



# Analysis of the Behavior of Light Steel Structures Against Nonlinear Static Earthquakes

Ehsan Alipour<sup>1\*</sup>, Amin Mahmudi Moghaddam<sup>2</sup>

<sup>\*1</sup> Master of Civil Engineering. Shams University. Gonbad Kavous, Iran

<sup>2</sup> Assistant Professor of Civil Engineering. Shams University. Gonbad Kavous, Iran

(ehsanalipour403@gmail.com)

(Date of received: 05/03/2024, Date of accepted: 11/06/2024)

## ABSTRACT

The purpose of this research was to analyze the behavior of light steel structures against nonlinear static earthquakes. Among the simplified nonlinear methods to be included in the next generation of regulations, nonlinear static analysis simply simulates the nonlinear behavior of structures, and as an efficient tool in seismic evaluation of buildings, it has attracted the attention of researchers. has done. Today, in order to make structures as strong as possible against earthquakes, various methods are used. One of the effective methods in improving the seismic behavior of structures is the use of light steel structural systems (LSF). These types of systems receive a small amount of lateral force during severe earthquakes, and due to two special properties, i.e. prefabricated structural members and suitable thermal insulation, this structural system is widely used. To be used in the developed countries of the world to build residential houses. Due to the fact that in the seismic design of these structures in the traditional way, the design of connections is not properly considered, in these structures, connections are considered as the main weak point of these structures during severe earthquakes. This research has studied the seismic behavior of the walls of the LSF system and its connections using the non-linear static analysis method in the ABAQUS finite element software. For this purpose, plans with the number of layers 4, 6 and 8 have been selected and according to the 2800 code of Iran earthquake. A static equivalent was designed. Finally, in the case of the buildings designed according to the 2800 regulations, the research findings showed: Non-linear static analysis is not accurate enough in comparison with non-linear dynamic analysis.

## Keywords:

Light steel structures, Earthquake, Non-linear static earthquake, Target displacement, Capacity.



## 1. Introduction

In recent years, considering the economic value of existing buildings, efforts have been made to examine the stability of buildings from the perspective of the minimum requirements, and in this regard, retrofitting guidelines have been developed. According to the philosophy of these guidelines, the safety factors considered in the design regulations should generally be greater than the safety factors of these guidelines. Considering the above discussion, it is expected that structures designed in accordance with the usual design regulations will also be approved by the retrofitting guidelines. In fact, the methods of retrofitting structures that are described in detail in guidelines such as FEMA356 and the Iranian retrofitting guidelines can be considered more accurate methods that are consistent with the design of existing structures. Review of Analytical Methods and Acceptance Criteria of the Reinforcement Instructions. In this section, we review the general analysis criteria, analysis methods including linear static, linear dynamic, nonlinear static, nonlinear dynamic methods, and member acceptance criteria in each of these methods from the perspective of the Reinforcement Instructions.

## 2. Modeling

### 2.1. Initial Assumptions

The structure should be modeled in three dimensions. In the cases mentioned in this section, a two-dimensional model can also be used for nonlinear analyses. If the structure has a rigid diaphragm and torsional effects are considered in the structure, a two-dimensional model can be used in nonlinear analyses. When the structure is modeled in two-dimensional nonlinear analyses, the three-dimensional properties of the structural components and members must be considered to calculate the stiffness and strength. In nonlinear analyses, if the connections are weaker or have less ductility than the connected members, or if it is estimated that the results will change by more than 10% by considering the connections in the model, their effect must be appropriately included in the structural model.

### 2.2. Main and non-main members

The structural members that are effective in the lateral stiffness or distribution of forces in the structure or are affected by the lateral displacement of the structure are divided into two groups: main and non-main. Main members are the members that are intended to resist the collapse of the building due to an earthquake. Other members that are not intended to withstand lateral loads compared to the main members are known as non-main members. These members may even be subjected to lateral loads. The main members should be evaluated for earthquake forces and deformations in combination with gravity loads and the non-main members should be evaluated for earthquake deformations in combination with gravity loads

### 2.3. Classification of Deformation and Force Controlled Components in Steel and Concrete Structures

In this section, we will explain the classification of deformation and force controlled components in steel structures. The classification of steel building components into deformation and force controlled components in steel dual system frames is as per Table (1).



