

# THE EFFECT OF ANALYSIS METHODS ON THE RESPONSE OF STEEL DUAL-SYSTEM FRAME BUILDINGS FOR SEISMIC RETROFITTING

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**Abstract** In the present paper, the focus is on the evaluation of steel dual-system frame buildings using 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 and 356) where the first two authors of the article tend to follow the previous work (Section 2). The difference of the results taken from these four types of analyses and also seismic performance of the dual-system buildings will be studied in both linear and nonlinear treatments. Three 2D models which include 3 common dual-system buildings (5, 10 and 15-story) have been chosen and designed subjected to earthquake based on the Standard No. 2800 of Iran (3<sup>rd</sup> revision). Then, the 2D models have been analyzed and controlled according to "Seismic Rehabilitation Code for Existing Buildings". The selected rehabilitation goal for this research is Fair (Controlling Life Safety and Collapse Prevention in two hazard levels derived from PSHA analysis). Based on the research results, the main role of lateral load-bearing is on bracing members. The linear analysis of bracing members evaluation, has a low accuracy, in evaluation of columns the results derived from linear static analysis shows more accuracy than linear dynamic and nonlinear static analyses. Also, the accuracy of nonlinear static analysis decreases when the number of stories increases.

**Keywords** Seismic Retrofitting, Steel Dual-System Frame Building, Linear Analysis, Nonlinear Analysis, Seismic Hazard Analysis

**چکیده** در این مقاله، ساختمان‌های فولادی دوگانه، بر اساس دستورالعمل به‌سازی لرزه‌ای با استفاده از چهار نوع روش اصلی تحلیل سازه‌ها (استاتیکی خطی، دینامیکی خطی، استاتیکی غیرخطی و دینامیکی غیرخطی) مورد ارزیابی قرار گرفته، تفاوت پاسخ حاصل از هر یک از ۴ نوع تحلیل یاد شده و همچنین عملکرد لرزه‌ای سازه‌ها ناشی از آن‌ها در حوزه رفتارهای خطی و غیرخطی بررسی می‌گردد. بدین منظور سه مدل دو بعدی شامل سه ساختمان متعارف ۵، ۱۰ و ۱۵ طبقه انتخاب و طبق استاندارد ۲۸۰۰ (ویرایش سوم) طراحی شده و با استفاده از نتایج تحلیل ریسک احتمالاتی در دو سطح خطر در مرکز تهران، این سه مدل تحت هدف به‌سازی مطلوب با روش‌های یاد شده تحلیل و کنترل گردیدند. مطابق نتایج این تحقیق، نقش اصلی باربری لرزه‌ای بر عهده اعضای مهاربندی می‌باشد. دقت تحلیل‌های خطی در ارزیابی اعضای مهاربندی کم می‌باشد، در ارزیابی ستون‌ها نتایج تحلیل استاتیکی خطی نسبت به تحلیل‌های دینامیکی خطی و استاتیکی غیرخطی به واقعیت نزدیکتر می‌باشد. همچنین با افزایش تعداد طبقات ساختمان، از دقت تحلیل استاتیکی غیرخطی در ارزیابی‌ها کاسته می‌شود.

## 1. INTRODUCTION

Seismic retrofitting is the modification of existing

structures to make them more resistant to seismic activity, ground motion, or soil failure due to earthquake events. Moreover, performance based

earthquake engineering implies design, evaluation, construction, monitoring the function and maintenance of engineering facilities (Bozorgnia, et al [1]). The results of the studies done in advance in the field of seismic response of steel frame buildings under the cyclic load of earthquake (e.g., Fragiacom, et al [2]) showed that due to plastic behaviour of structures in strong ground motions, strength can not be regarded a sufficient criteria for seismic design. Thus, the idea of performance based design which has a more comprehensive concept than previous common methods has been formed. In this method the criteria of design is presented based on performance goals. Therefore, with combining earthquake level and building performance level, a performance goal can be formed (Grecca, et al [3]). The 8-billion dollar loss caused by Loma Prieta earthquake in 1989, prompted SEAOC decision making group (SEAOC [4]) to form the primary idea of making performance based design code in 1992. But nothing special (except some limited activities) was done in this field until Northridge earthquake with magnitude of 6.7 Richter (with its 20-billion dollar loss) showed the importance of the issue more than ever. Following this issue, VISION 2000 committee (SEAOC [4]) has announced a report (Bertero [5]) for performance based design in 1995. Therefore, Bertero (Bertero [6]) reexamined the SEAOC instruction for new buildings and NEHRP did the same for seismic retrofitting of existing buildings in 1997 (FEMA [7]). Finally, a main and primary source called FEMA 273 (its new revision named FEMA 356 [8]) was made available for engineers and consultants in relation to performance based design.

## 2. RESEARCH METHODOLOGY

The aim of this research is to investigate seismic response of steel dual-system frame buildings for seismic retrofitting following the previous work of the first two authors of this paper (Ghodrati, et al [9]). In this research, the 2D models which include three common 5, 10, 15-story dual-system buildings have been chosen, designed according to 2800 Standard (Standard No. 2800 [10]). Based on "Seismic Rehabilitation Code for Existing Buildings" (IIEES [11]), the selected rehabilitation

goal which is used for the buildings control is fair (see Section 10), where the spectra have been derived from Section 8. The process has been carried out with the help of four main methods of analysis (Linear Static, Linear Dynamic, Nonlinear Static and Nonlinear Dynamic Analyses) in SAP 2000 program. In the end, the results were summarized and concluded.

