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Study on Role of the Applied Load and Height of the Structure on the Performance Level of the Dual Steel Moment Frame with the Coaxial Steel Bracing

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ABSTRACT

In order to design the resistant structures against earthquake forces and considering the defects of the elastic design and use of the behavior coefficient, importance of the design is specified based on the performance. Considering high importance of the load pattern in Pushover analysis which is one of the requirements of Capacity Spectrum Method to determine the Performance Point, effects of the loading model on the performance point of the structures with steel moment frame dual system with the steel coaxial bracing with different heights is studied and the results have been compared with the linear dynamical loading by studying the numerical examples.

Keywords: Performance level, Capacity spectrum, Pushover analysis, Performance point, Load pattern

Introduction:

One of the most important factors which can damage a structure during its life is the earthquake. Due to the random nature and dependence of the earthquake intensity on different factors such as distance from the fault, soil type, earthquake magnitude etc, the structure design has turned into a complex process so that it should resist against the earthquake and changing trend of the human knowledge can be described s follows based on the design and construction of the earthquake-resistant structures.

After elaborating this fact that the movement resulting from earthquake causes a force in foundation of the structure, the engineers were able to solve the motion equation and calculate the elasticity response spectrum for different earthquakes by making the accelerometer and recording different accelerations of the earthquakes. Based on this spectrum, the engineers were able to calculate the acceleration created in the upper part of the structure in an earthquake for the structure with the specified interval time. It is necessary to note that the force calculated for the earthquake was calculated between half of the building weight and twice as much as the building weight while the first values which had been presented as the earthquake design force in the codes were about 10% of the building weight. Considering that some structures designed with this force had resisted against some earthquakes and design based on the elastic design which didn't allow the stress of each element to pass the elasticity limit was not economically justifiable considering low probability of the earthquake occurrence during life of the structure, therefore, the engineers sought to find a method for reducing the values obtained from the elasticity spectrum. In order to justify this fact, in case a relatively large lateral force is applied on a structure, stress in the elements exceeds the elasticity limit and as a result, it is absorbed by the structure by the energy plasticity transformation. In this regard, considering the ratio of plastic transformation to elastic transformation, the concept of the behavior coefficient was defined and included in the codes. With this concept, the structure is designed for the force which is obtained by dividing the elasticity force by the behavior coefficient. After the force reaches the maximum tolerable value , the plastic behavior begins and energy is absorbed in this way. In the ordinary designs , there are some defects in this method according to the codes , for example, when the design is done in the elasticity limit , the main factor of the element fracture is the transformation of the element not resistance of the elements. One of the main factors of stability, applicability and suitable performance of the structure is drift of the structure. In addition, the actual value of the behavior coefficient changes from a structure to a structure. But a single behavior coefficient is given in the buildings codes for each structure form. Considering the fact that the earthquake force is firstly calculated by the designer with help of the elasticity spectrum in the resistance -based design and the design force is calculated by dividing this umber by the behavior coefficient and the elements are designed in such a manner that they have sufficient resistance against this force while the real ductility of this structure has not been studied and it is not clear if the calculated non-elastic force is lower than the structure tolerability. In addition, as it was mentioned, the main factor of fracture in elements and efficiency of the structure is the transformation which has not been controlled. In contrary to the resistance base design, a new method was innovated for the earthquake resistant design called performance base design.

1- Performance base design

Considering the defects of the resistance based design, necessity for the transformations based design was specified. In the performance base design, the performance means performance in terms of transformation not resistance. The real damage of the building and the damage incurred on the structure are evaluated in special seismic levels compared with the stipulated criteria using the performance base method [1]. The main goal of performance base earthquake engineering is to prepare the methods for layout, design, construction and maintenance of the building. These methods can predict performance of the structure when it is affected by an earthquake. The performance base design principles are such that a performance goal is considered for the structure considering the cases such as the building use , importance of the building in terms of its related activities , economic considerations , value of the building as a historical or cultural monument etc. The performance goal means having a specified performance level for the seismic hazard levels. Then the studied structure is affected by the earthquake with the definite hazard level and it is specified that whether performance of the structure meets the performance goal or not. In case the performance level is not met, this performance level should be reinforced properly until this performance level is achieved [3,4]. The performance goals, performance levels and seismic hazard level have been quantitatively and qualitatively defined by different bylaws. These concepts are explained in detail.

