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The Effect of Analysis Methods on the Response of Steel-Braced Frame Buildings for Seismic Retrofitting

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Abstract: In this study, steel-braced frame buildings which are designed according to 2800 Standard of Iran (3rd revision), will be evaluated by four main types of structural analysis (Linear Static, Linear Dynamic, Nonlinear Static and Nonlinear Dynamic Analyses) with regard to Seismic Rehabilitation Code for Existing Buildings in Iran (based on FEMA 273). The discrepancy of the results derived from these four types of analysis and also seismic performance of the buildings in both linear and nonlinear treatments will be analyzed. At first, Probabilistic Seismic Hazard Analysis (PSHA) for 2 hazard levels has been carried out at center of Tehran, then three 3D models including 3 common buildings (5, 10 and 15-story) have been selected and designed subjected to earthquake according to 2800 Standard. Following this, these three 3D models have been analyzed and controlled based on Seismic Rehabilitation Code for Existing Buildings. The selected rehabilitation goal for this research is Fair (Controlling Life Safety in Hazard Level 1 + Collapse Prevention in Hazard Level 2). According to the results of this research, the accuracy of linear analysis for evaluating bracing elements is very low, in evaluating columns the results of linear static analysis is much more acceptable than linear dynamic and nonlinear static analysis. Also, by increasing the number of stories the accuracy of nonlinear static analysis decreases.

Key words: Seismic retrofitting, steel-braced frame building, linear analysis, nonlinear analysis, seismic hazard analysis

INTRODUCTION

Seismic response of steel frame buildings has been analyzed under the cyclic load of earthquake due to strong ground motion during some previous studies (Fragiacomo *et al*., 2004). The results show that the strength is an insufficient criterion for seismic design because most of the structures in strong earthquakes are yielded and entered in the plastic area. Performance based design is a much more comprehensive design method in which the design criteria is based on performance goals. Performance goal can be regarded as fair and to the point criteria of seismic performance of structures such as lateral deformations, lateral displacements of story, element ductility and element loss index in comparison to specific criteria of earthquake hazard. In other words, with combination of earthquake level and building performance level a performance goal is formed (Grecca *et al*., 2004). The base of making building code according to performance design in 1992 by decision making group SEAOC was expanded in VISION 2000 (SEAOC, 1995) committee and it was ordered to do this job before 2000 but nothing special was done except some limited activities. The cause of forming this committee was the 8 billion dollar loss from Loma Prieta earthquake in 1989. In Northridge earthquake in January 1994 with magnitude of 6.7 Richter around 20 billion dollar loss occurred. Following this accident during one year, VISION 2000 committee gave some suggestions on the performance based design. The report of the committee was published in 1995 which included full earthquake engineering problems in the field of performance based design (Bertero, 1995). In 1997 Bertero (1997) reexamined the instructions of SEAOC for new buildings and NEHRP for seismic retrofitting of existing building (FEMA, 273, 1997) Therefore a primary source was prepared in relation to performance based design which included suggestions and guidelines for designing and retrofitting of buildings.

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MATERIALS AND METHODS

To do this research, at first Probabilistic Seismic Hazard Analysis (PSHA) in 2 hazard levels has been done in the center of Tehran. Following this, three 3D models including three common 5, 10, 15-story buildings were selected and were designed and typically formed based on 2800 Standard (Standard No. 2800, 2005) using ETABS software. The selected rehabilitation goal used for the controlling of these buildings is fair according to Seismic Rehabilitation Code for Existing Buildings in Iran (IIEES, 2002) (which is based on FEMA reports). This has been done using the four main analysis methods (Linear Static, Linear Dynamic, Nonlinear Static and Nonlinear Dynamic Analyses) in SAP2000 software. Finally the results have been summarized and concluded.

DIFFERENT METHODS OF STRUCTURAL ANALYSIS

There are four types of analysis in Seismic Rehabilitation Code for Existing Buildings in Iran (IIEES, 2002) which are as followings:

- Linear static analysis
- Linear dynamic analysis
- Nonlinear static analysis
- Nonlinear dynamic analysis

Linear static analysis: In this method, the Pseudo lateral load of earthquake is selected in a way that its base shear is equal to the base shear according to Eq. 1. The amount of base shear in this method is selected in a way to have the maximum deformation of structure with the predicted hazard level earthquake.

$$V = C_1C_2C_3C_mS_aW \quad (1)$$

Where:
W = Total dead load and anticipated live load
S_a = Spectral response acceleration, at the fundamental period and damping ratio of the building in the direction under consideration
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 stiffness degradation and strength deterioration on maximum displacement response
C_3 = Modification factor to represent increased displacements due to dynamic P-Δ effects
C_m = Modification factor to have impact for higher modes

