Consistency of the Seismic Design and Rehabilitation Requirements for Frame Structures: Case Study of Iranian Seismic Codes

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SUMMARY:
This paper investigates the consistency of the forced-based seismic design requirements for frame structures (Standard 2800) with those of performance-based seismic rehabilitation guidelines (Guideline 360) in Iran. Each of these documents is very similar to the mainstream of the international design and rehabilitation standards. Both reinforced concrete and steel moment resisting frames are considered. The frames are designed according to Iranian seismic standard (2800) and then are subjected to assessment requirements corresponding to basic safety objective (Desired Performance) for rehabilitation. Nonlinear static analysis method is mainly used for the assessment purpose. The study considers frames with different heights in three groups, i.e. short, moderate and tall frames. The results of the study show that in basic safety objective level some columns in lower stories of the moderate and tall frames (RC and steel frames) do not satisfy the performance requirements and require rehabilitation measures. Recommendations for consistency are proposed.

Keywords: *Seismic Design, Seismic Rehabilitation, Frame Structures.*

1. INTRODUCTION

Performance based Engineering principles play a central role is producing the new generation of the seismic design and rehabilitation requirements. While it is almost two decades that these principles has been formulated but the pace of progress in rehabilitation of existing buildings has been different, to some extent, compared to that in design of new structure. In many countries the design standards are still based on conventional code format while the recently produced rehabilitation guidelines or standards, benefiting from the available performance based principles, have different structure and format. This is rationally leads to the question of consistency between different design and rehabilitation rules. Considering inherent conservatism in design methods it is expected that the newly designed buildings should show acceptable performance if subjected to rehabilitation requirements. Investigating possible inconsistencies will be useful in identifying the important issues for future code revisions.

For many years force based design has been the main method used in many seismic codes. About one hundred years ago, first attempts were made to define seismic loads in mathematical term and buildings were set to be designed for a lateral load equal to a fraction of their effective weight. Since then many researches has attempted to improve this method, and to add different effective parameters to calculate the seismic effects (Beavers, 2002).

Nowadays this method is the most popular process in seismic design codes such as Iranian Code of Practice for Seismic Resistance Design of Buildings known as Standard 2800. But in recent years with re-examination of the seismic design philosophy and following the observations made on massive damages in some buildings after facing strong ground motion a turning point has reached. Earthquake engineering community has put forward new initiatives to find more efficient ways in evaluating earthquake forces and assessing the performance of the buildings. Performance based design of structures is a new method which attracted considerable attention in the past twenty years of its introduction. It is based on our expectations of structural response during an earthquake. Performance based design, first introduced by Gulkan and Suzen in 1974 and later it was reflected in FEMA and ATC reports.

In 2002 the International Institute of Earthquake Engineering and Seismology in Iran prepared the first rehabilitation guidelines for the existing structures based on relevant FEMA, ATC and SAC reports. After a review process the second issue was officially published in 2005 which is known as “Code 360” (MPORG, 2007). The main objective of this paper is to assess the consistency of design and rehabilitation standards. Due to higher level of conservatism in methodologies and codes that are used for the design of the new structures than those which are intended for the rehabilitation of existing buildings, it is rationally expected that the newly designed structures should not require rehabilitation if subjected to evaluation and assessment process defined in rehabilitation standards. In this paper we will attempt to verify this expectation by designing structures based on design codes and using rehabilitation procedure to evaluate their response.

In the following first a brief background to the design based on standard 2800 and rehabilitation methods based on Code 360 is presented. Then the focus is turned to the application of these methods to some structural systems including reinforced concrete and steel frame structures. Possible conceptual and practical differences are discussed and some recommendations are made.

2. FUNDAMENTALS OF FORCE BASED DESIGN

In force-based design procedure, the response of the structure is usually evaluated through linear analysis because nonlinear methods are complicated and special skills are needed to interpret their results. In this procedure the effect of ductility and nonlinearity of elements is defined through a parameter which is known as response “reduction factor”. This parameter reduces seismic design forces and leads to lower strength but higher ductility elements. In Figure 1 “base shear- lateral displacement” curve is shown for a structure that results from performing a pushover nonlinear analysis on the equivalent one degree of freedom model of a structure under lateral load which corresponds to its first mode. In Figure 2, pushover curve idealized as a bilinear curve and the structural reduction factor, which defines the structure’s nonlinear behavior, is estimated through the following procedure.