## 3. DIFFERENT METHODS OF STRUCTURAL ANALYSIS

Four different types of analysis are mentioned in "Seismic Rehabilitation Code for Existing Buildings" (IIEES [11]) which are as followings:

A. Linear Static Analysis, B. Nonlinear Static Analysis, C. Linear Dynamic Analysis, D. Nonlinear Dynamic Analysis.

A comprehensive description is described in FEMA 273 [7] and a brief description is presented for each:

**3.1. Linear Static Analysis** The pseudo lateral load of earthquake in linear static method is selected in a way that the base shear be equal to the base shear shown by Equation 1. In this method, the amount of the mentioned base shear is chosen in such a way that the maximum deformation of structure be in accordance with the predicted hazard level earthquake (IIEES [11]).

$$V = C_1 C_2 C_3 C_m S_a W \quad (1)$$

Lateral force distribution on building height according to the weight, height and base shear force of the stories are presented in Equation 2.

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

**3.2. Linear Dynamic Analysis** Linear dynamic analysis is done in two ways; 1-Response spectrum, 2-Time-history analysis. In the research, response spectrum method by the help of obtained spectra from probabilistic seismic hazard analysis (Section 8) has been used.

### 3.3. Nonlinear Static Analysis (Pushover)

Nonlinear static procedures (NSPs) are now widely used in engineering practice to predict seismic demands in building structures (Kalkan, et al [12]). Reliability and accuracy of this type of analysis has been verified by some researches (Moghadam, et al [13]), and also new methods have been developed gradually such as Modal Pushover Analysis (Chopra, et al [14]) and Adaptive Pushover Procedure (Antonio, et al [15]). For structures with rigid diaphragms, the mathematical based model of the building should undergo the monotonically increasing lateral forces or displacements until either a target displacement (Equation 3) is reached or the building collapses (IIES [11]).

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} g \quad (3)$$

As mentioned below, two types of lateral load distribution have been used on structures:

- Distribution Type I: Distribution corresponding to lateral forces derived from linear dynamic method (spectrum analysis).
- Distribution Type II: Uniform distribution (in which lateral forces is calculated corresponding to the mass distribution at each floor level).

**3.4. Nonlinear Dynamic Analysis** The most appropriate method which is used for the structural analysis is nonlinear dynamic procedure, even though as Elnashai states the “necessity domain” of nonlinear dynamic analysis as against static inelastic analysis is ever decreasing [16]. In this method, solving the differential equation of dynamic equilibrium of motion (Equation 4) is actually the main goal.

$$K u(t) + C \dot{u}(t) + M \ddot{u}(t) = r(t) \quad (4)$$

Where:  $M$ ,  $C$ ,  $K$  are mass, damping and stiffness matrixes, respectively.  $r(t)$  is external force vector.  $\ddot{u}$ ,  $\dot{u}$  and  $u$  are the acceleration, velocity and displacement vectors, respectively (Clough [17]).

Nonlinear dynamic analysis is done in two general methods: 1-Direct Integration, 2-Modal Analysis (Bathe [18]). Direct integration can be done by methods of Houbolt, Central Difference,

Wilson- $\theta$  and Newmark. Direct integration method (Wilson- $\theta$  and Newmark) has been used in the present research.

## 4. STUDIED MODELS

Three symmetric and regular 5, 10, 15-story dual-system buildings have been selected. The models are regarded as common building because the ratio of height to width varies from 1.5 to 3. For each model:

- \* The height of first story is 3.8 m and the rest are 3.2 m.
- \* Bay width for each direction is 4 m.
- \* Cross brace system (because of wide usage) + moment resistant frame is used.

The buildings are located in the center of Tehran. They are residential and have an average importance. The resistance system against lateral loads is braced frame + moment resistant frame in all models. The elevations of the buildings under study are shown in Figure 1.