Distribution of lateral force on building height based on the base shear force, height and weight of the stories are:

$$F_i = \frac{W_i h_i^k}{\sum_{j=1}^{n} W_j h_j^k}.V \quad (2)$$

In which, Where, F_i is the force on the story i, W weight of the story i and h height of the story i from the base level and the amount of K equals to:

$$K = 0.5T+0.75$$

Where:
K = 1.0 for T ≤ 0.5 sec
K = 2.0 for T ≥ 2.5 sec

Where:
T = The fundamental period of the building in the direction under consideration

Linear dynamic analysis: Linear dynamic analysis can be done with two methods; response spectrum or time-history analysis. Special assumptions of this method in the limit of linear behavior are:

- Structural behavior can be calculated with a linear combination from different vibrational modes of structure which are independent of each other
- Period of structure in each mode is constant during earthquake

As mentioned before response spectrum method has been used in this research. The amount of vibration modes in response spectrum method should be selected in a way that the total percent of contribution of effective mass for each direction excitation in selected modes be at least 90%. In addition in each direction at least three primary modes of vibration and all modes which have more than 0.4 sec time period should be considered. To do this analysis, obtained spectrum from probabilistic seismic hazard analysis was used.

Nonlinear static analysis: In this method, the mathematical model of the building is subjected to monotonically increasing lateral forces or displacements until either a target displacement (Eq. 4) is exceeded or the building collapses (for structures with rigid diaphragms) (IIEES, 2002).

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

Where:

T_e = The effective fundamental period in the direction under consideration

S_a = Spectral response acceleration, at the fundamental period and damping ratio of the building

C_0 = Modification factor to relate spectral displacement and likely building roof displacement

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 hysteretic shape on the maximum displacement response

C_3 = Modification factor to represent increased displacements due to dynamic P-Δ effects

In this research 2 types of lateral load distribution were used on structures (IIEES, 2002):

- **Distribution type I:** Distribution corresponding to lateral forces derived from linear spectrum dynamic analysis.
- **Distribution type II:** Uniform distribution, in which lateral forces is calculated corresponding to the mass distribution at each floor level, like Eq. 5:

$$F_i = \frac{W_i}{\sum_{j=1}^{n} W_j}.V \quad (5)$$

Where:

F_i = The force on level i

W_i = Weight of level i

V = Base shear force

Nonlinear dynamic analysis: In this method, the structure response is calculated regarding nonlinear behavior material and geometrically nonlinear behavior of structures. In this method, it is supposed that stiffness and damping matrix can be changed from one step to another but is constant in each time step. The response of model under the earthquake acceleration is calculated using numerical method.

Nonlinear dynamic analysis is the most accurate method which is used for the structural analysis. In fact the main goal in this method is to solve differential equation of dynamic equilibrium of motion (Eq. 6). Nonlinear dynamic analysis is done with 2 general methods of Direct Integration and Modal Analysis (Bathe, 1996). Direct Integration includes different methods such as Houbolt, Central Difference, Wilson θ and Newmark. In this research Direct Integration Method (Wilson θ and Newmark) has been used.

$$Ku(t) + C\dot{u}(t) + M\ddot{u}(t) = r(t) \quad (6)$$

Where:

K, C, M = Stiffness, damping and mass matrixes, respectively

$u, \dot{u}, \ddot{u}$ = Displacement, velocity and acceleration vectors, respectively

r(t) = External force vector (Clough and Penzin, 1993)

STUDIED MODELS

Three symmetric and regular 5, 10, 15 story steel-braced buildings have been selected. The ratio of their height to width varies from 1.5 to 3 and is regarded as common buildings. It is good to note that these three models are 3D and all the processes of analysis, design and evaluation are done using these 3D models. For each model:

- Bay width for each direction is 4 m
- The height of first story is 3.8 m and the rest are 3.2 m
- Cross brace system is used (Because of wide usage)

The type of building is residential with average importance located in the center of Tehran. In all models the resistance system against lateral loads in both directions are braced frame. In order to tolerate the gravity loads of the stories, one-way slab system is used for floors. Plans and 3D elevations of the buildings under study are shown in Fig. 1 and 2, respectively:

MATERIAL SPECIFICATIONS AND ELEMENT SECTIONS

Specification of the material is stated as:

F_y = 235 Mpa, F_u = 392 Mpa, E = 2* 10^5 Mpa, ν = 0.3

Box and 2IPE, IPE and Box sections, according to DIN Standard, are chosen for columns, beams and bracings, respectively.