2-1- performance levels

Different performance levels mean the maximum permissible drift of the stories level and the transformations in the elements and the maximum expected physical fractures which can quantitatively describe life security level for the buildings residents and the serviceability of the structure after the earthquake occurrence. It means that the increase of fractures and drifts from the defined level changes the performance level of the structure. Status of all structural and non-structural components interferes in definition of the performance levels [5,6].

2-2- seismic risk levels:

Different risk levels are defined as the average return period or probability of occurrence in the life expectancy of the structure. The average return period indicates the average time in year between the earthquakes with magnitude equal to or more than a specified limit. The occurrence probability of earthquake indicates probability of an earthquake with a specified magnitude or above the specified limit during a time period [6].

2-3- the performance goal

The performance goal studies the relationship between the performance levels and seismic levels so that it specified what performance level the related structure has under any specified seismic level. In case this level is met, the design will be safe.

2- The structure analysis methods:

In performance base design method, we need to determine the capacity and the seismic need of the structure to determine the performance and its components. In fact, the need shows the seismic motions and the capacity shows ability of the structure against the seismic demands. In order to achieve this goal, we should use the analytical methods which can model behavior of the structure and its components as well as the seismic motions. The analytical methods which have been proposed are divided into two linear and nonlinear classes each being performed statically or dynamically. Four available methods for analysis include: linear static procedure (LSP), linear dynamic procedure (LDP), nonlinear static procedure (NSP) and nonlinear dynamic procedure (NDP).

Each one of the analytical procedures indicates a level for the structure analysis. When we use higher level, we will see more accurate model of the real performance of the building against the earthquake but more effort is necessary to spend time for gathering the primary data and conducting the related calculation. The design related strategies have been based on the four analysis levels. However, only two nonlinear choices clearly model performance of different components under different damage levels resulting from the earthquake. Considering the fact that earthquakes are dynamical phenomena, these dynamical characteristics have considerable effect on the magnitude and distribution of the damages [7]. It is evident that the optimal method has nonlinearity effect and dynamic effect and as a result, NDP is preferred over other methods. This procedure has some problems which have limited its use such as high volume of calculation, high volume of the input data, problematic elaboration of the results and difficult selection of the suitable time histories. In recent years, NSP versus NDP is more considered and the reason is ability of the above procedure to calculate the structural parameters approximately without needing the modeling, complex calculation and hypotheses and time history of the earthquake which we need in the dynamic analysis. Another important factor which increases the tendency to use the nonlinear static analysis procedure is the ability of this procedure to follow behavior of the structurestep by step during its non-elastic performance and to follow the fracture mechanism in the elements. This is not easily possible in the dynamic analysis [3].

3- Capacity Spectrum Approach

One of the methods for determining the performance level is the Capacity Spectrum Approach. In this approach, the maximum response of the structure (performance level) to the seismic demand (seismic risk level) is calculated considering capacity of the structure. The drift curve in the structure is compared with the earthquake response capacity curve which indicates the earthquake demand in terms of the applied force which in fact shows capacity of the structure using a graphical method. After making the changes which include the damping energy absorption of the structure, the performance point which is the confluence of the structure capacity curve and the earthquake demand curve is calculated. Performance level of the structure in the seismic risk level which is used in the analysis is determined using the performance point. Since it is not possible to consider the drifts in all degrees of freedom in the structures with some degrees of freedom, roof drift curve is used in terms of the base shear as the capacity curve. How these curves are drawn and the performance point is obtained is explained later.

