13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 2356

INVESTIGATION ON NONLINEAR ANALYSIS OF EXISTING BUILDING AND INTRODUCING SEISMIC RETROFITTING OUTLINES WITH CASE STUDY

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SUMMARY

The advantages of the performance design method encourage the engineers to acquire and develop a better knowledge of the procedures. In our research, we have tried to clarify the application of the method by using some practical examples. We have considered four buildings of 2, 5, 10 and 15 stories. The steel structures were designed according to AISC/ASD and the current seismic provisions for buildings (UBC and BHRC 2800). We have carried out performance based design using both linear (LDP) and nonlinear (NSP) procedures. Considering the results, it appears that nonlinear static procedure (pushover) using displacement coefficient method is a reliable way to evaluate the structures.

INTRODUCTION

The practice of earthquake engineering is rapidly evolving and both understanding of the behavior of buildings subjected to strong earthquakes and our ability to predict this behavior are advancing. An analysis of structure shall be conducted to determine the distribution of forces and deformations induced in the structure by earthquake shaking.

Steel moment frame buildings are designed to resist earthquake ground shaking based on the assumptions that they are capable of extensive yielding and plastic deformation, without loss of strength. Designs of ordinary seismic resistant structures are mostly carried out by application of equivalent static design method using simplified assumptions. The Uniform Building Code (UBC [1], BHRC 2800 [2]) and some other seismic design provision codes propose the application of a seismic force reduction factor R (called behavior factor), to estimate the inherent over-strength and ductility of seismic force resisting systems. These methods provided relatively conservative results. In order to permit more reliable performance in seismic resistant structures, the new design provisions are intended to consider more realistic characteristics of structures.

FEMA prestandard for seismic rehabilitation of buildings (FEMA 356 [3]) is based on performance-base design methodology that differs from seismic design procedures to design new buildings, currently specified in National Building Codes and standards [1].

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The analysis procedure proposed by FEMA 356 [3] can be carried out by one of the following methods:

- Linear analysis complying with linear static procedure (LSP) or linear dynamic procedure (LDP).
- Nonlinear analysis complying with nonlinear static procedure (NSP, often called "Pushover" analysis) or nonlinear dynamic procedure (NDP).

The linear analysis procedures are based on linear behavior of the material and rely on traditional use of linear stress-strain relationship. The procedures are adjusted regarding overall building deformations and material behavior criteria, to permit better consideration of the probable nonlinear characteristic of seismic response. Nonlinear static procedure uses simplified nonlinear technologies to estimate seismic deformations.

ANALYSIS PROCEDURES

The application of linear analysis procedures depends on the condition of irregularity. The analysis of irregularity identifies the magnitude and uniformity of distribution of inelastic demands on elements of lateral-force-resisting system. This shall be defined by demand capacity ratios (DCRs) of each primary component, calculated for each action such as axial force, moment and shear. Linear procedures shall not be used unless earthquake demand on building comply with the demand capacity ratio (DCR) requirements.

$$DCR = Q_{UD}/Q_{CE}$$

where :

Q_{UD} =Force due to the gravity and earthquake loads.

Q_{CE} =Expected strength of component at the deformation level under consideration for deformation – controlled action.

The results of the linear procedures can be very inaccurate when applied to building with highly irregular structural systems, unless the building is capable of responding to the design earthquake in a nearly elastic manner.The linear procedures are applicable:

-If all component DCRs ≤ 2

-If one or more component DCRs exceeds 2.0 and no irregularities are present.

If all of the computed DCRs for a component are less than or equal to 1.0, then the component is expected to respond elastically to the earthquake ground shaking.

Linear static procedure shall not be used for buildings when the fundamental period of the building, T, is greater than or equal to $3.5T_s$. (T_s = characteristic period of the response spectrum).

For building in which linear procedures are applicable, but the linear static procedure is not permitted, use of linear dynamic procedure shall be permitted.