DESIGNING AND ANALYSIS SOFTWARE

In order to make and design the assumed models, ETABS ver8.5.4 (Computers and Structures, Inc., 2004) has been used (the members of the all primary models

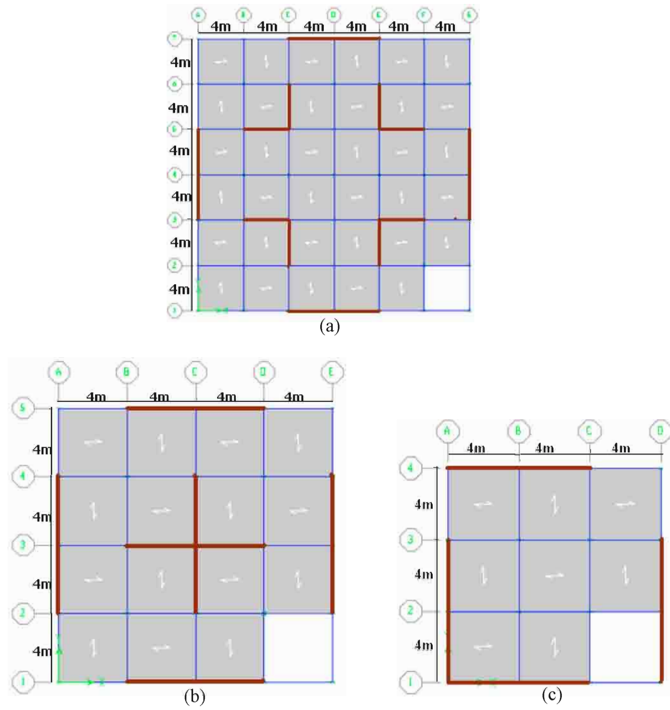

Fig. 1: Plans of studied buildings, a 5-story, b 10-story and c 15-story (thick line: bracing)

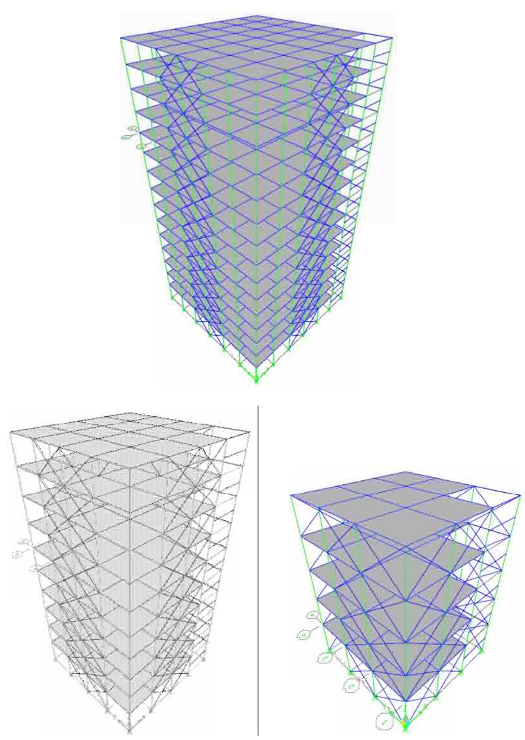

Fig. 2: 3D elevations of studied buildings

were typically formed after being designed). Then the models were transferred to SAP2000 ver9.1.6 (Computers and Structures, Inc., 2005) and the four mentioned analysis were done by this program.

LOADING AND DESIGNING BASED ON 2800 STANDARD

Gravity loading of mentioned buildings is based on National Building Code for Structural Loadings (Iranian, 2004) and the lateral loading is based on 2800 Standard (Standard No. 2800, 2005). Dead and live area loads and lateral wall loads in the stories are 700, 200 and 800 kgf m^{-2}, respectively and in the roof are 600, 150 and 250 kgf m^{-2}, respectively. To consider the effect of earthquake loading according to 2800 Standard (Standard No. 2800, 2005), static equivalent loading method is used. Seismic parameters values are mentioned below:

- Base Design Acceleration: A = 0.35 g
- Soil Type: Type II (T_{soil} = 0.5 sec)
- Importance Factor: I = 1

Design Code AISC-ASD89 which is supported by the aforementioned program has been used for designing members. Specific criteria for steel-braced framed buildings which are earthquake resistant according to 2800 Standard (Standard No. 2800, 2005) and Iranian National Building code (Iranian, National Building Code for Steel Structures, 2004) are stated as below:

- Controlling the least slenderness of bracing members
- Reduction of allowed compressive stress in bracing members
- Controlling of columns in load combinations stated below

a Axial pressure $P_{DL} + 0.8\ P_{LL} + 2.8\ P_E \leq P_{SC}$
b Axial tension $0.85\ P_{DL} + 2.8\ P_E \leq P_{ST}$

PROBABILISTIC SEISMIC HAZARD ANALYSIS (PSHA)

As mentioned earlier, probabilistic seismic hazard analysis in center of Tehran in two hazard levels 1 and 2 has been done. This caused seismic evaluation of buildings to occur for these 2 hazard levels. Hazard level 1 is determined based on 10% earthquake probability of accedence in 50 years (return period = 475 years). Hazard level 2 is determined based on 2% earthquake probability of accedence in 50 years (return period = 2475 years). In Fig. 3 the obtained design spectra are shown.

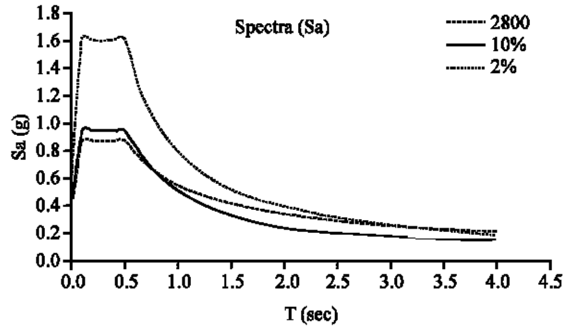

Fig. 3: Design spectra based on PSHA (Hazard levels 1 and 2) and 2800 standard

APPROPRIATE ACCELEROGRAMS AND SCALING PROCESS

Selecting appropriate accelerograms: In this research 7 accelerograms (Table 1) have been used for the nonlinear dynamic analysis and as a result their average response value can be used to control the deformations and internal forces. The accelerograms which are used for nonlinear dynamic analysis should have at least matching specifications with the site of the structure. These specifications include PGA, frequency contents, duration and harmony with design spectra (Lestuzzi *et al*., 2004). In order to use the accelerograms in nonlinear dynamic analysis, the spectrum of this accelerogram should be as much as possible in harmony with design spectrum of the site. In fact before using the accelerograms, they should be scaled.

Scaling accelerograms: In this research by using spectrum scaling method, accelerograms have been scaled. In this method the maximum acceleration of each accelerograms is scaled to 1 g. Then the response of Single-Degree-Of-Freedom (SDOF) system is calculated versus these records. Area under this spectrum is obtained between periods of 0.1 and 3 sec. The area under site spectrum curve between the two periods is calculated. By multiplying scaled accelerogram to 1 g by the ratio of site spectrum area over accelerogram spectrum area and finally by site design acceleration, the scaled accelerogram is obtained. In this method the energy of accelerograms is harmonized with design spectrum (Lestuzzi *et al*., 2004).

DISCUSSION

Designed models using 2800 Standard (Standard No. 2800, 2005), are analyzed based on Seismic Rehabilitation Code for Existing Buildings (IIEES, 2002), using four

Table 1: Accelerograms and the utilized scale coefficients in nonlinear dynamic analysis

Record No.	Name	Date	Station	PGA (g)	Area under normalized spectrum (T = 0.1-3.0)	Scale factor Hazard level 1	Scale factor Hazard level 2
1	CAPEMENDOCINO	1992	CAPEMEND-RIO270	0.385	2.67	1.30	1.34
2	KOCAELI	1999	KOCAEILI-SKR090	0.376	2.46	1.41	1.45
3	KOBE	1995	KOBE-KJM000	0.821	3.11	1.11	1.15
4	NORTHRIDGE	1994	NORTHT-OPPR360	0.514	3.06	1.13	1.17
5	SUPERSTITION HILLS	1987	SUPERST-B-PTS315	0.377	3.11	1.11	1.15
6	LOMA PRIETA	1989	LOMAP-CLS090	0.479	2.68	1.29	1.33
7	N. PALM SPRINGS	1985	PALMSP-NPS210	0.594	2.39	1.45	1.49