## 5. MATERIAL SPECIFICATIONS AND ELEMENT SECTIONS

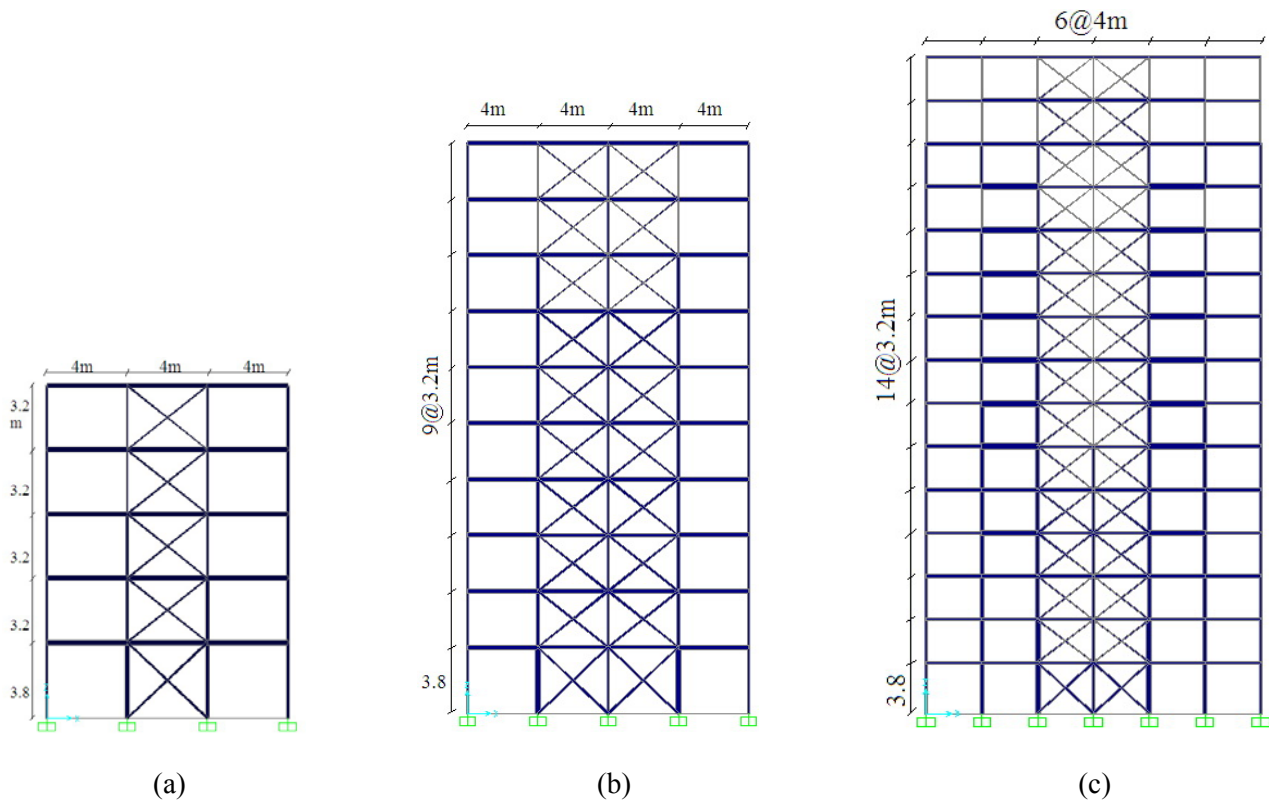
Material specifications are mentioned below:

$E = 2 * 10^5$  MPa,  $F_y = 235$  MPa,  $F_u = 392$  MPa,  $\nu = 0.3$ .

Box, IPE and Box sections, based on DIN Standard, have been selected for columns, beams and bracings, respectively.

## 6. DESIGNING AND ANALYSIS SOFTWARE

To model and design the assumed buildings, ETABS ver 8.5.4 (Computers and Structures, Inc. [19]) has been used. The models have been then taken to SAP2000 ver9.1.6 (Computers and Structures, Inc. [20]). Finally the four stated analyses were done with the help of this software.



**Figure 1.** Elevations of studied buildings, (a) 5-story (b) 10-story and (c) 15-story.

## 7. LOADING AND DESIGNING ACCORDING TO 2800 STANDARD

Gravity loading of the models is based on “National Building Code for Structural Loadings” (Standard No. 519 [21]). Accordingly the lateral loading is based on 2800 Standard (Standard No. 2800 [10]). Dead and live area loads calculated in the stories are 220 MPa/m, 40 MPa/m, respectively. The same loads in the roof are 145 MPa/m, 30 MPa/m, respectively. To evaluate the effect of earthquake lateral loading according to 2800 Standard (Standard No. 2800 [10]), static equivalent loading method has been used. Values of seismic parameters are stated below:

- \* Importance Factor:  $I = 1$
- \* Base Design Acceleration:  $A = 0.35 \text{ g}$
- \* Soil Type: Type II ( $T_{\text{soil}} = 0.5 \text{ s}$ )

Design Code AISC-ASD89 has been used for designing members. Specific criteria for steel dual-system frame buildings which are earthquake

resistant according to 2800 Standard (Standard No. 2800 [10]) and (National Building Code for Steel Structures [22]) are mentioned below:

- \* Reduction of allowed compressive stress in bracing members
- \* Controlling the least slenderness of bracing members
- \* Controlling of columns in load combinations mentioned below:

A. Axial pressure  $P_{DL} + 0.8P_{LL} + 2.8P_E \leq P_{SC}$

B. Axial tension  $0.85P_{DL} + 2.8P_E \leq P_{ST}$

## 8. PROBABILISTIC SEISMIC HAZARD ANALYSIS (PSHA)

A probabilistic seismic hazard analysis has been done previously (Ghodrati, et al [9]) in center of Tehran in two hazard levels (HL1 and HL2). For

this analysis (PSHA), appropriate attenuation relationships for Iranian plateau have been chosen (Ghodrati, et al [23]). The buildings have been evaluated in two hazard levels. Hazard level 1 is determined based on 10 % earthquake probability of exceedance in 50 years where the return period equals 475 years. Hazard level 2 is determined based on 2 % earthquake probability of exceedance in 50 years where the return period equals 2475 years. The obtained design spectra are shown in Figure 2.