**Table 1.** Classification of components controlled by force and deformation in steel double system frames [1]

Controlled by Force or Shapeshifting	Related Effort	Member
the force	Compressive axial force	Column
shape change	Tensile axial force	Column
the force	Shear	Column
shape change	Bending	Column
shape change	Bending	Beam
the force	Shear	Beam
the force	Torsio	Beam-to-column connection
the force	-	Column

### 3. Structural Analysis Methods

In order to estimate the internal forces and deformations of structural components due to an earthquake of the selected hazard level, it is necessary to analyze the structure using one of the following methods.

- 1- Linear static analysis method
- 2- Linear dynamic analysis method
- 3- Nonlinear static analysis method
- 4- Nonlinear dynamic analysis method

#### 3.1. Linear Static Analysis

To use the linear static analysis method, the limitations mentioned in section (2-3) of the rehabilitation instructions must be taken into account. The basic assumptions of the method are:

- 1 - The behavior of the materials is linear.
- 2 - The loads resulting from the earthquake are constant (static).
- 3 - The total force acting on the structure is equal to a coefficient of the weight of the building.

In this method, the lateral force resulting from the earthquake is selected in such a way that the resulting base shear is equal to the base shear force according to equation (3-6) of the rehabilitation instructions. The base shear value in this method is chosen so that the maximum deformation of the structure is consistent with that predicted in an earthquake of the desired hazard level. If the structure behaves linearly under the effect of the applied load, the forces obtained for the structural members will also be close to the predicted values during the earthquake, but if the structure behaves nonlinearly, the forces calculated in this way will be more than the values of material displacement. Therefore, when examining the acceptance criteria in section (3-4-1) of the Construction Manual, the results of the linear analysis are modified for structures that behave nonlinearly during the earthquake.



### **3.2. Linear Dynamic Analysis**

Linear dynamic analysis can be performed in two spectral or time history methods. The specific assumptions of this method in the range of linear behavior are:

- 1 - The behavior of the structure can be calculated as a linear combination of the states of the different vibration modes of the structure that are independent of each other.
- 2- The period of vibration of the structure in each mode is constant during the earthquake.

In this method, similar to the linear static analysis method, the response of the structure in an earthquake of the desired hazard level is multiplied by coefficients according to paragraph (3-3-2-4) of the improvement instructions so that the maximum deformation of the structure matches that predicted in the earthquake. For this reason, the internal forces in ductile structures that will have nonlinear behavior during an earthquake are estimated to be greater than the tolerable forces in the structure. For this reason, when examining the acceptance criteria in section (3-4-1) of the improvement instructions, the results of the linear analysis for structures that have nonlinear behavior during an earthquake are modified. The limitations of using this method are stated in section (2-3) of the improvement instructions.

### **3.3. Nonlinear Static Analysis**

In this method, the lateral load caused by the earthquake is applied statically and gradually increasing to the structure until the displacement at a specific point (control point), under the effect of the lateral load, reaches a certain value (target displacement) or the structure collapses.

## **4. Material and Method**

In this chapter, by presenting initial assumptions regarding geometry, configuration, and loading, initial models are prepared and used for analysis and design in accordance with the criteria of "Code of Design of Buildings Against Earthquakes - Iran Standard 2800" and Section 6.

### **4.1. Definitions of the Initial Models**

In order to compare the design results of steel buildings with coaxial steel bracing based on the criteria of the Iranian Standard 2800 and the evaluation of buildings based on the criteria of the "Guidelines for Seismic Improvement of Existing Buildings" and also to compare the results of the evaluation of models using the nonlinear static and nonlinear dynamic methods as the base method, 3 symmetrical and regular 4, 6, 8-story steel models with a dual system of cross bracing and a medium-flexural frame and 3 symmetrical and regular 4, 6, 8-story steel models with a dual system of convergent 7-shape bracing and a medium-flexural frame have been prepared.

For all three models:

- \*The size of the openings in both directions is 14 meters.
- \*The height of the ground floor is 3 meters and the floors are 3.2 meters.
- \*The building system in both directions for the 2 models of the dual system of cross bracing and a medium-flexural frame and 3 models of the dual system of braces 7 is a medium-sized bending frame (Figures 1 and 2).