Table 2: Loading details according to linear static method

Earthquake force evaluation-linear static procedure

Building	5-Story		10-Story		15-Story	
Period	T = 0.51 sec		T = 0.87 sec		T = 1.15 sec	
Hazard level	HL 1	HL 2	HL 1	HL 2	HL 1	HL 2
Story Force	$f_{i\,(ton)}$	$f_{i\,(ton)}$	$f_{i\,(ton)}$	$f_{i\,(ton)}$	$f_{i\,(ton)}$	$f_{i\,(ton)}$
15	-	-	-	-	531.03	848.90
14	-	-	-	-	545.02	871.27
13	-	-	-	-	492.73	787.67
12	-	-	-	-	445.97	712.92
11	-	-	-	-	400.31	639.93
10	-	-	287.62	459.59	355.68	568.59
9	-	-	293.21	468.51	312.13	498.96
8	-	-	257.27	411.09	269.67	431.09
7	-	-	221.74	354.31	228.21	364.81
6	-	-	186.95	298.72	187.91	300.39
5	221.41	364.34	152.87	244.27	149.47	238.94
4	213.39	351.15	119.75	191.34	113.07	180.76
3	163.14	268.46	87.59	139.97	79.16	126.54
2	112.28	184.76	56.45	90.19	48.33	77.26
1	61.38	101.00	27.67	44.21	21.93	35.06

Table 3: The results of linear static analysis, percentage of the members which do not satisfy the acceptance criteria

	HL 1		HL 2	
Building	Bracing	Column	Bracing	Column
5-story	0	34	0	51
10-story	0	38	0	56
15-story	0	39	0	55

methods including Linear Static, Linear Dynamic, Nonlinear Static and Nonlinear Dynamic procedures. The selected rehabilitation goal for this research is fair (Life Safety in Hazard Level 1 + Collapse Prevention in Hazard Level 2). In nonlinear static analysis two types of load distributions (Types I and II) are implemented on the structures. In linear dynamic analysis, the spectrum method and in nonlinear dynamic analysis, the time-history method has been used.

Linear static method: Assumed models with the shown forces in Table 2 are loaded and then evaluated. Acceptance criteria were implemented according to Seismic Rehabilitation Code for Existing Buildings (IIEES, 2002), the summary is stated as:

- Deformation-controlled actions in primary and secondary components and elements shall satisfy the equation:

$$m.k.Q_{CE} \geq Q_{UD}$$

- Force-controlled actions in primary and secondary components and elements shall satisfy the equation:

$$k.Q_{CL} \geq Q_{UF}$$

General assumptions are used in evaluating all models which are stated as:

- Knowledge factor: K = 1
- Rehabilitation goal: Fair

The results of this evaluation are shown in Table 3. According to this method all bracing members (deformation-controlled) have satisfied the acceptance criteria but lack of acceptance of this criteria is visible in some percentage of columns (force-controlled).

Table 4: The values of parameters used in linear dynamic analysis

	Building						
	5-Story		10-Story		15-Story		
	Hazard level		Hazard level		Hazard level		
Parameters	HL 1	HL 2	HL 1	HL 2	HL 1	HL 2	Note
C1	1.00	1.00	1.00	1.00	1.00	1.00	$1+\frac{T_0-T}{2T_0-0.2}$
C2	1.00	1.00	1.00	1.00	1.00	1.00	Linear analysis
C3	1.00	1.00	1.00	1.00	1.00	1.00	$\theta<0.1$
T0	0.50	0.50	0.50	0.50	0.50	0.50	Soil type II
T	0.51		0.87		1.15		2800 standard
$T_{dynamic}$	0.54		1.02		1.72		Modal analysis

Table 5: The results of linear dynamic analysis, percentage of the members which do not satisfy the acceptance criteria.

	HL 1		HL 2	
Building	Bracing	Column	Bracing	Column
5-story	0	11	0	66
10-story	0	1	0	31
15-story	0	4	0	34

Table 6: Needed parameters for nonlinear static analysis, distribution type I

	Building						
	5-Story		10-Story		15-Story		
	Hazard level		Hazard level		Hazard level		
Parameters	HL 1	HL 2	HL 1	HL 2	HL 1	HL 2	Note
C0	1.30	1.30	1.30	1.30	1.30	1.30	Distribution type I
C1	1.00	1.00	1.00	1.00	1.00	1.00	Te>T0
C2	1.10	1.20	1.10	1.20	1.10	1.20	LS-->C2=1.1 CP-->C2=1.2
C3	1.00	1.00	1.00	1.00	1.00	1.00	$\alpha>0$
Sa (g)	0.945	1.555	0.577	0.922	0.436	0.697	Spectrum
T0 (sec)	0.50		0.50		0.50		Soil type II
Te (sec)	0.51		0.87		1.15		Experimental
δ_t (cm)	8.74	15.69	15.54	24.82	20.51	35.77	$\delta_t = C_0C_1C_2C_3S_a\frac{T_e^2}{4\pi^2}g$

Linear dynamic method: Assumed models were analyzed and evaluated with spectrum obtained from PSHA. The values of parameters used in linear dynamic analysis are shown in Table 4. Acceptance criteria are implemented according to Clause (10-1).