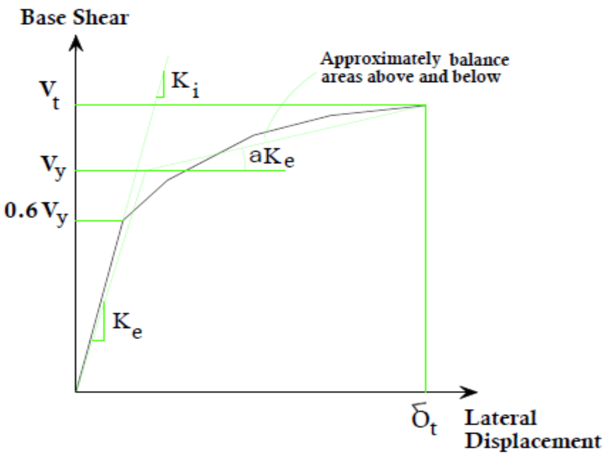


Figure 1. Base Shear-Lateral displacement curve

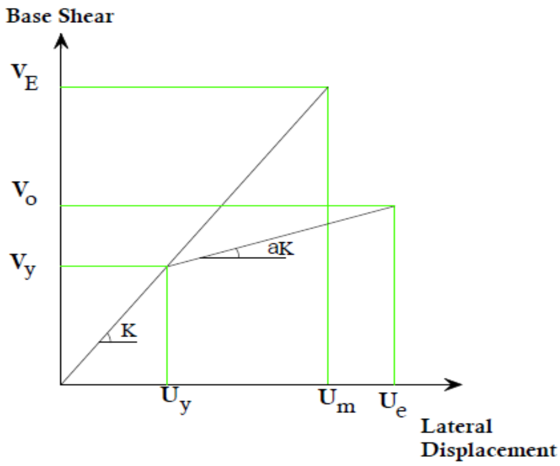


Figure 2. Bilinear idealized curve

The definition of the ductility “μ” for an equal single degree of freedom system with frequency “f” is given by Equation 2.1:

$$\mu = \frac{U_m}{U_y} \tag{2.1}$$

Then structural response reduction factor is defined as:

$$\begin{cases} R_\mu = 1 & f > 33\ \text{Hz} \\ R_\mu = \sqrt{2\mu - 1 + \alpha(\mu - 1)2} & 2 < f < 33\ \text{Hz} \\ R_\mu = 1 & f \leq 2\ \text{Hz} \end{cases} \tag{2.2}$$

Where "U_m" is the maximum and "U_y"is the yield displacement for single degree of freedom system. This process first introduced by "Newmark & Hall". If the effect of structural reduction factor ignored in force based design procedure, design will result in an elastic system and its elements would not be required to behave in nonlinear range. So structural reduction factor helps to use ductility as a characteristic of structures to dissipate earthquake energy and leads to economical designs. Response reduction factor of total system consists of three main parameters which include ductility, damping and over strength (Chopra, 2003) as shown in equation 2.3:

$$R = R_s.R_\mu.R_£ \tag{2.3}$$

Figure.3 and equations 2.4 to 2.10 show how force base design reduction factor, which known as "Behavior Factor" in Standard 2800, is estimated for a sample lateral resistant system (Hoseinzadeh, 2011):