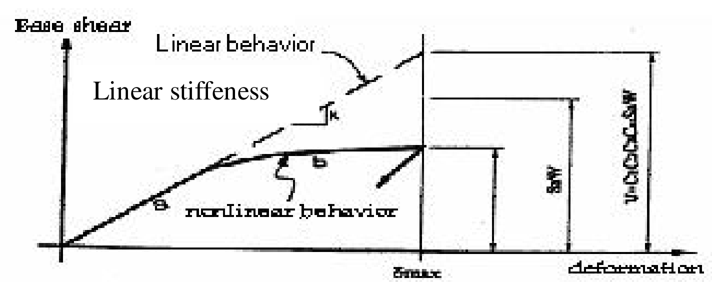

Fig.1. Comparison between actual and ideal behavior of structures

COMPUTING MODEL

For this research we have considered four buildings having 2 , 5 , 10 and 15 stories. All the building were made of steel structures. The seismic resisting system in both perpendicular directions of each structure is

moment resisting frame. All 4 constructions have the same plan as shown in Figure 2. The story height is 3.6 meters. All of the beams are selected from IPE shapes. The columns have the hollow rectangular (box) section.
The steel structures are made of structural steel (ST37) with yield strength F_{Y}=2400 Kg/cm^2 and minimum tensile strength of F_u=4000 Kg/cm^2. The poisson ratio, ν , and elastic modulus of steel E , are considered equal to 0.3 and 2.1*10^{6}Kg/cm^2 respectively.

The dead load of 750 Kg/m^2 and live load of 200 Kg/m^2 are considered to be applied at all stories and roofs surfaces. The structures were design according to AISC "Specification for Structural Steel Building" [4,5]. The structures were analyzed using the common computer linear and nonlinear finite element programs.

All the constructions are regular from point of view of mass and stiffness distribution as well as geometry in-plane and out-of-plane. They are also regular in height.

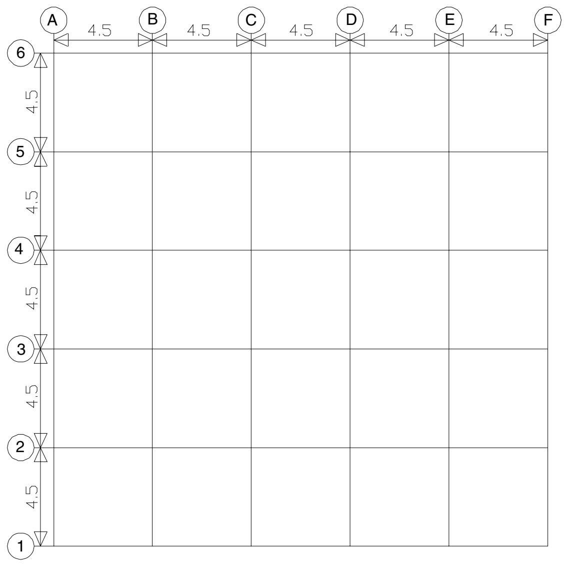

FIG.2. PLAN OF ALL BUILDINGS

LINEAR PROCEDURE

For linear analysis, the buildings have been modeled and analyzed as three -dimensional systems. The multidirectional seismic effects were considered. Elements of the buildings have been designed for combination of forces and deformation in x and y directions according to procedure proposed by FEMA 356[3].
Eigenvalue analyses of mathematical model of buildings have been used to obtain the fundamental period of structures, T_0 . The values of calculated periods are shown in Table 1.

TABLE 1. PERIOD OF STRUCTURES

δ_t (Cm)	T(sec)	C_m	C_3	C_2	C_1	C_0	Building
14.78	0.8577	1	1	1.1	1	1.2	2 stories
30.6	1.32	0.9	1	1.1	1	1.4	5 stories
54.5	1.94	0.9	1	1.1	1	1.5	10 stories
77.5	2.52	0.9	1	1.1	1	1.5	15 stories

To apply the linear static procedure (LSP) actions and deformations in elements and components of each building have been calculated using the pseudo lateral load "V" in accordance the equation:

$$V = C_1C_2C_3C_mS_aW$$

where :

C_1=modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response.

C_1 =1.5 for T<0.10 second

C_1 =1 for $T>T_s$

T = fundamental period of the building

T_s = characteristic period of response spectrum

C_2 = modification factor to respect the effect of stiffness degradation and strength deterioration on maximum displacement response (for linear procedures C_2 =1)

C_3 = modification factor to represent increased displacements due to dynamic $P-\Delta$ effects.

C_m = effective weight factor to account for higher mode mass participation effects.

(C_m =1 for 2 stories buildings or when T >1 second)

S_a = response spectrum acceleration at the fundamental period and damping ratio of the building.

W= design gravity load including total dead load a portion of live loads.