The results of this evaluation are shown in Table 5. According to this method all bracing members (deformation-controlled) have satisfied the acceptance criteria but lack of acceptance of this criteria is visible in some percentage of columns (force-controlled).

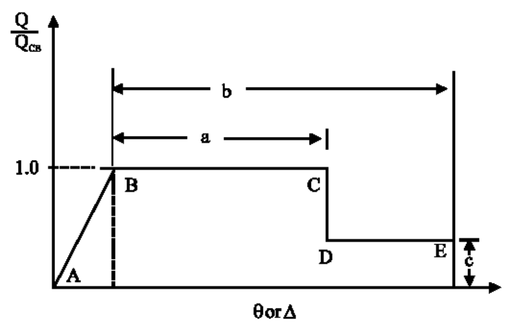

Fig. 4: Generalized force-deformation relation for steel elements or components (FEMA 356, 2000)

Nonlinear static method: Assumed models were analyzed and evaluated by nonlinear static method (Target Displacement Method). In Table 6 and 7 needed parameters for nonlinear static analysis are shown.

For modeling the stiffness of members in nonlinear static method, the principles of Seismic Rehabilitation Code for Existing Buildings (IIEES, 2002) are used. For modeling force-deformation curve of members which is shown in Fig. 4, values of a, b, c shown in Table 8 are used. Strain-hardening of components is accounted according to the slope of 3% of the elastic slope.

The results of this evaluation are shown in Table 9. As it is shown, some percentage of bracing members

Table 7: Needed parameters for nonlinear static analysis, distribution type II

	Building						
	5-Story		10-Story		15-Story		
	Hazard level		Hazard level		Hazard level		
Parameters	HL 1	HL 2	HL 1	HL 2	HL 1	HL 2	Note
C0	1.20	1.20	1.20	1.20	1.20	1.20	Distribution type II
C1	1.00	1.00	1.00	1.00	1.00	1.00	Te>T0
C2	1.10	1.20	1.10	1.20	1.10	1.20	LS-->C2=1.1 CP-->C2=1.2
C3	1.00	1.00	1.00	1.00	1.00	1.00	$\alpha>0$
Sa (g)	0.945	1.555	0.577	0.922	0.436	0.697	Spectrum
T0 (sec)	0.50		0.50		0.50		Soil type II
Te (sec)	0.51		0.87		1.15		Experimental
δ_t (cm)	8.07	14.49	14.34	22.91	18.93	33.02	$\delta_t = C_0C_1C_2C_3S_a\frac{T_e^2}{4\pi^2}g$

Table 8: Modeling parameters and acceptance criteria of bracing members in nonlinear static analysis

		Nonlinear Hinge parameters											
		Compression						Tension					
Section (Box)	d/t	a	b	c	IO	LS	CP	a	b	C	IO	LS	CP
80×80×5	16.00	0.5	7.00	0.40	0.25	3.00	4.00	11	14	0.8	0.25	7	9
90×90×5	18.00	0.5	6.37	0.36	0.25	2.68	3.61	11	14	0.8	0.25	7	9
100×100×8	12.50	0.5	7.00	0.40	0.25	3.00	4.00	11	14	0.8	0.25	7	9
120×120×8	15.00	0.5	7.00	0.40	0.25	3.00	4.00	11	14	0.8	0.25	7	9
140×140×10	14.00	0.5	7.00	0.40	0.25	3.00	4.00	11	14	0.8	0.25	7	9
160×160×10	16.00	0.5	7.00	0.40	0.25	3.00	4.00	11	14	0.8	0.25	7	9
180×180×16	11.25	0.5	7.00	0.40	0.25	3.00	4.00	11	14	0.8	0.25	7	9
200×200×17.5	11.43	0.5	7.00	0.40	0.25	3.00	4.00	11	14	0.8	0.25	7	9
220×220×20	11.00	0.5	7.00	0.40	0.25	3.00	4.00	11	14	0.8	0.25	7	9

Table 9: The results of nonlinear static analysis, percentage of the members which do not satisfy the acceptance criteria

	HL 1				HL 2			
	Type I		Type II		Type I		Type II	
Building	Bracing	Column	Bracing	Column	Bracing	Column	Bracing	Column
5-Story	50	68	70	52	Instability	Instability	Instability	Instability
10-Story	18	20	30	14	68	52	46	22
15-Story	2.6	3.4	0.8	2	32	28	30	22

Table 10 The results of nonlinear dynamic analysis, percentage of the members which do not satisfy the acceptance criteria