4-1- drawing the structure capacity curve

In order to draw the capacity curve (figure 3-A), Pushover analytical procedure is used. The Pushover analytical procedure estimated the nonlinear behavior of the structure under the earthquake. Although the forces are statically applied, it is expected that this method specify a suitable estimate of the maximum nonlinear dynamical response of the structure for many buildings. As specified from Pushover method, the structure is pulled with a lateral load pattern until there is instability in the structure. In this method, the model should be created in such a manner that the nonlinear characteristics relating to the elements which result from the nonlinear behavior of the material can be applied in suitable manner such as considering the plastic joints [9]. As a result, Pushover analysis of the base shear curve is drawn in terms of roof drift.

4-2- lateral load pattern in Pushover Analysis

It is possible to consider the different load patterns in analysis but distribution of the lateral load on the structure model should be similar to what will occur in the earthquake and create critical modes of transformation and internal forces. Therefore, in order to distribute the push force, it is necessary to specify what kind of the yield modes should be studied. Any different model of fracture will have its own load and any push force should be similar to the inertia force which is applied to the structure at time of dynamical response. Figure 1 shows a load of a simple building model.

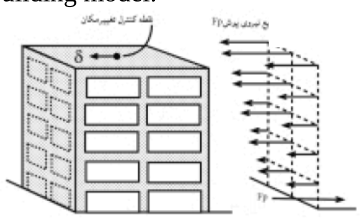


Figure 1- an example of loading on the building for the static nonlinear analysis [10]

The main types of lateral load distribution are as follows [6, 11]:

Application of a centralized force in the roof level: generally, this method is only used for one-story structure (figure 2-A).

Uniform distribution: in this method, lateral load distribution is calculated based on weight of each story and in case of the fixed weight of the stories; force applied to all stories will be fixed (figure 2-B). This pattern is used to study the critical models in the lower stories. Due to the low point of the lateral forces effect point, ratio of the base shear to overturning moment is large.

Distribution based on the lateral load distribution in linear static method according to the codes without considering the bull whip effect:

This method is used when at least 75% of the structure mass participates in the first vibration mode. In case of the fixed weight of the stories, the intended distribution is converted to the triangular distribution (figure C-2). Relation 1 shows how to apply this distribution.

(1)

$$F_i = \left[\frac{w_i.h_i}{\sum w_i.h_i}\right].v$$

In this relation, h_i , w_i and F_i are he height , weight and the centralized force of the i th story and v is the base shear.

Distribution based on the lateral load distribution in linear static method according to the codes considering the bull whip effect: in this method, higher modes effects are considered to some extent due to the bull whip effects (figure 2-D). The stories level force is calculated according to relation 2 and the bull whip force is applied to the roof level. The bull whip force is shown with F_t .

(2)

$$F_i = \left[\frac{w_i.h_i}{\sum w_i.h_i}\right].(v - F_t)$$

Distribution based on the first vibration mode: this method is used when at least 75% of the structure mass participates in the first vibration mode (figure 2-E). This distribution is more accurate than the previous methods due to consideration of the dynamic specification of the structure but this method doesn't consider high modes effect. Relation 3 shows how to apply this distribution. φ_i is the level mode of i th.

(3)

$$F_i = \left[\frac{w_i.\varphi_i}{\sum w_i.\varphi_i}\right].v$$

Distribution based on the linear dynamic analysis: in this load distribution, dynamical specifications of the structure and high modes effect are considered. For this reason, it is more accurate than other methods (Figure 2-F). In this method, the base shear was calculated in nth mode (v_n) with relation 4. Then the i level centralized force in mode (F_{in}) n is calculated by relation 5. Then, the centralized force of each level is calculated with help of forces of the same level in different modes using a method such as SPSS

(4)

$$v_n = \left[\frac{\mu_n^2}{M_n}\right].S_{a\ n}$$

(5)

$$F_{in} = \left[\frac{w_i.\varphi_{in}}{\sum w_i.\varphi_{in}}\right].v_n$$

In these relations , μ_n , M_n and $S_{a\ n}$ are the modal participation factor , effective mass and the spectral acceleration of n th mode and φ_{in} is form of the n th mode in i th level.