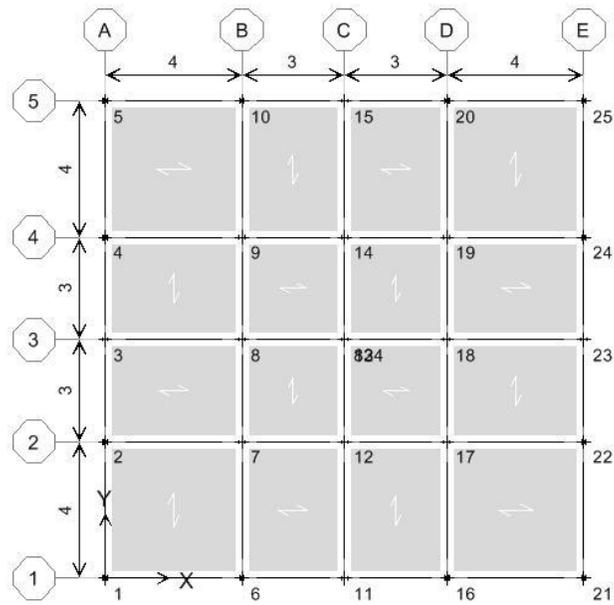


Figure 1. Plan of designed models.

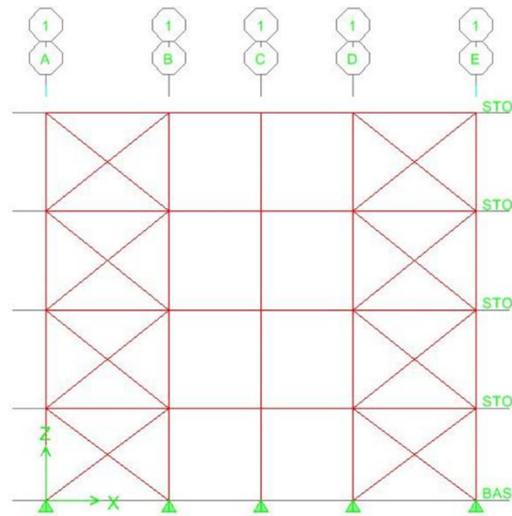


Figure 2. How to place braces in spans 1 and 5

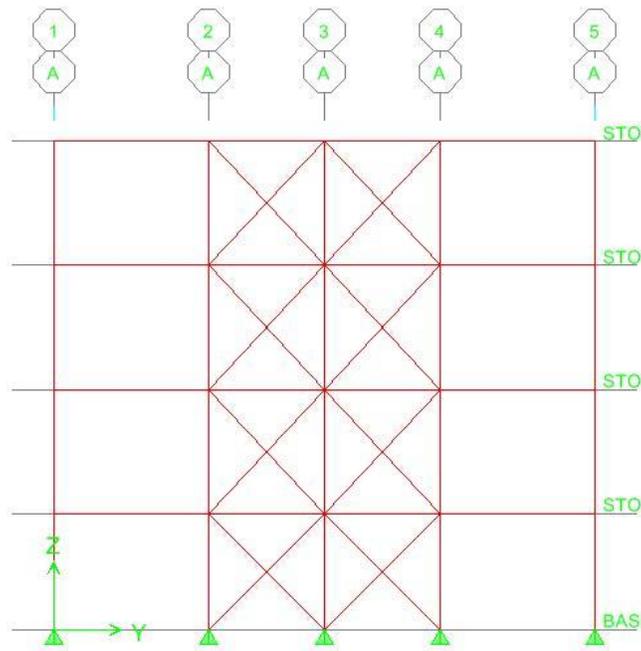


Figure 3. How to place the windbreak in the opening A,E

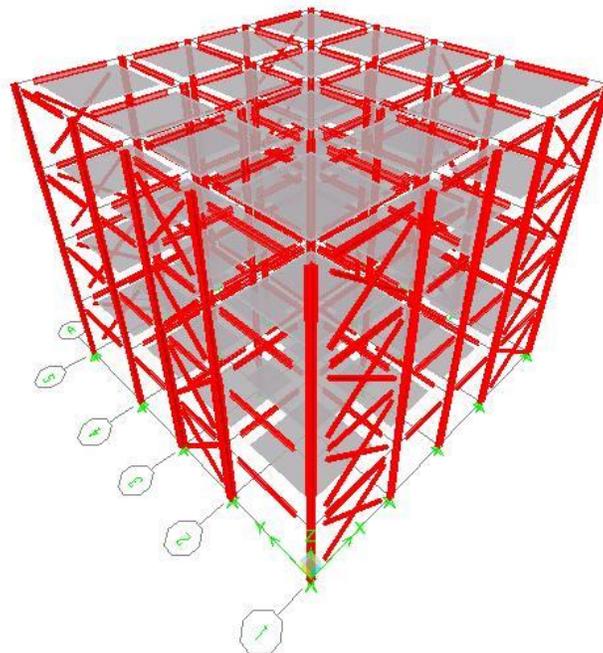


Figure 4. 3D view of a 4-story building



#### 4.2. Gravity Loading

In order to calculate dead and live loads in the above buildings, the values of gravity loads were taken into account according to the Iranian Loading Regulations as shown in Table 2.

**Table 2.** Live and dead load values in gravity loading

Load type	Load Description	Load intensity	Unit	Total
Dead floors	Floor Load	540	Kg/m <sup>2</sup>	640
	Equivalent Load of Slating	100	Kg/m <sup>2</sup>	
Dead roof	Floor Load	540	Kg/m <sup>2</sup>	620
	Putting and Bituminizing	80	Kg/m <sup>2</sup>	
Live	Floors	200	Kg/m <sup>2</sup>	200
Live	Roof	150	Kg/m <sup>2</sup>	150
Dead	External Wall Load	800	Kg/m	800
Dead	Parapet Load	240	Kg/m	240

#### 4.3. Seismic Loading

To consider the effects of earthquake loading, the equivalent static loading method has been used. The values of seismic parameters are selected as follows.

- Base acceleration of the design: Zone 2 - High relative risk area,  $A=0.3g$
- Soil type: ( $T_0=0.1$ ,  $S=1.5$ ) Type 2 soil
- Importance factor: Group 3 - Medium importance building,  $I=1.0$

Now the values of the experimental periods, earthquake coefficient, in floors have been calculated for all four models and are shown in the following tables

**Table 3.** Parameters used in seismic loading of a 4-story building

Parameter	value	unit
A	0/3	g
I	1	-
R	7	-
H	12/6	m
$T=0.05H^{(3/4)}$	0/33	sec
S	1/5	-
B	2/5	-
$C=ABI/R$	0/125	-



**Table 4.** Parameters used in seismic loading of 6-storey building

Parameter	value	unit
A	0/3	g
I	1	-
R	7	-
H	25/4	m
$T=0.05H^{(3/4)}$	0/56	sec
S	1/5	-
B	2/5	-
$C=ABI/R$	0/125	-

**Table 5.** Parameters used in seismic loading of 8-storey building

Parameter	value	unit
A	0/3	g
I	1	-
R	7	-
H	38.2	m
$T=0.05H^{(3/4)}$	0/78	sec