Regarding the restrictions imposed by FEMA 274 [6], application of linear static procedure is limited to structures with fundamental period less than $2.5T_s$ (T_s = characteristic period of response spectrum).

Therefore, instead of linear static procedure (LSP), linear dynamic procedure (LDP) was select for seismic analysis of all the four buildings. The buildings were modeled with assumption of linearly elastic stiffness distribution. Equivalent vicous damping ratio of 5% was considered for all buildings. Model spectral analysis was carried out using linearly-elastic response spectra. The dynamic analyses were carried out using the response spectrum method. The actions and deformations were multiplied by the modification factors C_1, C_2 and C_3. All the buildings were assumed to have rigid diaphragms. Normalized spectral curves for soil with period of 0.5 second and acceleration of 0.35 (see Figure 3) were considered [2].

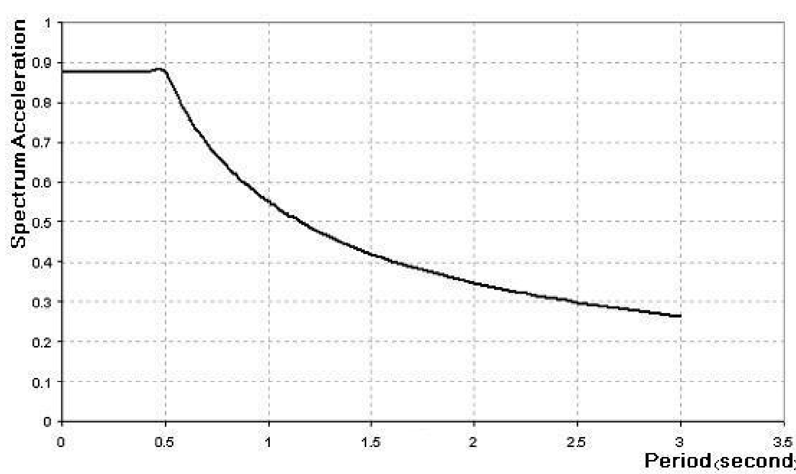

Fig.3. Normalized spectral curve

The considered combinations of gravity and earthquake loads are [3,8]:
Combo1 : 1.1(DL + LL) + Ex
Combo2 : 0.9DL + Ex
Combo3 : 1.1(DL + LL) + Ey
Combo4 : 0.9DL + Ey
Combo5 : 1.1(DL + LL) - Ex
Combo6 : 0.9DL - Ex
Combo7 : 1.1(DL + LL) - Ey
Combo8 : 0.9DL - Ey

The acceptability of component force and deformation actions have been evaluated for each component in accordance with the requirements (see FEMA 356 [3]). The actions in the structures have been classified as being deformation–controlled (ductile) or force-controlled (nonductile).

Design strengths used for deformation-controlled actions are denoted Q_{CE} and have been taken as equal to generalized component expected strengths. Expected strengths have been defined as mean maximum resistance expected over the range of deformations to which the generalized component is likely to be the subjected. The expected deformation capacities of deformation-controlled actions have been specified using the general procedures.

Strength used in design for force-controlled actions are denoted Q_{CL} and have been taken as equal to lower-bound strengths (mean strength minus one standard deviation). Values for component demand modification factor, m, for different performance levels (immediate occupancy, IO, life safety, LS, and collapse prevention) have been determined.

Table 2. Results of analysis for column C3 in first floor

Buildings	Load	Force	M2	M3	P/P_{CL}	DCR	m	M_{UDY}/M_{CEY}	$M_{UD.X}/M_{CEX}$	$\Sigma\sigma$
2 stories	COMBO1	22	0	3903	0.09	1.26	6	0	0.17	0.25
10 stories	COMBO1	-2.8	3.9	2278.9	0.02	1.46	6	0	0.21	0.22
10 stories	COMBO2	-283	-1	-7002	0.67	1.65				
15 stories	COMBO1	187	-2	9330	0.29	1.08	4	0	0.17	0.46
15 stories	COMBO2	-490	-2	-9492	0.77	1.57				

Values of m for column C3 for each building have been presented in Table (2). The other parameters shown in Table 2 are defined as:
M_{CEx} = expected bending strength of the column for the x- axis
M_{CEy}= expected bending strength of the column for the y- axis
M_X = bending moment in the member for the x- axis
M_y = bending moment in the member for the y- axis
P_{CL} = lower-bound compression strength of the column
Deformation-controlled design generalized action, Q_{UD}, due to gravity load and earthquake loads have been calculated according to equation:

$$Q_{UD} = Q_G + Q_E$$

where:
Q_G = Generalized action due to design gravity loads.
Q_E = Generalized action due to design earthquake loads.
As a sample, the results or the linear dynamic procedure analysis for column C3 (see Figure 2) at the first floor of each buildings are presented in Table 2.