## 9. APPROPRIATE ACCELEROGRAMS AND THE SCALING PROCESS

**9.1. Selecting Appropriate Accelerograms** In this study, 7 accelerograms (Table 1) similar to the previous work of the first two authors of this paper (Ghodrati, et al [9]) have been used for the nonlinear dynamic procedure, so their average response values have been used to control the deformations and internal forces. The accelerograms used for nonlinear dynamic procedure should have

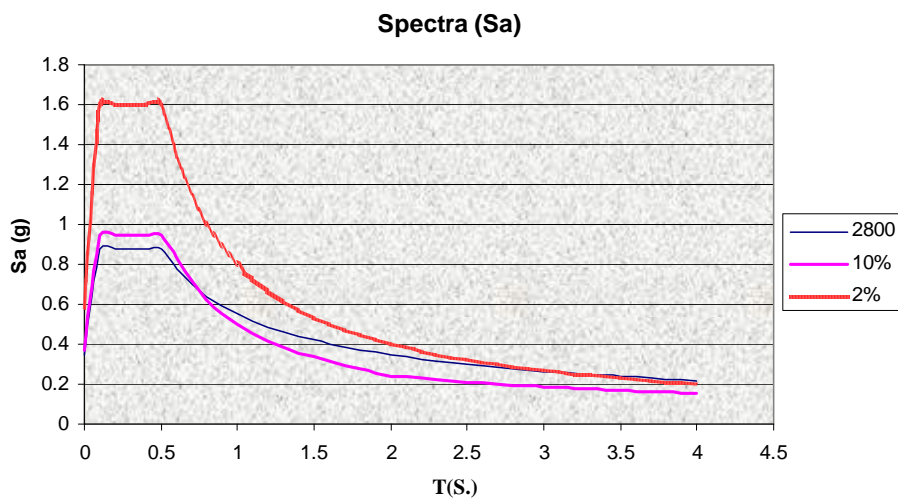


Figure 2. Design spectra based on PSHA and 2800 Standard (Ghodrati, et al [9]).

TABLE 1. Accelerograms and the Utilized Scale Factors (Nonlinear Dynamic Analysis) (Ghodrati, et al [9]).

Record No.	Record Name	Date	Station	PGA (g)	Area under Normalized Spectrum (T=0.1-3.0)	Scale Factor (HL 1)	Scale Factor (HL 2)
1	Capemendocino	1992	Capemend-rio270	0.385	2.67	1.3	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

matching specifications with the site of the structure. The mentioned specifications include PGA, duration, frequency contents and conformity with design spectra (Lestuzzi, et al [24]). To use the accelerograms in nonlinear dynamic analysis, the spectrum of this accelerogram should be in great conformity with design spectrum of the site. Therefore, they should be scaled before using the accelerograms.

**9.2. Scaling Accelerograms** These accelerograms have been scaled by using spectrum scaling method. In this method at first the maximum acceleration of each accelerograms is scaled to 1g (g=gravity acceleration). Then the response of SDOF system (single-degree-of-freedom) is calculated versus the specified records. The area under this spectrum is calculated between periods of 0.1 second and 3 seconds. Accordingly, the area under site spectrum curve between the same two periods is determined. The scaled accelerogram can be obtained by Equation 5:

$$A_{sc} = A_{ng} \times \frac{A_{1s}}{A_{2s}} \times (PGA)_s \quad (5)$$

Where  $A_{sc}$  is the scaled accelerogram,  $A_{ng}$  is the accelerogram normalized to 1g,  $A_{1s}$  is the site spectrum area,  $A_{2s}$  is the accelerogram spectrum area and  $(PGA)_s$  is the site design acceleration. The energy of accelerograms is conformed to design spectrum by using this method (Lestuzzi, et al [24]).

## 10. DISCUSSION

Designed models which are based on 2800 Standard (Standard No. 2800 [10]) have been analyzed according to “Seismic Rehabilitation Code for Existing Buildings” (IIEES [11]), using four main types of analyses which include Linear Static, Nonlinear Static, Linear Dynamic and Nonlinear Dynamic procedures. The selected rehabilitation goal for this study is Fair (Controlling Life Safety and Collapse Prevention in two hazard levels). In nonlinear static method, two different kinds of load distributions (Types I and II) are implemented on the models. Spectrum method and time-history method has been used in linear dynamic and

nonlinear dynamic analysis, respectively.

For Deformation-Controlled actions, design actions  $Q_{UD}$  shall be calculated based on:

$$Q_{UD} = Q_G \pm Q_E$$

$$Q_E = EQ$$

$$Q_G = DL + LL$$

and for Force-Controlled actions, design actions  $Q_{UF}$  shall be calculated based on:

$$Q_{UF} = Q_G \pm \frac{Q_E}{C_1 C_2 C_3 J}$$

$$Q_E = EQ, \quad (\text{IIEES [11]}).$$

$$Q_G = DL + LL$$

Where LL is the effective live load (action), equal to 0.25 of the unreduced design live load, but not less than the actual live load.

**10.1. Linear Static Procedure** The mentioned models with the forces presented in Table 2 have been loaded and then evaluated. Acceptance criteria were implemented based on “Seismic Rehabilitation Code for Existing Buildings” (IIEES [11]), a brief description is shown below:

1. Deformation-controlled actions should satisfy the following equation in primary and secondary components and elements:

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

2. Force-controlled actions should satisfy the following equation in primary and secondary components and elements:

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

Where  $Q_{CE} = P_{CE} = 1.7AF_{as}$  or  $Q_{CE} = T_{CE} = AF_{ye}$  for braces in compression or tension, respectively.