S	1/5	-
B	1/8	-
$C=ABI/R$	0/09	-

## 5. Material and Method

The ETABS ver 9.1.2 computer software was used to model, analyze, and design the assumed models. It is worth noting that the aforementioned software has the ability to analyze and design three-dimensional building structures in accordance with the world's authoritative regulations [3].

### 5.1. Analysis

After determining the gravity forces, seismic loads, and also preparing structural models, these models are analyzed and subjected to force analysis. It is noteworthy that P- $\Delta$  analysis is performed by default for all models.

### 5.2. Design

In order to take into account, the seismic design criteria for the design of steel structures, the UBC97-ASD regulation was used, and because the design load combinations of this regulation do not match the load of the Iranian regulation 2800, we enter the load combinations as follows in the Define > load combinations section [5].

DL+LL

0.75(DL+LL+EL)

0.75(DL+LL-EL)

0.75(DL+EL)

0.75(DL-EL)

The profiles selected for the beams are IPE type, and the columns for the 4-story building are double IPE type and for the 6 and 8-story buildings are BOX type. The values of the stress ratio in the design have been tried to be around 0.7 to 1.



### 5.3. Method

The nonlinear thrust static analysis method consists of two main stages, in the first stage the target displacement is determined and in the second stage lateral forces are applied to the structure in a thrust manner, this force continues until the target displacement is achieved. The methods that have been considered by researchers to determine the target displacement are the displacement coefficient method and the capacity spectrum method. These two methods are examined below and the results are compared [6]. The introduced buildings, after loading, analyzing, and finalizing the member sections, were subjected to nonlinear static analysis in the order mentioned below, and their base section-roof displacement diagrams were drawn under different loading patterns.