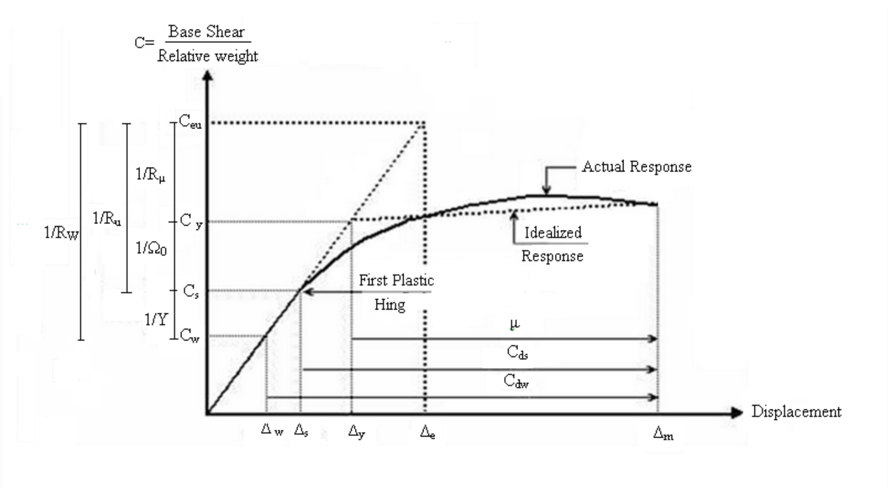


Figure 3. Estimating behavior factor from a pushover curve

$$R_\mu = \frac{C_{eu}}{C_y} \tag{2.4}$$

$$\Omega_0 = \frac{C_y}{C_s} \tag{2.5}$$

$$Y = \frac{C_S}{C_w} \tag{2.6}$$

$$\mu = \frac{\Delta_m}{\Delta_y} \tag{2.7}$$

$$C_{ds} = \frac{\Delta_m}{\Delta_s} \tag{2.8}$$

$$R_u = \frac{C_{eu}}{C_s} = \left(\frac{C_{eu}}{C_y}\right).\left(\frac{C_y}{C_s}\right) = R_\mu\,.\Omega_0 \tag{2.9}$$

$$R_w = \frac{C_{eu}}{C_w} = \left(\frac{C_{eu}}{C_y}\right).\left(\frac{C_y}{C_s}\right).\left(\frac{C_S}{C_w}\right) = R_\mu\,.\Omega_0.Y \tag{2.10}$$

Where:

Rμ, μ, Ω and Y respectively are force reduction factor due to ductility, ductility of structure, over strength factor and allowable stress factor.

Overstrength factor is normally used to provide sufficient safety margin for minor inevitable errors and differences between the design assumptions and actual behavior of the structure, it components and materials. Structural reduction factor takes part in “Design shear force” of Iranian 2800 standard in the “Base shear coefficient” as seen in equation 2.11 (BHRC, 2005):

$$C = \frac{A.B.I}{R} \tag{2.11}$$

Where:
A = $\frac{1}{g}$ × (Peak Ground Acceleration)
B = Response factor which describes the structure's response to the ground motion considering the four soil types introduced in the code and the structure's height.
I = structural importance factor

Structures are designed for a lateral load which is calculated based on equation 2.12, and distributed throughout the structure (BHRC, 2005):

$$V = C.W \tag{2.12}$$

V = Base shear force
C = Base shear coefficient
W= Effective height of building as defined in 2800 standard.

This lateral force distributed along the structure height using equation 2.13 :

$$F_i = \frac{w_x h_x}{\sum_{i=1}^{n} w_i.h_i}(V - F_t) \tag{2.13}$$

Linear method does not give accurate results because of simplifying assumptions used to make the analysis easier to run. For important and complicated structures monitoring nonlinear behavior of elements during earthquakes is needed. Therefore it might be necessary to use nonlinear analysis methods.

2. FUNDAMENTALS OF PERFORMANCE BASED DESIGN

In contrast to prescriptive design approaches, performance-based design provides a systematic methodology for assessing the performance capability of a building, system or component. It can be used to verify the equivalent performance of alternatives, deliver standard performance at a reduced cost, or substantiate higher performance needed for critical facilities (FEMA 445, 2006). It can also be used to assess the potential seismic performance of existing structures and estimate potential losses in the event of a seismic event, assess the potential performance of current prescriptive code requirements for new buildings, and serve as the basis for improvements to code-based seismic design criteria.
In a more general term “performance-based seismic design explicitly evaluates how a building is likely to perform, given the potential hazard it is likely to experience, considering uncertainties inherent in the quantification of potential hazard and uncertainties in assessment of the actual building response. It permits design of new buildings or upgrade of existing buildings with a realistic understanding of the risk of casualties, occupancy interruption, and economic loss that may occur as a result of future earthquakes” (FEMA 445, 2006).

This paper uses the performance based evaluation method as part of rehabilitation process based on rehabilitation guidelines in Iran, Guideline 360, (MPORG, 2007). These guidelines are quite similar to those of FEMA 356 and ASCE 41 for existing buildings. Nonlinear static analysis method is used for evaluation of the designed structures. In this method for each structure and for considered seismic level a target displacement is defined and the structural system and components are checked based on acceptance criteria defined for the expected performance objective. Lateral loads shall be applied to the mathematical model in proportion to the distribution of inertia forces in the plane of each floor diaphragm. For all analyses, at least two vertical distributions of lateral load shall be applied. There are three main types of lateral load patterns introduced in FEMA356 to push structures through pushover analysis:

1. A uniform distribution consisting of lateral forces at each level proportional to the total mass at each level, as shown in equation 2.14 :

$$S_j = \frac{m_j}{\sum_{i=1}^{N} m_i} \tag{2.14}$$

2. Equivalent lateral load that applies when more than 75% of the total mass participates in the fundamental mode in the direction under consideration as shown in equation 2.15:

$$S_j = \frac{m_j {h_j}^k}{\sum_{i=1}^{N} m_i {h_i}^k} \tag{2.15}$$

3. A vertical distribution proportional to the story shear distribution calculated by combining modal responses from a response spectrum analysis of the building, including sufficient modes to capture at least 90% of the total building mass, and using the appropriate ground motion spectrum with SRSS combination method 2.16:

$$f_j = \frac{f_j}{\sum_{i=1}^{N} f_i} \tag{2.16}$$

In every step lateral load increases until structure reaches target displacement or yield mechanism occur. Finally "Base shear- Lateral displacement" curve, known as pushover curve, is plotted. In pushover analysis hinges are defined as a property of elements to model nonlinear behavior of elements. Figure 4 shows how the backbone curve is defined based on hysteretic cycles from test results for reinforced concrete or steel elements. These hysteretic cycles are the most important property for the elements which allows for dissipation of the earthquake energy. Figure 5 shows an idealized hinge definition.

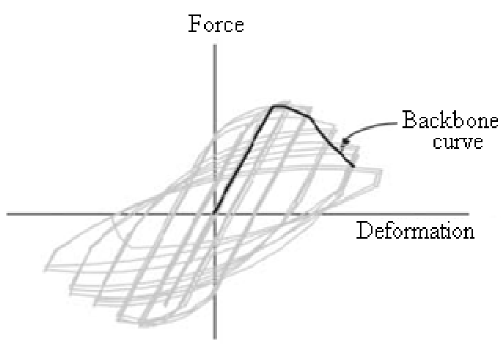


Figure 4. Hysteretic cycles of an element

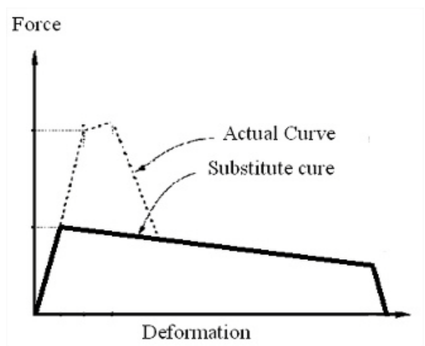


Figure 5. Idealized hinge

Final judgment about structural elements sufficiency is carried out based on nonlinear behavior of hinges and whether they pass acceptance criteria or not. Also pushover curve (capacity curve) of the structure is plotted by solving the matrix equation 2.17 as shown in Fig 6. In each step the stiffness matrix is revised based on developing nonlinearity in the system and equation 2.17 is solved for the structure.

$$[F] = [K].[\Delta] \qquad (2.17)$$

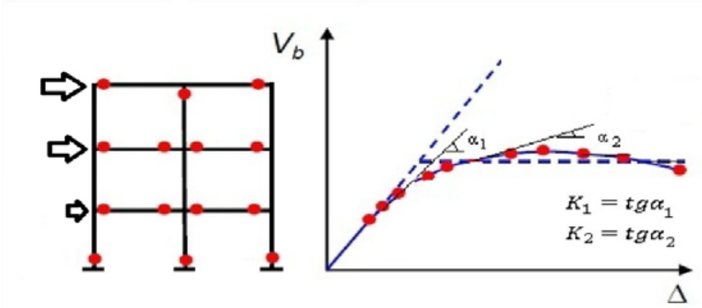


Figure 6. Hinge formation in elements and stiffness change for the system in pushover analysis

As indicated above the Iranian Guideline 360 is very similar to the FEMA356. Here we will focus on the provisions of these guidelines. Equation 2.18 shows how to calculate target displacement in FEMA356 or Guideline 360:

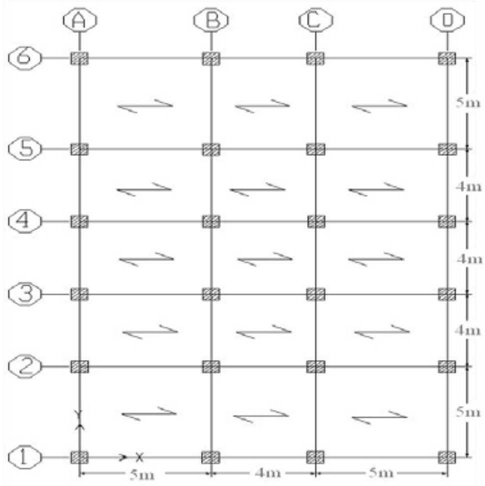
$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e}{4\pi^2} g \qquad (2.18)$$

Where:
δ_t = Target displacement at each floor level.
C_0= Modification factor to relate spectral displacement of an equivalent SDOF system to the roof displacement of the building MDOF system.
C_1= Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response.
C_2= Modification factor to represent the effect of pinched hysteretic shape, stiffness degradation and strength deterioration on maximum displacement response.
C_3= Modification factor to represent increased displacements due to dynamic P-Δ effects.
S_a= Response spectrum acceleration, at the effective fundamental period and damping ratio of the building in the direction under consideration.
T_e= Effective fundamental period of the building in the direction under consideration.
g = acceleration of gravity.

The methods rely on the assumption that the effects of higher modes on the response of the structure are negligible. Obviously this is not the case for all structures. Some improved methods have also been proposed to overcome this drawback such as multimodal or adaptive pushover analysis methods.

3. INTRODUCTION OF THE STRUCTURAL MODELS

Steel and reinforced concrete moment resisting frames with intermediate ductility level (according to Iranian seismic code, Standard 2800) are considered in this study to investigate the level of consistency of the design and rehabilitation provisions. The frames are part of a building with regular symmetric plan as shown in Fig 7. This building is designed with both RC as well as steel frames as its lateral load resisting systems, so that to present comparable results for both systems. Buildings with 5, 10 and 15 stories are considered in the study. Figure 8 shows elevation and plan view of the buildings.



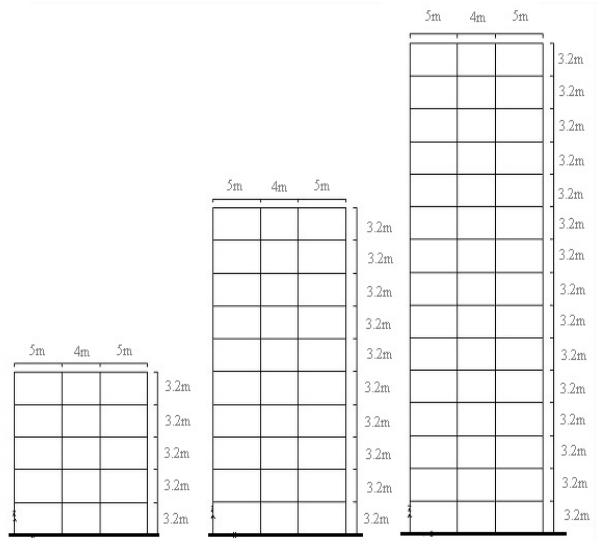


Figure 7. Plan view of buildings

Figure 8. Elevations and spans of buildings

The buildings are assumed to be located at a seismic zone with maximum base acceleration of 0.3g and with the soil type III. Important seismic parameters according to Iranian Standard 2800 are summarized in Table 1.

Table 1. Seismic properties for the structures

Important Factor(I)	1
Seismic Zone(A)	0.3g
Soil	III
Structural Behavior Factor(R)	7

The buildings are designed based on seismic loads form Iranian seismic Standard 2800 and using Iranian national regulations for designing steel and reinforced concrete as design codes. The elastic analyses were carried out using SAP2000 program. To perform nonlinear pushover analyses as part of evaluation and rehabilitation process two dimensional models of the frames in two directions were used for steel structures while for reinforced concrete structures three dimensional models are used. The assessment of the design structures are done assuming performance objective for rehabilitation assessments is the desired level, as defined in the Guidelines 360. This requires assessment of the building at two seismic hazard levels, namely BSE-1 and BSE-2. As BSE-2 is not explicitly defined in Guidelines 360, it is assumed that the seismic intensity at this level is equal to 1.5 times of that of BSE-1 level.

4. RESULT

Fig 9 shows schematic distribution of plastic hinges in 10 storey reinforced concrete frames, one frame form each longitudinal and transverse directions of the building. In figure 9, two frames of analyzed structures under different lateral load patterns to show schematic distribution of hinges in armed concrete elements.