and

$$Q_{CE} = M_{CE} = Z.F_{ye} \text{ for beams.}$$

For columns, if  $\frac{P_{UF}}{P_{CL}} < 0.15$  then

$$\frac{P_{UF}}{kP_{CL}} + \frac{M_{UDx}}{m_x k M_{CEx}} + \frac{M_{UDy}}{m_y k M_{CEy}} \leq 1.0;$$

if  $0.15 \leq \frac{P_{UF}}{P_{CL}} \leq 0.5$  then

$$\frac{P_{UF}}{kP_{CL}} + \frac{C_{mx} M_{UDx}}{m_x k \left(1 - \frac{P_{UF}}{kP_{ex}}\right) M_{CEx}} + \frac{C_{my} M_{UDy}}{m_y k \left(1 - \frac{P_{UF}}{kP_{ey}}\right) M_{CEy}} \leq 1.0$$

and

$$\frac{P_{UF}}{kAF_{ye}} + 0.85 \left[ \frac{M_{UDx}}{m_x k M_{PCEx}} + \frac{M_{UDy}}{m_y k M_{PCEy}} \right] \leq ;$$

1.0;  $M_{UDx} \leq m_x k M_{PCEx}$ ;  $M_{UDy} \leq m_y k M_{PCEy}$

if  $\frac{P_{UF}}{P_{CL}} > 0.5$  then

$$\frac{P_{UF}}{kP_{CL}} + \frac{C_{mx} M_{UFx}}{k \left(1 - \frac{P_{UF}}{kP_{ex}}\right) M_{CLx}} + \frac{C_{my} M_{UFy}}{k \left(1 - \frac{P_{UF}}{kP_{ey}}\right) M_{CLy}} \leq 1.0$$

and

$$\frac{P_{UF}}{kAF_{ye}} + 0.85 \left[ \frac{M_{UFx}}{kM_{PCLx}} + \frac{M_{UFy}}{kM_{PCLy}} \right] \leq ;$$

1.0;  $M_{UFx} \leq kM_{PCLx}$ ;  $M_{UFy} \leq kM_{PCLy}$

The general assumptions have been used in evaluating all models are presented below:

- \* K, the knowledge factor = 1
- \* The rehabilitation goal is fair

The results of the evaluation are presented in Table 3. Based on this method, all beams and bracing members (deformation-controlled) have satisfied the acceptance criteria but lack of

acceptance of these criteria quite stands out in some percentage of columns.

**10.2. Linear Dynamic Procedure** The mentioned models have been analyzed and evaluated with spectrum obtained from PSHA analysis (Section 8).

The values of parameters used in this method are presented in Table 4. Acceptance criteria are implemented like Section 10.1.

The results of the evaluation are shown in Table 5. Based on this method, all bracing and beams members (deformation-controlled) have satisfied the acceptance criteria. In hazard level 2, lack of acceptance of these criteria is evident in some percentage of columns.

**10.3. Nonlinear Static Procedure** The mentioned models have been analyzed and evaluated by nonlinear static analysis (Target Displacement Method). Needed parameters for this analysis are presented in Table 6.

In order to model the stiffness of members in nonlinear static procedure, the principles of "Seismic Rehabilitation Code for Existing Buildings" (IIEES [11]) have been used. For modeling force-deformation curve of members (Figure 3), nonlinear parameters and acceptance criteria of section TUBO 100x100x10 are shown in Table 7 as a sample. Strain-hardening of components is accounted based on the slope of 3% of the elastic slope. In this table, d/t shows the ratio of depth/thickness. The results of the evaluation are presented in Table 8. As presented below, all beams (deformation-controlled) have satisfied the acceptance criteria, some percentage (exact values shown in Table 8) of bracing members (deformation-controlled) and columns have not satisfied acceptance criteria. Note that by increasing the stories the percentage of columns which have not satisfied the acceptance criteria will decrease.

**10.4. Nonlinear Dynamic Procedure** The mentioned models have been analyzed and evaluated using seven stated accelerograms (Table 1) and direct integration method. Description and attribution of nonlinear hinges of members are like Section 10.3. The results of the evaluation are presented in Table 9. As it is clear, some percentages

**TABLE 2. Loading Details Based on Linear Static Method.**

Earthquake Force Evaluation-Linear Static Procedure						
Building	5-Story		10-Story		15-Story	
Period	T = 0.514 s.		T = 0.85 s.		T = 1.15 s.	
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	-	-	-	-	37.82	65.95
14	-	-	-	-	52.52	91.59
13	-	-	-	-	48.49	84.57
12	-	-	-	-	44.47	77.56
11	-	-	-	-	40.12	69.97
10	-	-	30.85	53.62	35.76	62.37
9	-	-	41.39	71.95	31.45	54.85
8	-	-	36.47	63.39	27.41	47.80
7	-	-	31.63	54.98	23.27	40.59
6	-	-	27.00	46.93	19.01	33.15
5	31.48	57.49	22.13	38.47	15.22	26.54
4	38.40	70.11	17.32	30.10	11.50	20.06
3	29.55	53.97	12.75	22.16	8.01	13.97
2	20.49	37.42	8.24	14.31	4.90	8.54
1	11.23	20.51	4.10	7.13	2.23	3.88

**TABLE 3. Percentage of the Members which Don't Satisfy the Acceptance Criteria (Linear Static).**

Hazard Level 2			Hazard Level 1			LSP
Beam	Column	Bracing	Beam	Column	Bracing	Building
0	50	0	0	40	0	5-Story
0	56	0	0	24	0	10-Story
0	47.6	0	0	32.4	0	15-Story