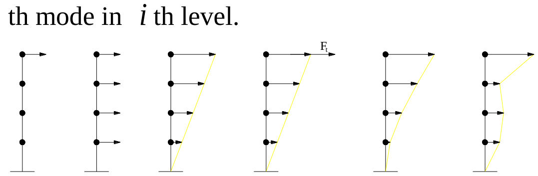


Figure 2- types of the lateral load patterns

4.3- conversion of the capacity curve to the capacity spectrum

As mentioned above, the structure capacity curve is compared with the earthquake spectrum in order to determine the performance point of the structure. For this reason, it is necessary to convert the capacity curve (figure A-3) which is as the base shear ((V) versus the roof drift ((Δ_{roof}) to a capacity equivalent curve (figure 3-B) in the spectral acceleration coordinates ((S_a) versus the spectral displacement ((S_d) (format ADRS).

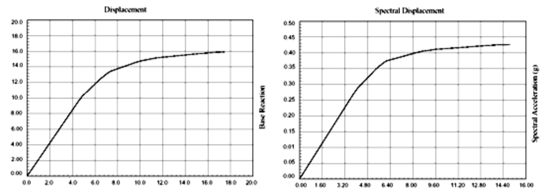
Capacity curve to capacity spectrum curve conversion equations are similar to the modal dynamic analysis equations are used to convert the base shear to the modal force and convert the roof drift to the modal drift. The only difference is replacement of the dynamical mode form with the Pushover mode (drifts of the structure's stories in the maximum response point) [9].

Capacity curve to capacity spectrum curve conversion equations include:

$$S_a = \left(V/W\right)/\alpha_1 \quad , \quad S_d = \Delta_{roof}/\alpha_2 \tag{6}$$

$$\alpha_1 = \left[\sum_{i=1}^{N} w_i\varphi_i\right]^2 \Big/ w\sum_{i=1}^{N} w_i\varphi_i^2 \quad , \quad \alpha_2 = \varphi_{roof}\left[\sum_{i=1}^{N} w_i\varphi_i \Big/ \sum_{i=1}^{N} w_i\varphi_i^2\right] \tag{7}$$

In these relations, W is the total weight of the building, α_1 and α_2 is part of the mass in Pushover mode and is ratio of the Pushover mode drift to the roof drift. w_i , φ_i and φ_{roof} are the weight and i level mode form and Pushover mode form in the roof level.



(a (b)

Figure 3-A: capacity curve, B: capacity spectrum

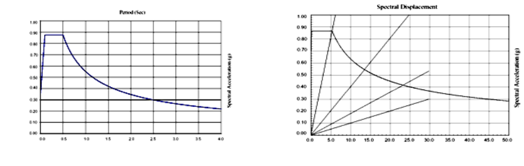
4-4- conversion of the demand spectrum to ADRS format

The demand spectrum or the standard response spectrum which is defined by the codes is drawn as spectral acceleration ((S_a) versus period ((T) (figure 4-A). since we should compare the capacity spectrum with the demand spectrum to calculate the performance point, we should drawn the demand spectrum in terms of ADRS format (figure 4-B). for this purpose, one can use the following relation:

(8)

$$S_d = \left[\frac{T}{2\pi}\right]^2 S_a$$

In ADRS format, the lines which pass the origin have fixed period.



(A) (B)

Figure 4-A: spectral acceleration –period curve, B-spectral displacement - spectral acceleration curve

4- determining the performance point :

After drawing the demand spectrum and capacity spectrum in the coordinates, one can define the maximum potential drift of the structure in the specified earthquake). In case the confluence of the demand spectrum and the capacity spectrum is the elasticity (linear) limit of the capacity spectrum, the confluence is the performance point. But in case the confluence is the nonlinear limit of the capacity spectrum, some corrections are needed because hysteresis damping is created in the structure with entrance of the structure to the nonlinear limit and the viscous damping is added. The demand spectrum presented by the codes was prepared for damping of 5%. The structures viscous damping is the same in the elastic limit. But when the damping increases, some corrections should be made in the demand spectrum due to the nonlinear behavior. This correction is made by applying some factors considering the percentage of damping effective on the demand spectrum. In order to determine these factors, hysteresis damping of the structure should be specified [5].