		HL 1							HL 2						
Building	Member	Cape mendocino	Kobe	Kocaeli	Loma prieta	Northridge	Palm springs	Superstition hills	Cape mendocino	Kobe	Kocaeli	Loma prieta	Northridge	Palm springs	Superstition hills
5-story	Bracing	43	50	20	Instability	Instability	18	Instability	Instability	78	Instability	Instability	Instability	Instability	Instability
	Column	65	79	46	Instability	Instability	46	Instability	Instability	80	Instability	Instability	Instability	Instability	Instability
10-story	Bracing	65	70	58	56	63	69	61	93	98	74	89	88	77	instability
	Column	66	71	39	47	64	55	53	73	75	52	68	64	57	instability
15-story	Bracing	27	25	43	18	Instability	34	23	Instability	Instability	60	Instability	Instability	Instability	Instability
	Column	32	29	44	27	Instability	42	39	Instability	Instability	51	Instability	Instability	Instability	Instability

(deformation-controlled) and columns (force-controlled) have not satisfied acceptance criteria. Also 5-story building in Hazard Level 2 experienced instability.

Nonlinear dynamic method: Assumed models were analyzed and evaluated using seven mentioned accelerograms and direct integration method. Description and attribution of nonlinear hinges of bracing members is like Table 8. The results of this evaluation are shown in Table 10. As it is visible, some percentage of bracing members (deformation-controlled) and columns (force-controlled) have not satisfied the acceptance criteria. Instability of different buildings against some earthquakes is visible in the both hazard levels.

CLASSIFICATION OF RESULTS

To reach a better understanding and also integration of analysis, the results of 4 types of analyses are presented for comparison in a curve form from Fig. 5-8.

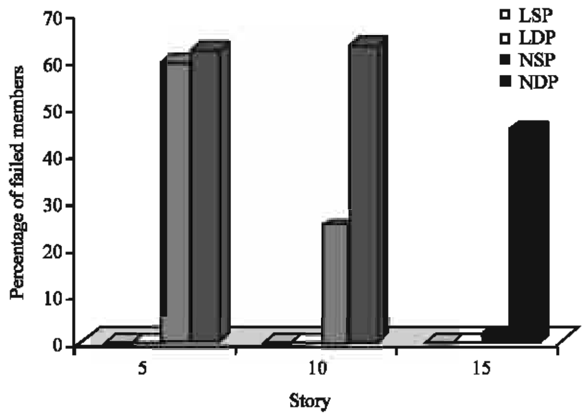

Fig. 5: Comparison of results accuracy obtained from 4 types of analyses, bracing members (HL1)

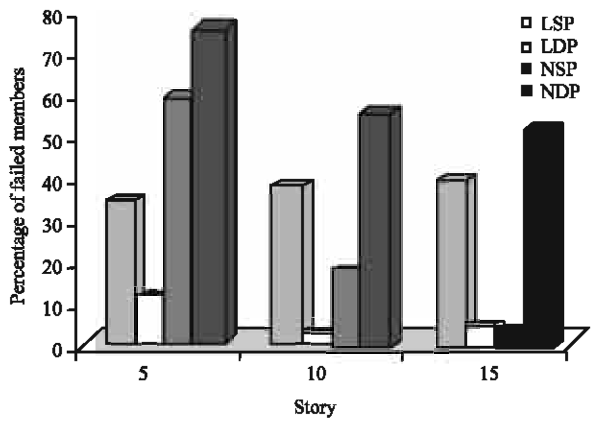

Fig. 6: Comparison of results accuracy obtained from 4 types of analyses, columns (HL1)

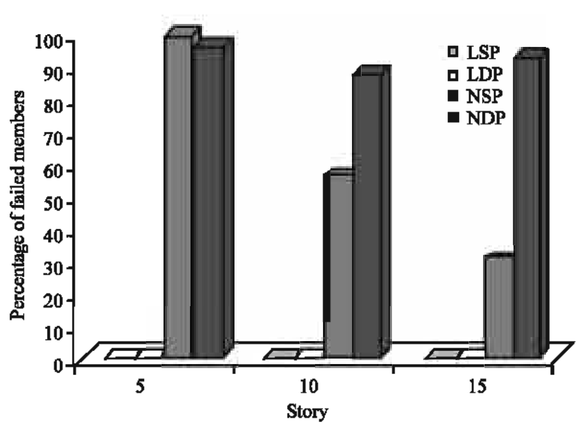

Fig. 7: Comparison of results accuracy obtained from 4 types of analyses, bracing members (HL2)

In these curves:
LSP = Shows linear static analysis
LDP = Shows linear dynamic analysis