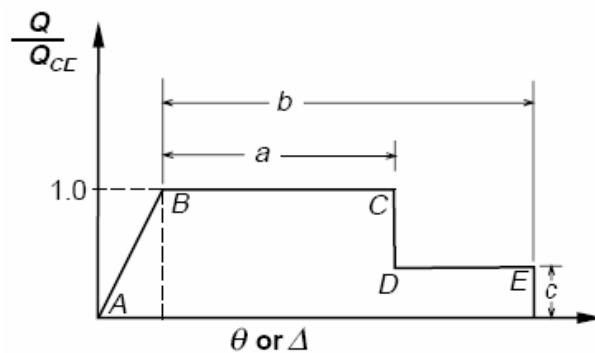


**TABLE 4. The Values of Parameters (Linear Dynamic).**

Building	5-Story		10-Story		15-Story		Note
Hazard Level (HL)	HL 1	HL 2	HL 1	HL 2	HL 1	HL 2	
C1	1	1	1	1	1	1	$1 + \frac{T_0 - T}{2T_0 - 0.2}$
C2	1	1	1	1	1	1	Linear Analysis
C3	1	1	1	1	1	1	$\theta < 0.1$
T0	0.5	0.5	0.5	0.5	0.5	0.5	Soil Type II
T	0.514		0.85		1.15		2800 Standard
T <sub>dynamic</sub>	0.52		0.77		1.3		Modal Analysis

**TABLE 5. Percentage of the Members which Don't Satisfy the Acceptance Criteria (Linear Dynamic).**

Hazard Level 2			Hazard Level 1			LDP
Beam	Column	Bracing	Beam	Column	Bracing	Building
0	40	0	0	0	0	5-Story
0	4	0	0	0	0	10-Story
0	0	0	0	0	0	15-Story



**Figure 3.** Generalized force-deformation relation for steel elements (FEMA 356 [8]).

**TABLE 6. Needed Parameters for Nonlinear Static Analysis.**

Parameters for Nonlinear Static Analysis-Load Distribution Type I and II							
Building	5-Story		10-Story		15-Story		Note
Hazard Level (HL)	HL 1	HL 2	HL 1	HL 2	HL 1	HL 2	
C0	1.4	1.4	1.5	1.5	1.5	1.5	Distribution Type I and II
C1	1.0	1.0	1.0	1.0	1.0	1.0	$T_e > T_0$
C2	1.1	1.2	1.1	1.2	1.1	1.2	LS-->C2=1.1, CP-->C2=1.2
C3	1.0	1.0	1.0	1.0	1.0	1.0	$\alpha > 0$
Sa (g)	0.92	1.54	0.59	0.94	0.436	0.697	Spectrum
T0 (s.)	0.5		0.5		0.5		Soil Type II
Te (s.)	0.514		0.852		1.15		Experimental
$\delta t$ (cm)	8.46	14.15	15.98	25.47	21.5	34.36	$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} g$

**TABLE 7. Parameters and Acceptance Criteria in Nonlinear Static Analysis (Bracing Members).**

Nonlinear Hinge Parameters													
Section (TUBEBOX)	d/t	Compression						Tension					
		a	b	c	IO	LS	CP	a	b	c	IO	LS	CP
100x100x10	10	0.5	7	0.4	0.25	3	4	11	14	0.8	0.25	7	9

**TABLE 8. Percentage of the Members which Don't Satisfy the Acceptance Criteria (Nonlinear Static).**

Hazard Level 2						Hazard Level 1						NSP	
Type II			Type I			Type II			Type I				
Beam	Column	Bracing	Beam	Column	Bracing	Beam	Column	Bracing	Beam	Column	Bracing	Building	
0	10	60	0	25	80	0	10	60	0	15	40		5-Story
0	4	55	0	14	75	0	4	50	0	10	60		10-Story
0	1.9	40	0	3.8	40	0	0	23.3	0	0	16.7		15-Story

**TABLE 9. Percentage of the Members which Don't Satisfy the Acceptance Criteria (Nonlinear Dynamic).**

Hazard Level 2							Hazard Level 1									
Superstition Hills	Palm Springs	Northridge	Loma Prieta	Kocaeli	Kobe	Cape Mendocino	Superstition Hills	Palm Springs	Northridge	Loma Prieta	Kocaeli	Kobe	Cape Mendocino	Member	Building	
instability	instability	instability	30	60	instability <sup>3</sup>	60	30	10	50	50	30	50	50	Bracing	5-Story	
instability	instability	instability	20	30	instability	20	0	0	20	20	0	0	20	DC <sup>1</sup>		Column
instability	instability	instability	90	100	instability	100	100	90	100	90	100	100	100	FC <sup>2</sup>	Beam	
instability	instability	instability	0	0	instability	0	0	0	6.67	0	0	0	0	Bracing		10-Story
instability	instability	instability	instability	instability	instability	instability	instability	57.5	instability	instability	45	instability	50	DC	Column	
instability	instability	instability	instability	instability	instability	instability	instability	0	instability	instability	3.33	instability	6.67	FC		Beam
instability	instability	instability	instability	instability	instability	instability	instability	60	instability	instability	20	instability	65	Bracing	15-Story	
instability	instability	70	instability	instability	instability	instability	instability	instability	instability	28.3	25	51.7	51.7	58.3		DC
instability	instability	13.3	instability	instability	instability	instability	instability	instability	instability	0	1.3	9.3	1.3	6.67	FC	Beam
instability	instability	78.3	instability	instability	instability	instability	instability	instability	instability	46.7	36.7	50	43.3	66.7	Column	
instability	instability	0	instability	instability	instability	instability	instability	instability	instability	0	0	0	0	0	Beam	

<sup>1</sup>DC: Deformation-Controlled

<sup>2</sup>FC: Force-Controlled

<sup>3</sup>Instability mode happens when a row or column of the stiffness matrix becomes zero or negative.

of beams, bracing members (deformation-controlled) and columns have not satisfied the acceptance criteria. Lack of stability of different buildings against some earthquakes is visible in the hazard levels 1 and 2 especially for the 10-story model.