One of the common methods for determining the hysteresis damping is use of the bilinear capacity spectrum. This method is used to determine damping for a definite spectral displacement ad acceleration ((d_p, a_p) such that a tangent line is drawn with the primary hardness of the structure ((K_i) and then another line is drawn from the desired point ((d_p, a_p) so that the level below the bilinear curve id equal to the level below the primary curve so that the wasted energy of the system remain fixed(figure 5).

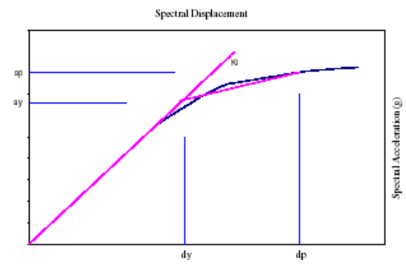


Figure 5- bilinear capacity spectrum

One can calculate hysteresis damping (β_0) using the relation presented by Chopra [12].

(9)

$$\beta_0 = \frac{1}{4\pi}\frac{E_D}{E_S}$$

In this regard, E_D is the lost energy throb damping and E_S is the maximum strain energy.

One can calculate the hysteresis damping (β_0) in percentage using the bilinear capacity spectrum:

(10)

$$\beta_0 = \frac{63.7(a_y d_p - d_y a_p)}{a_p d_p}$$

The damping percentage (β_{eq}) which equals to total viscous and hysteresis damping is calculated by the fooling relation:

(11)

$$\beta_{eq} = \beta_0 + 5$$

Since the energy loss decreases in some structures due to uncompleted hysteresis cycles, therefore, behavior of the building is divided into three classes of A(behavior with stable and acceptable hysteresis cycles) , B(the average structural behavior), and C(the weak structural behavior by decreasing the level below the hysteresis cycles) and suitable hysteresis damping correction efficient (K) has been presented for each one of them . For each structure, effective damping (β_{eff}) can be calculated by the following relation:

(12)

$$\beta_{eff} = k\beta_0 + 5$$

Since the confluence of the reduced demand spectrum with the equivalent damping and capacity spectrum is the performance point and determining the equivalent damping in the above method depends on the performance point, therefore, determining the performance point will be based on trial and error. ATC40 recommends four methods to determine the performance point as follows:

4-5-1- precise method: in this method, value of (β_{eff}) is calculated for each value of (d_p, a_p) on the capacity curve using the capacity bilinear curve method and a pointy which is the confluence (d_p) and the reduced demand spectrum with (β_{eff}) is drawn. The product is a banana shaped curve (figure 6). The confluence of the banana shaped curve is the performance point. In this method, there is no need for trial and error contrary to other methods presented by ATC40 and it is suitable to use it for the computer programs.

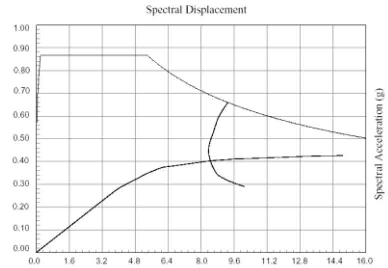


Figure 6- finding the performance point by the precise method

5- Numerical examples

In this section, the moment resistance frame with the cross coaxial bracing has been selected to study role of the load pattern on the performance level of the high structures. The reason for selection of this system was that the code instructions considered some height limitations for the simple braced frames. Therefore, one should use the moment frame system or the braced moment frame for the high structures. Because lateral hardness of the moment frames is relatively low, it is difficult to control the lateral displacement in these frames. Therefore, one of the most suitable systems for the high structures is the braced moment frame system.

Buildings with 5, 15 and 25 stories were selected for study. The frames applied in these buildings have 4 openings with length of 4 m and the lateral openings have bracing. The building plan is regular and height of each story was considered to be 3 m. in order to simplify the critical frame analysis, these buildings were selected and this frame was studied. The loading and primary design of these frames were based on the sixth and tenth issues of the national building regulations. Place of construction was a region with very high seismic risk and heavy snowfall in the site land of type II. According to rules of the sixth issue, design was done in such a manner that the moment frame was able to tolerate at least 25% of the lateral load and total system is able to tolerate the total lateral load. After the primary design, the performance level of the structure was determined in capacity spectrum method and performance point precise determination method for the earthquake with occurrence probability of 100% within 50 years by allocating the plastic conforming to FEMA standard to the elements considering the triangular load pattern in Pushover analysis. In case one or more elements have the performance level exceeding the lateral security, these elements were reinforced until this performance level is achieved. The performance level of the structure was evaluated considering 6 load patters mentioned in section 2-4. Curves relating to calculation of the performance point are shown in figures 7,8 and 9.