Results for reinforced concrete structure indicate that in 5 storey building, no specific problem seen in beam and column elements. In other words no rehabilitation is required for any elements in the building. In 10 storey reinforced concrete building hinge's rotations under BSE-1 seismic level remain in acceptable range and no element fails. However under BSE-2 seismic level some hinges go beyond the collapse prevention (CP) acceptable limits and fail to satisfy collapse prevention criteria. Similar situation is seen for both BSE-1 and BSE-2 seismic levels for 15 stories reinforced concrete models.

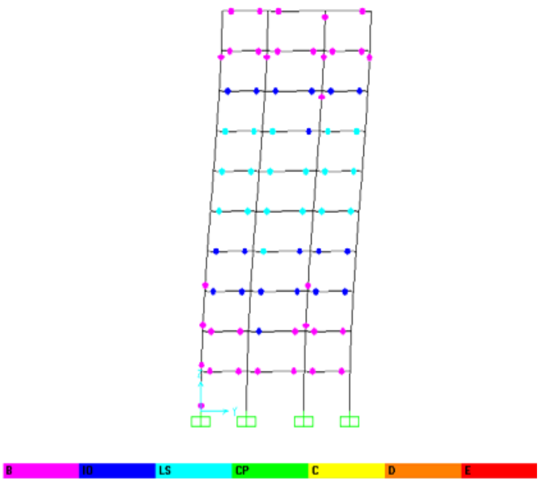


Figure 9. schematic distribution of hinges in RC frames in two principle directions of the 10 storey building

With regards to steel structures for 5 storey steel frame similar to 5 storey reinforced concrete frame there were no failure in elements. However for 10 and 15 storey steel buildings bottom columns failed to satisfy acceptance criteria especially at BSE-2 seismic level. Figures 10 to 13 show some pushover curves plotted for models in BSE-2 and for uniform and dynamic load patterns.