## 11. CLASSIFICATION OF RESULTS

In order to get a good understanding of analyses, the results of four types of analyses are shown for comparison in a column form in Figure 4 to 9. In these columns:

LSP: stands for linear static procedure, LDP: stands for linear dynamic procedure, NSP: stands for nonlinear static procedure, NDP: stands for nonlinear dynamic procedure.

The vertical axis shows the percentage of failed members (members which have not satisfied acceptance criteria) and the nonlinear analysis results would be considered as benchmark for results accuracy.

## 12. CONCLUSION

The quantitative results have been shown in Sections 10 and 11. Before expressing the final conclusion, the limitations and assumptions made for the analyses are:

a. Very high seismicity zone, b. Regular and symmetric buildings, c. Primary design based on equivalent static analysis and d. The 7 mentioned accelerograms.

Hence, the summary of the results are:

1. The accuracy of linear analysis (static and dynamic) in evaluation of the bracing members is unreliable.
2. The results of linear static analysis are closer to reality than linear dynamic and nonlinear static analyses in evaluation of the columns.
3. In the dual-system steel frame buildings, the main role of lateral load-bearing is on bracing members and beams have no such important role.
4. In particular, the results obtained from

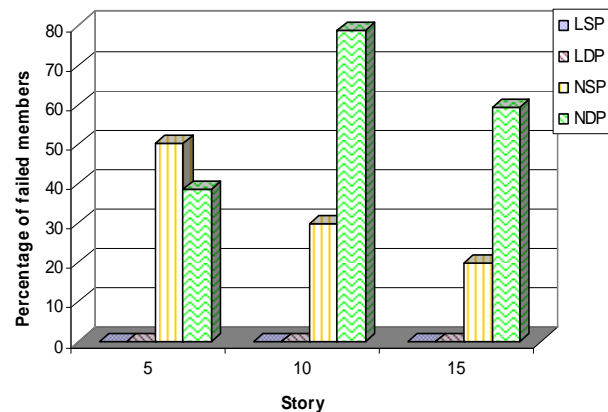


Figure 4. Comparison of results accuracy derived from 4 types of analyses, bracing members (HL 1).

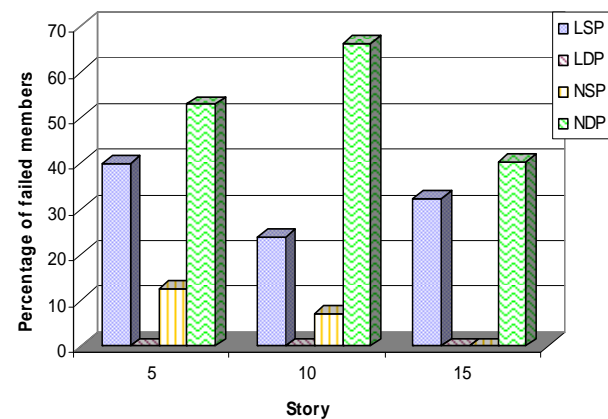


Figure 5. Comparison of results accuracy derived from 4 types of analyses, columns (HL 1).

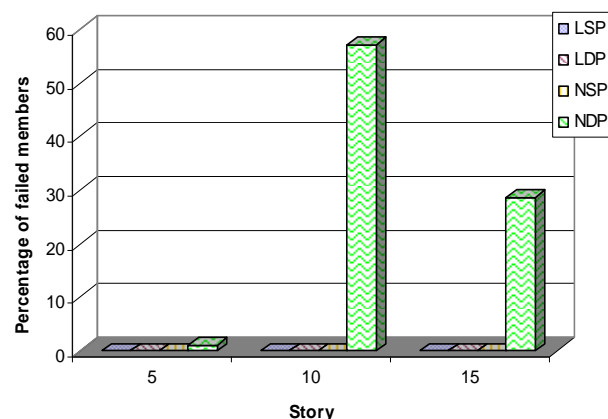
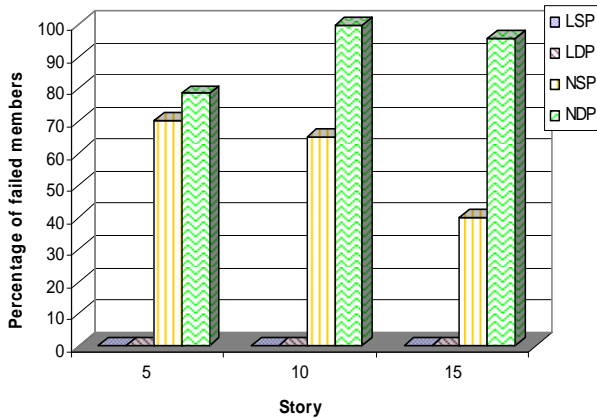
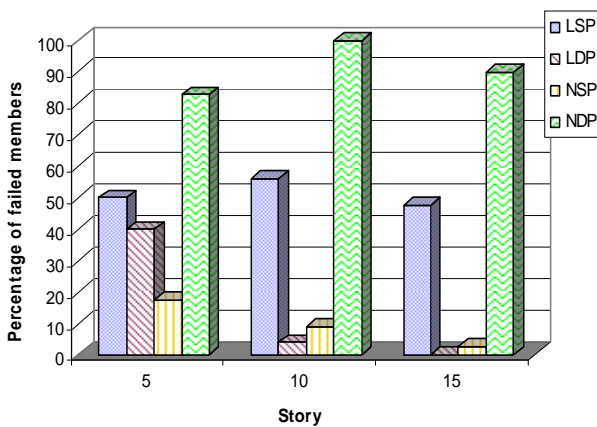