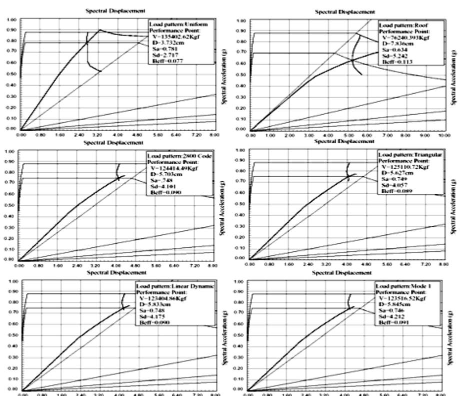


Figure 7- finding the performance point of 5-story structure with the precise method for the different load patterns

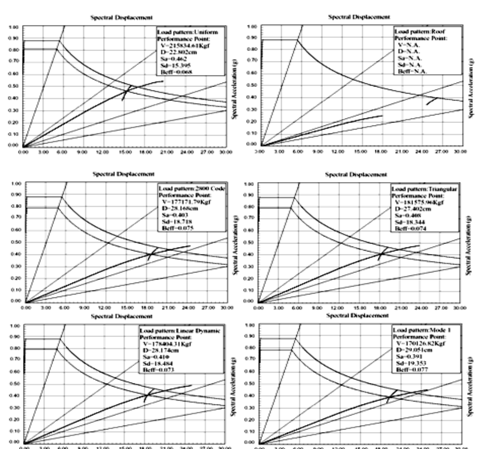


Figure 8- finding the performance point of 15-story structure with the precise method for the different load patterns

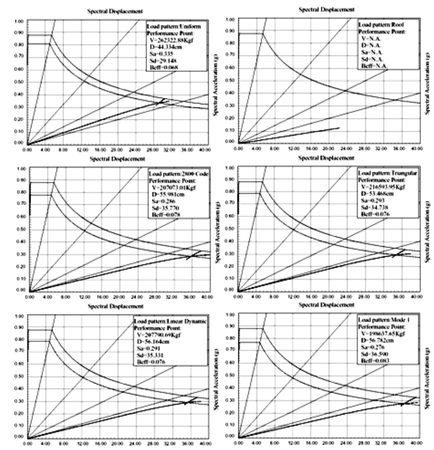
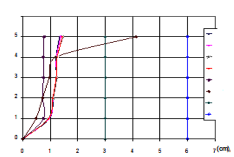


Figure 8- finding the performance point of 25-story structure with the precise method for the different load patterns

In order to determine the performance level, the stories drift was controlled based on ATC40 criteria in addition to status of the elements in the performance point which is seen in figures 10,11 and 12.



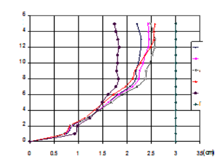


Figure 10-drift of the stories in the 5-story structure
Figure 11- drift of the stories in the 15-story structure

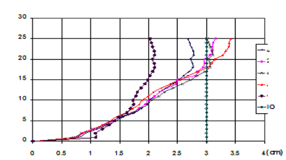


Figure 12- drift of the stories in the 25-story structure

Specifications of the performance point and performance level of each structure are given in tables 1,2 and 3 considering different load patterns.