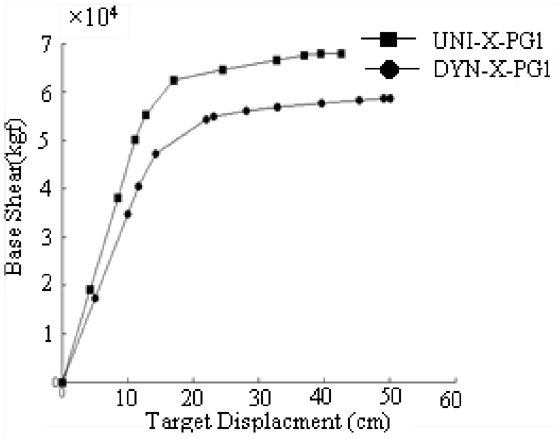


Figure10. Pushover curve for 5 stories-steel frame

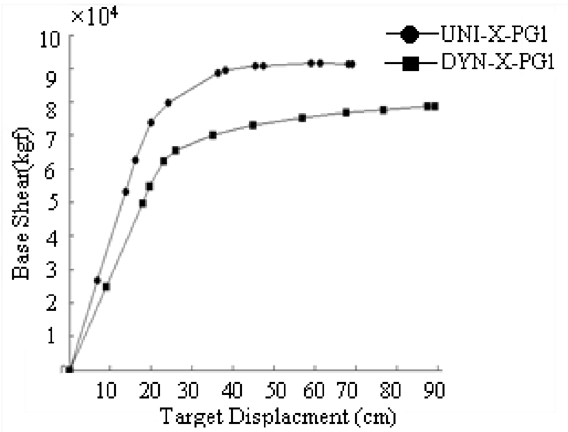


Figure11. Pushover curve for 5 stories RC frame

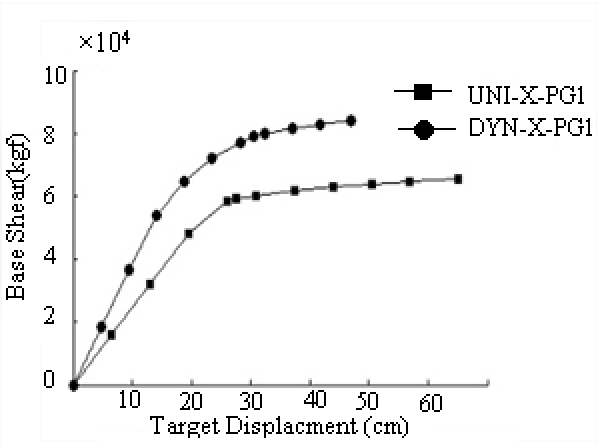


Figure12. Pushover curve for 15 stories-steel frame

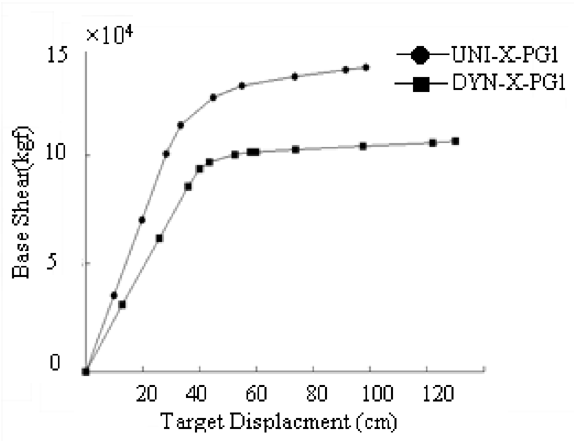


Figure13. Pushover curve for 10 stories RC frame

A selection of some important results is presented in tables 2 to 5. The tables show number of hinges in different ranges of the performance as defined by the rehabilitation guideline. Those with acceptable performance are shown in cells with grey color.

Table 2. Hinge results for 5 stories steel frame

BSE	load pattern	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeE	Total
1	DYN-X-PG1	40	8	22	0	0	0	0	0	70
	UNI-X-PG1	46	6	18	0	0	0	0	0	70
2	DYN-X-PG1	36	8	21	4	0	1	0	0	70
	UNI-X-PG1	43	2	22	2	0	1	0	0	70

Table 3. Hinge results for 10 stories steel frame

BSE	load pattern	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeE	Total
1	DYN-X-PG1	92	9	39	0	0	0	0	0	140
	UNI-X-PG1	102	4	33	0	0	1	0	0	140
2	DYN-X-PG1	78	8	46	6	0	0	0	0	138
	UNI-X-PG1	98	8	27	5	0	2	0	0	140

Table 4. Hinge results for 15 stories steel frame

BSE	load pattern	AtoB	BtoIO	IOtoLS	LStoCP	CPtoC	CtoD	DtoE	BeE	Total
1	DYN-X-PG1	118	10	68	0	0	0	0	0	196
	UNI-X-PG1	143	15	42	0	0	0	0	0	200
2	DYN-X-PG1	116	6	66	8	0	0	0	0	196
	UNI-X-PG1	133	6	43	14	0	0	0	0	196

Table 5. Hinge results for 10 stories reinforced concrete frame

BSE	load pattern	AtoB	BtoIO	IOtoLS	LstoCP	CptoC	CtoD	DtoE	BeE	Total
1	DYN-X-PG1	883	190	167	0	0	0	0	0	1240
	UNI-X-PG1	977	196	67	0	0	0	0	0	1240
2	DYN-Y-PG2	791	222	126	95	0	6	0	0	1240
	UNI-X-PG2	897	135	157	47	0	4	0	0	1240

As it can be seen in tables there are some hinges which are deformed beyond the acceptable limits in BSE-1 and BSE-2 and these results emphasis the fact that these structures fail to satisfy rehabilitation objectives.

CONCLUSION

1. There were considerable differences between analysis results of models in BSE-1 and BSE-2 seismic force levels. As indicated earlier in this research the ratio between these two levels assumed to be 1.5. However for rational judgments of the relative performance it seems necessary to provide a more explicit definition for BSE-2 level within the rehabilitation guidelines.
2. As the height of steel frames rise, columns located in lower stories show force controlled behavior due to increase in P/P_{CL} ratio to 0.5, which according to existing provisions in Guideline 360, they do not satisfy basic safety objective acceptance criteria.

3. In reinforced concrete models either in BSE-1 and BSE-2 seismic levels, columns showed acceptable behavior but beam elements, especially in mid height and lower stories went beyond basic safety objective acceptance criteria. Accordingly it appears to be necessary to introduce some modifications in Guideline 360 provisions to achieve better consistency with design provisions.
4. Columns in reinforced concrete models have better performance than beams and the acceptance criteria is not satisfied almost only in beam elements. However in steel frame elements there are no failed hinges except in 15 storey frame at BSE-2 seismic level, in which the column hinges fail first.

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