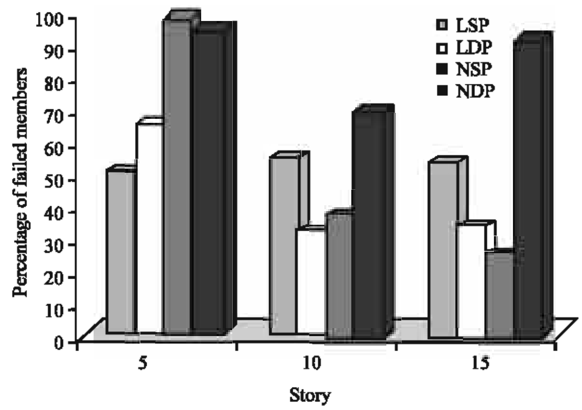

Fig. 8: Comparison of results accuracy obtained from 4 types of analyses, columns (HL2)

NSP = Shows nonlinear static analysis
NDP = Shows nonlinear dynamic analysis

And vertical axis shows the percentage of members which have not satisfied acceptance criteria (failed members).

CONCLUSION

The accuracy of linear analysis (static and dynamic) in evaluation of the bracings is very low and not reliable.

In evaluation of the columns, the results of linear static analysis is closer to reality than linear dynamic and nonlinear static analyses.

In general, the results of nonlinear static analysis have more accuracy and are more reliable than linear static and linear dynamic analysis.

Nonlinear static analysis in the evaluation of 15-story building has less accuracy than 5 and 10-story buildings. (This may be due to lack of contribution of higher modes effects in load distribution pattern used in nonlinear static method. Because in tall buildings higher modes have substantial effect, it is recommended that for tall buildings, MPA method (Chopra and Goel, 2002) be used in nonlinear static analysis.)

According to nonlinear dynamic analysis for the models designed based on 2800 Standard, the following results have been obtained:

5-story building
Hazard level 1: More than 65% of members do not satisfy the acceptance criteria.
Hazard level 2: The structure experienced instability.

10-Story building
Hazard level 1: Around 60% of members do not satisfy the acceptance criteria.

Hazard level 2: Around 80% of members do not satisfy the acceptance criteria.

15-Story building:
Hazard level 1: Around 45% of members do not satisfy the acceptance criteria.
Hazard level 2: The structure experienced instability.

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NOTATIONS

C = Damping Matrix
C_0 = Modification factor to relate spectral displacement of an equivalent SDOF system to the roof displacement of the building MDOF system calculated
C_1 = Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response
C_2 = Modification factor to represent the effects of pinched hysteretic shape, stiffness degradation and strength deterioration on the maximum displacement response
C_3 = Modification factor to represent increased displacements due to P-Δ effects
C_m = Effective mass factor to account for higher mode mass participation effects
E = Modulus of elasticity
F_i = Lateral load applied at floor level i
F_y = Yield strength of the material
F_u = Tensile strength of the material
K = Stiffness matrix
M = Mass matrix
P_{DL} = Axial force in member, due to dead load,
P_E = Axial force in member, due to earthquake
P_{LL} = Axial force in member, due to live load
P_{SC} = Column axial load capacity, compression
P_{ST} = Column axial load capacity, tension
Q = Generalized force in a component
Q_{CE} = Expected strength of the component or element at the deformation level under consideration for deformation-controlled actions
Q_{CL} = Lower-bound strength of a component or element at the deformation level under consideration for force-controlled actions
Q_{UD} = Deformation-controlled design action due to gravity loads and earthquake loads
Q_{UF} = Force-controlled design action due to gravity loads in combination with earthquake loads
S_a = Spectral response acceleration (g)
T = Fundamental period of the building in the direction under consideration
T_0 = Period at which the constant acceleration region of the design response spectrum transitions to the constant velocity region
T_e = Effective fundamental period of the building in the direction under consideration
V = Pseudo lateral load
W = Effective seismic weight of a building including total dead load and applicable portions of other gravity loads
h_i = Height from the base to floor level i
h_j = Height from the base to floor level j
g = Acceleration of gravity
k = Knowledge factor
m = Component or element demand modifier (factor) to account for expected ductility associated with this action at the selected Structural Performance Level
$r(t)$ = External forces vector
$u(t)$ = Displacement vector
$\dot{u}(t)$ = Velocity vector
$\ddot{u}(t)$ = Acceleration vector
w_i = Portion of the effective seismic weight W located on or assigned to floor level i
w_j = Portion of the effective seismic weight W located on or assigned to floor level j
Δ = Generalized deformation
θ = Generalized deformation, radians
δ_t = Target displacement
ν = Poisson's ratio

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