Since the goal is to examine the criteria of the 2800 regulations, the improvement of the foundation in which the goal is to satisfy the life safety performance level in an earthquake of hazard level-1 was selected.

Since the way the lateral load is distributed in nonlinear static analysis has a great impact on the accuracy of the results, choosing the appropriate model for lateral load distribution is of particular importance. Three different loading patterns were used in conducting these analyses.

1- Gravity loading of the structure to provide initial conditions in the analysis under the effect of lateral forces (Gravity)

2- Loading in accordance with the equivalent static load of Regulation 2800, this pattern is the result of the lateral force distribution proposed in the Building Design Regulations against Earthquakes (Figure 4-1). In this method, the lateral force of the earthquake is determined based on the main period of oscillation of the building and using the reflection spectrum of the design. (Push2)

3- Uniform loading proportional to the floor mass (Push4).

In this type of distribution, the shear load of the building base is applied equally to the center of mass of the floors.

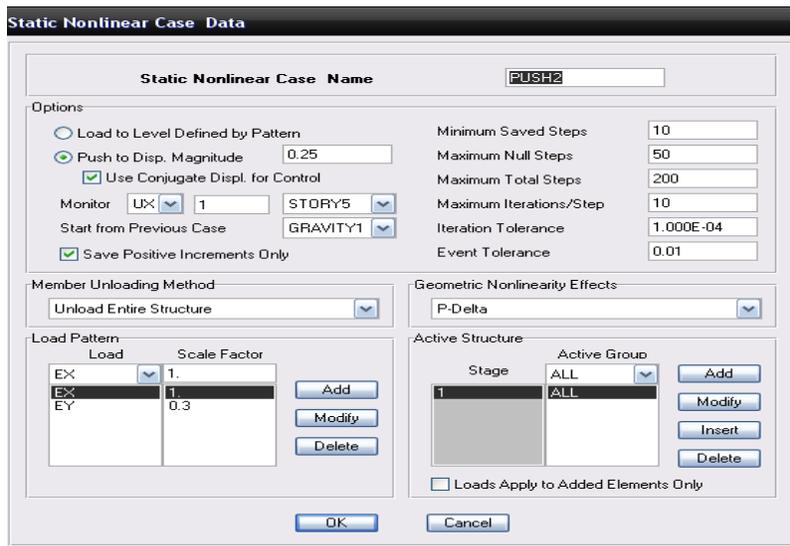


Figure 5. Window for defining lateral load distribution pattern in nonlinear static analysis.



The nonlinear behavior of the structure is applied by defining plastic joints in the model. Any nonlinear analysis requires the introduction of nonlinear joint specifications. These joints are defined at points of the structural components where there is a possibility of the internal forces of the member exceeding the yield strength. In the nonlinear analysis of the ETAB&SAP program, the introduced loading pattern is applied in stages and each time the stiffness matrix is modified based on the deformed geometry. In addition, in each stage, the amount of internal force of the members at the defined joint location is compared with the amount of the introduced joint yield strength. If the internal force has not reached the yield strength, some amount is added to the structural load while maintaining the stiffness of the previous stage, but if the internal force has exceeded the joint yield strength, the stiffness of the joint and consequently the stiffness of the structure are modified according to the force-displacement curve. This staged loading continues until the mechanism limit is reached or the displacement criterion specified by the user is reached. In this way, the program generates the force-displacement curve of each section based on their geometric characteristics and the presented pattern. The failure of a beam in a flexural frame is due to the increase in the moment and shear values at both ends of the beam, resulting in the occurrence of plastic bending and shear joints at both ends. Therefore, shear joints V2 should be assigned to the two ends of the beam and flexural joints M3 to the middle of the beam. In columns, axial joints P and PMM joints can be formed, so we assign these joints to the two ends of the column. Modeling parameters and acceptance criteria for introducing joints in nonlinear methods are considered according to Table (3) of the Seismic Improvement Guidelines (Figure 6) [7].