Table 1- Specifications of the performance point and performance level of 5-story structure

Loading type	Linear dynamics	First mode	Triangular	Earthquake according to code 2800	Uniform	Roof level
Base shear in the performance point	123404	123517	125111	124414	135402	76240
Roof displacement in the performance point	5.83	5.85	5.63	5.70	3.73	7.84
performance point based on the elements status	LS	LS	LS	LS	LS	CP
Performance level based on the drift	IO	IO	IO	IO	IO	LS

Table 2- Specifications of the performance point and performance level of 15-story structure

Loading type	Linear dynamics	First mode	Triangular	Earthquake according to code 2800	Uniform	Roof level
Base shear in the performance point	178404	170126	181576	177172	215835	Without performance
Roof displacement in the performance point	28.17	29.05	27.40	28.17	22.80	Without performance
performance point based on the elements status	LS	LS	LS	LS	LS	Without performance
Performance level based on the drift	IO	IO	IO	IO	IO	Without performance

Table 3- Specifications of the performance point and performance level of 25-story structure

Loading type	Linear dynamics	First mode	Triangular	Earthquake according to code 2800	Uniform	Roof level
Base shear in the performance point	207791	198638	216594	207073	262323	Without performance
Roof displacement in the performance point	56.16	56.78	53.47	55.98	44.33	Without performance
performance point based on the elements status	CP	LS	LS	LS	LS	Without performance
Performance level based on the drift	LS	LS	IO	LS	IO	Without performance

6- conclusion

When the earthquake occurs, dynamic specifications of the structure will change due to formation of the plastic hinges and change of the physical specifications of the elements. Therefore, none of the provided load patterns can indicate real behavior of the structure during earthquake. Considering the fact that almost all dynamic characteristics of the structure are considered in the linear dynamic load pattern and the structure alternate time increasing effects are considered considering the effective damping in Pushover analysis, one can consider this patter as the best pattern and compare other load patterns with this pattern. This comparison is studied for different structures and given in tables 4,5 and 6.

Table 4- comparing the performance point and the performance level of the 5-story structure in different load patterns

Loading type	Linear dynamics	First mode	Triangular	Earthquake according to code 2800	Uniform	Roof level
Ratio of Base shear in the performance point to the spectral loading	1	1.001	1.014	1.008	1.097	0.618
Ratio of roof displacement in the performance point to the spectral loading	1	1.002	0.965	0.978	0.640	1.343

Table 4- comparing the performance point and the performance level of the 15-story structure in different load patterns

Loading type	Linear dynamics	First mode	Triangular	Earthquake according to code 2800	Uniform	Roof level
Ratio of Base shear in the performance point to the spectral loading	1	1.049	0.983	1.007	0.827	Without performance
Ratio of roof displacement in the performance point to the spectral loading	1	0.970	1.028	1.000	1.236	Without performance

Table 6- comparing the performance point and the performance level of the 25-story structure in different load patterns

Loading type	Linear dynamics	First mode	Triangular	Earthquake according to code 2800	Uniform	Roof level
Ratio of Base shear in the performance point to the spectral loading	1	0.956	1.042	0.997	1.262	Without performance
Ratio of roof displacement in the performance point to the spectral loading	1	1.011	0.952	0.997	0.789	Without performance

Study of these tables shows that:

- Loading in the roof level except very short structures (1 and 2 stories) cannot be a suitable pattern because the performance pint is not calculated for the high structures and the obtained performance level for the short structure is different.
- In uniform loading, the error of the performance point increases with increase of height and the obtained performance level for the high structure is different. Therefore, it cannot be a suitable pattern but in study of the critical mode for the low stories of this load pattern.
- In the triangular loading, the error of the performance point increases with increase of height and the obtained performance level for the high structure is different. Therefore, it is a suitable pattern for the short and average structures but it is not recommended for the high structures.
- In loading according to code 2800 (considering the bull whip force), the error of the performance point is very low and when the height increases, it decreases. Therefore, this load pattern is recommended for the high structures and we can justify that effects of the high dynamic modes are considered in the bull whip force.
- In the loading according to the first mode of the performance point is low and when the height increases, it increases. Therefore, this load pattern is not recommended for the high structures and we can justify that effects of the high dynamic modes are low in the short structures and high in the high structures.
- Due to high lateral hardness of the selected system, the determining factor of the performance point of is the status of the plastic hinges created in the elements and controlling the lateral displacement is not effective on the braced structural performance level.

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