The results of our analyses show that all the beams and columns in to 2 and 5 stories buildings shall be considered deformation-controlled. For 10 and 15 stories building all the beams are also deformation-controlled, but some of the columns are considered force-controlled.

NONLINEAR STATIC PROCEDURE (NSP)

The nonlinear static procedure (Pushover analysis) is generally a more reliable approach to characterizing the performance of a structure than are linear procedures. The NSP shall be permitted for structures in which higher mode effects are not significant. To determine if higher modes are significant a modal response spectrum analysis has been performed for all model structures. Sufficient modes were considered to capture 90% mass participation.

For nonlinear static procedure the mathematical models incorporating the nonlinear load-deformation characteristics of all individual components of each buildings were established. Then the models were subjected to monotonically increasing lateral loads representing inertia forces in an earthquake until either a target displacement is exceeded or building collapses.The target displacement indicates the maximum displacement which will be experienced by the structures during the earthquake. The target displacement, δ_i, at each floor level shall be calculated in accordance with the equation:

$$\delta_i = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} g$$

where :

T_e = Effective fundamental period of the building.

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

g =Acceleration of gravity (9.81 m/sec^2)

The magnitude of the target displacements for all the building were calculated and presented in Table 3.

Table 3: Target displacement δ_i (cm)

Building	2 stories	5 stories	10 stories	15 stories
δ_i	14.78	30.6	54.5	77.5

Table 4: different of base shear

	2 stories	5 stories	10 stories	15 stories
Spectrum	438 (ton)	757 (ton)	1134 (ton)	1413 (ton)
Uniform	535 (ton)	905 (ton)	1401 (ton)	1765 (ton)

The control node was considered at the center center of mass at the roof of each building. The displacement of control node in each mathematical model was calculated for specified gravity and lateral loads. Acceptance was based on forces and deformations in components and corresponding to a minimum horizontal displacement of the control node equal to the target displacement, δ_i.

According to FEMA 356 procedures, lateral loads shall be applied to the mathematical model of building 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. Among the different types of vertical distributions of lateral loads the following two pattern seems to be more suitable to the studied cases:

- Uniform distribution at each level propositional to the lateral mass.
- Distribution calculated by response spectrum analysis of the buildings.

These methods have been applied to the four considered buildings. Table 4 presents the values of the base shear calculated by uniform distribution and spectrum distribution methods.

ANALYSIS RESULTS

When structures are designed according to the seismic provisions, (eg. UBC or AISC …) no human loss is otherwise expected. It means that safety level of the seismic building codes, normally coincide with the life safety performance level. In another words, the designed structures have enough ductility to support the target displacement without failure. The above matter is presented by Figures 5 to 8. As we can see all structures have supported the imposed target displacement. To evaluate the behavior of these structures beyond the target displacement, the applied load were increased up to the collapse load. We have realized that the collapse first happens in all the structures due to uniform vertical distribution of the lateral loads.