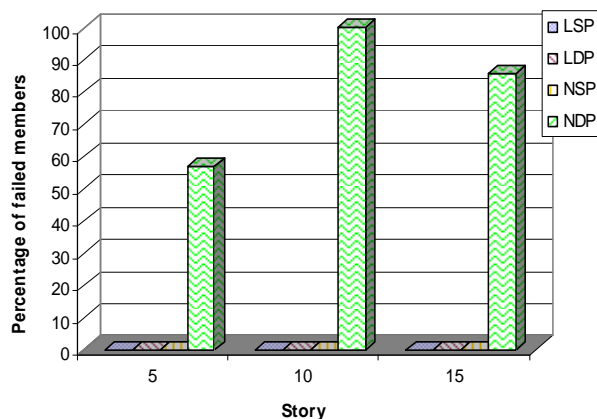
Figure 6. Comparison of results accuracy obtained from 4 types of analyses, beams (HL 1).



**Figure 7.** Comparison of results accuracy derived from 4 types of analyses, bracing members (HL 2).



**Figure 8.** Comparison of results accuracy derived from 4 types of analyses, columns (HL 2).



**Figure 9.** Comparison of results accuracy derived from 4 types of analyses, beams (HL 2).

nonlinear static analysis are more accurate and more reliable than linear static and linear dynamic analysis.

- The results of nonlinear static analysis in the evaluation of 15-story building are less accurate than 5 and 10-story buildings. (This might be caused by lack of contribution of higher modes effects in load distribution pattern used in this method. Since in tall buildings higher modes have substantial effects, it is recommended that in nonlinear static analysis of tall buildings, MPA method (e.g., Chopra, et al [14]) be used).
- Based on nonlinear dynamic analysis for the buildings designed according to 2800 Standard, the results have been stated below:

**A.5. Story Building** Hazard Level 1: Around 29 % of members do not satisfy the acceptance criteria.

Hazard Level 2: Around 66 % of members do not satisfy the acceptance criteria.

**B.10. Story Building** Hazard Level 1: Around 68 % of members do not satisfy the acceptance criteria.

Hazard Level 2: The structure experienced instability.

**C.15. Story Building** Hazard Level 1: Around 41 % of members do not satisfy the acceptance criteria.

Hazard Level 2: Around 90 % of members do not satisfy the acceptance criteria.

## 13. ACKNOWLEDGEMENT

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## 14. NOTATIONS

- A Area of bracing,  
 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,	$S_a$	Spectral response acceleration, g,
$C_2$	Modification factor to represent the effects of pinched hysteretic shape, stiffness degradation and strength deterioration on the maximum displacement response,	$T$	Fundamental period of the building in the direction under consideration,
$C_3$	Modification factor to represent increased displacements due to P- $\Delta$ effects,	$T_0$	Period at which the constant acceleration region of the design response spectrum transitions to the constant velocity region,
$C_m$	Effective mass factor to account for higher mode mass participation effects,	$T_e$	Effective fundamental period of the building in the direction under consideration,
$E$	Modulus of elasticity,	$V$	Pseudo lateral load,
$F_{as}$	Allowable compression stress,	$W$	Effective seismic weight of a building including total dead load and applicable portions of other gravity loads,
$F_i$	Lateral load applied at floor level $i$ ,	$Z$	Plastic section modulus,
$F_y$	Yield strength of the material,	$h_i$	Height from the base to floor level $i$ .
$F_{ye}$	Expected yield strength,	$h_j$	Height from the base to floor level $j$ .
$F_u$	Tensile strength of the material,	$g$	Acceleration of gravity,
$J$	Force-delivery reduction factor	$k$	Knowledge factor,
$K$	Stiffness Matrix,	$m$	Component or element demand modifier (factor) to account for expected ductility associated with this action at the selected Structural Performance Level,
$M$	Mass Matrix,	$r(t)$	External forces vector,
$M_{CE}$	Expected flexural strength of a member,	$u(t)$	Displacement vector,
$M_{PCE}$	Expected plastic flexural strength,	$\dot{u}(t)$	Velocity vector,
$M_{PCL}$	Lower-bound plastic flexural strength,	$\ddot{u}(t)$	Acceleration vector,
$M_{UD}$	Bending moment (deformation-controlled),	$w_i$	Portion of the effective seismic weight $W$ located on or assigned to floor level $i$ ,
$M_{UF}$	Bending moment (force-controlled),	$w_j$	Portion of the effective seismic weight $W$ located on or assigned to floor level $j$ ,
$P_e$	Euler critical force,	$\Delta$	Generalized deformation,
$P_{CE}$	Expected compression strength of the column,	$\theta$	Generalized deformation, radians,
$P_{CL}$	Lower-bound compression strength of the column,	$\delta_t$	Target displacement,
$P_{DL}$	Axial force in member, due to dead load,	$\nu$	Poisson's ratio,
$P_E$	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,		
$P_{UF}$	Column axial force,		
$Q$	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,		

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