | 0.04 |
| 10 | 2.22 | 0.03 | 2.70 | 0.11 |

## CONCLUSION

The following conclusions can be drawn from the analytical investigation reported in this paper on reinforced concrete frame structures designed to have full ductility as per current seismic codes:

- Reinforced concrete frames designed on the basis of current building codes possess significant overstrength.
- Overstrength associated with restricted choice of member sizes, rounding up of sizes and dimensions during design, and differences between nominal and factored resistances varies between approximately 4 to 5 for a single-story building, and approximately 2 for a 10-story building, when the working stress level design is used on the basis of the current Iranian Code. When the ultimate stress level employed in North American Building Codes is employed in design, these values translate into approximately 2.7 to 3.3 for the single-story building and 1.3 for the 10-story building.
- Overstrength beyond yielding, associated with structural redundancy, does not show a significant variation with building height, with values ranging approximately between 1.2 and 1.6.
- Overall overstrength ratio reduces with the number of stories. Ratios established by dynamic analysis decreases from approximately 5 for a single-story building to approximately 2.5 for a 4-story building when the design is based on the working stress level of the Iranian Code. When the design is based on the ultimate stress level of North American Building Codes, the overstrength varies between 3.3 for the single-story building and 1.7 for the 4-story building. However, the overstrength factor remains approximately constant for buildings between 4 to 10 stories high, with an average ratio of 2.5 for buildings designed by the Iranian Code and 1.7 for buildings based on the North American Codes.
- Overstrength ratios established by dynamic inelastic time history analyses and static inelastic pushover analyses show similar values. Hence, it may be sufficiently accurate to establish structural overstrength by the static pushover analysis.

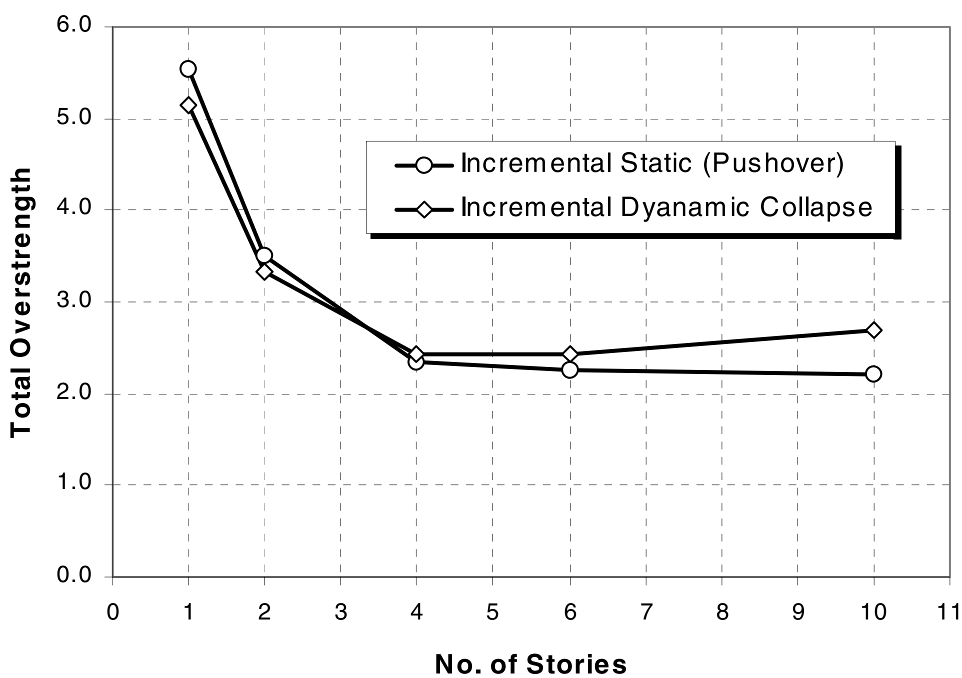


**Figure 5: Comparison between estimated overstrength for structures under incremental static and incremental dynamic collapse analysis methods**

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