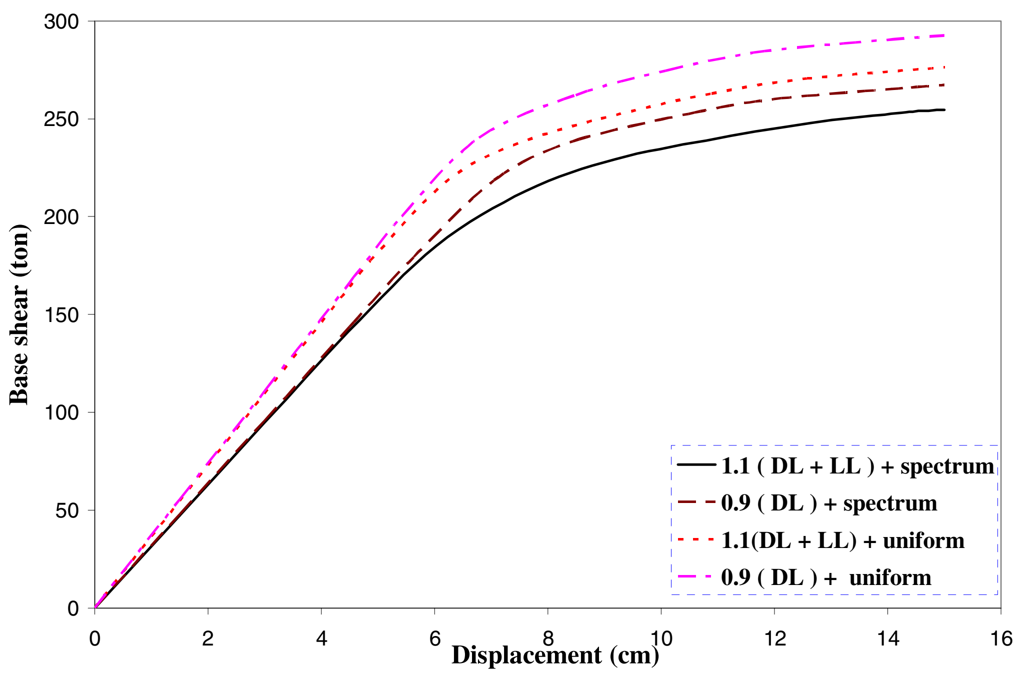

Fig.5: Pushover diagrams for 2 stories building

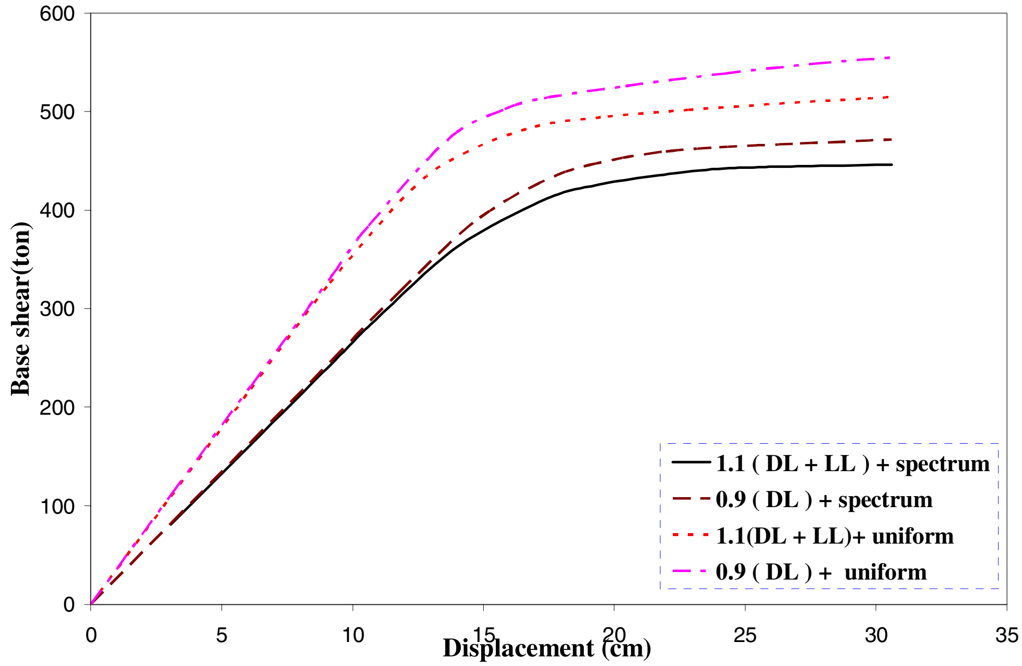

Fig. 6: Pushover diagrams for 5 stories building

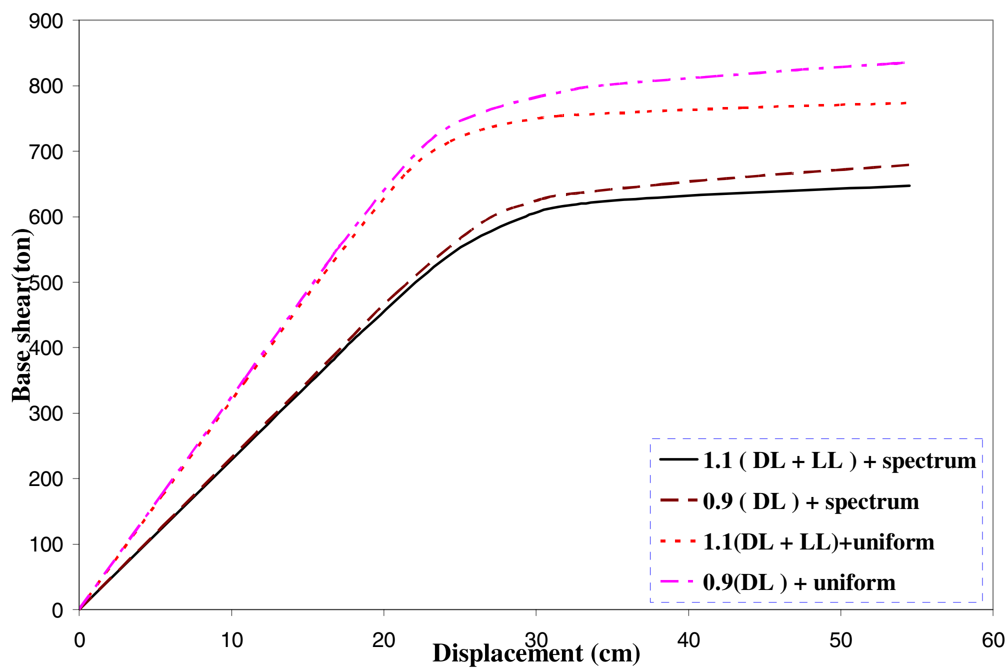

Fig.7: Pushover diagrams for 10 stories building

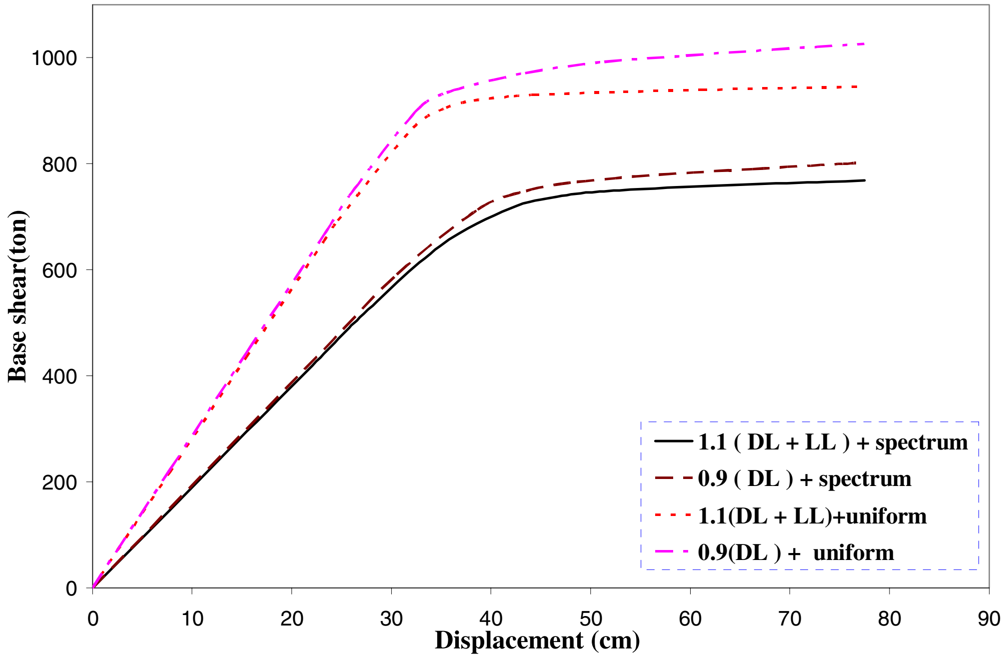

Fig. 8: Pushover diagrams for 15 stories building

CONCLUSIONS

In this research we have considered 4 buildings of different heights. The buildings were designed according to AISC specifications for design of steel structures , and they have been reinforced to resist the earthquakes according to the exiting seismic codes (UBC , AISC ,…) . These structures have been examined according the Fema 356 " prestandard for the seismic rehabilitation of buildings". We have conclude that:

1. Nonlinear static procedure gives more accurate results.
2. Uniform vertical distributions of lateral loads are more critical for these types of structures.
3. Although FEMA 356 propose the application of analytical and experimental period for structures, but our research show that the use of analytical period is more reasonable. Application of experimental period led to more conservative results.
4. Design of structures following the usual "seismic design provision codes" enable one have good safety factors.

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