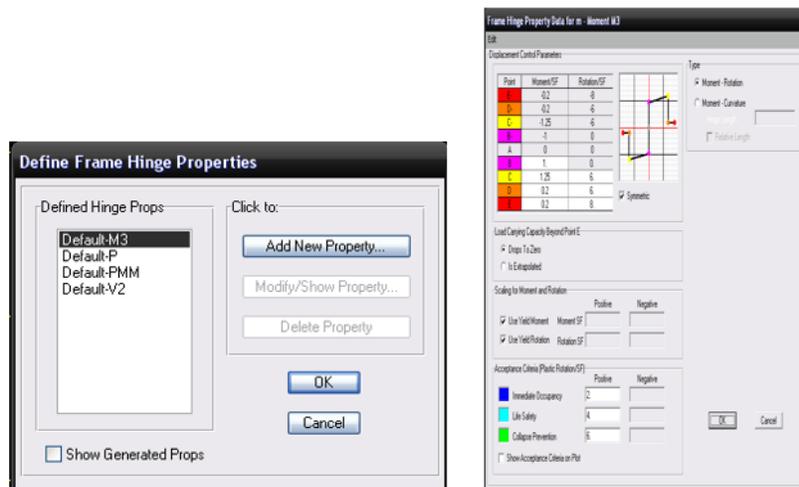


Figure 6. Windows related to the definition of joint characteristics

The Federal Emergency Management Agency, FEM, has provided recommendations in the field of nonlinear joints, and SAP2000ver11 & ETABSver9 programs also use the FEME356 regulations for the shape of nonlinear joints. The shape of the joints depends on the type of joint (flexural, shear, axial, etc.) and the type of member, and each of the joints has nonlinear behavior in one of the force components. The properties of steel joints according to FEME356 are mentioned below [8].



In modeling, all main and non-main members should be included and their nonlinear behavior should be modeled as close to reality as possible. If there is a reduction in its effects, it should also be included in the member behavior model.

In addition, the following should be included in the analysis.

- 1- Ground acceleration should be selected according to Regulation 2800.
- 2- Structural analysis should be performed for at least three accelerometers in each direction.
- 3- Earthquake effects in two perpendicular directions should be considered according to Regulation 2800.
- 4- The structural response should be calculated for each of the accelerometers. If less than 7 accelerometers are considered in each direction, the structural response should be assumed to be equal to the maximum response value. However, if 7 or more accelerometers are considered in each direction, the structural response can be selected to be equal to the average response value.

Nonlinear dynamic analysis is the most accurate method used in structural analysis. Nonlinear dynamic analysis is performed by two general methods: direct integration and modal analysis. The direct integration method also includes various methods such as the Hubolt method, the mean difference, Wilson  $\theta$ , and Newmark. One of the methods that is generally used in the analysis of arbitrary nonlinear systems is the step-by-step integration of the coupled equations of motion. In this method, the analysis of multi-degree-of-freedom systems is performed exactly the same as the analysis of linear single-degree-of-freedom systems. The response function is divided into short and equal time intervals and during each time interval, the response is calculated for a linear system with the characteristics determined at the beginning of the time interval. At the end of each time interval, the characteristics of the system are restored to match the deformation and stress state at that time. Therefore, nonlinear analysis is approximated by a sequence of linear systems that change in sequence. The stepwise integration method is also applicable to linear systems, in which case the computational procedure is considerably simplified because the structural properties do not need to be modified at each time step. In some cases, the use of this direct integration method is more useful than the superposition of modes because it does not require the determination of mode shapes and frequencies, which is a very large computational task in systems with many degrees of freedom. In general, stepwise integration is the best method for determining the response of large and complex structures that are subjected to short-term shock loads and tend to excite many vibration modes, and at the same time the response of the structure is required for a short period of time.

### ***5.3.1. Analysis by Nonlinear Dynamic Method***

the presented models are analyzed using the nonlinear dynamic time history method (direct integration method).

Accelerograms selected for nonlinear dynamic analysis should have characteristics that are consistent with the structure site as much as possible. These characteristics include PGA, frequency content, duration of extreme movements, and consistency with the design spectrum[14]. In order to use accelerograms in nonlinear dynamic analysis, the spectrum of this accelerogram should be consistent with the design spectrum of the structure site as much as possible. In fact, before using accelerograms, they must be aligned. The method of aligning accelerograms is given below.



This method is the simplest method of aligning accelerograms. The method is that all accelerograms are divided by their PGA and all are multiplied by the base acceleration value of the area design. In this way, we will have accelerograms that have the same PGA and are equal to the base acceleration of the area design. As stated, the selected accelerograms must also be compatible with the area design spectrum, but in this method, only their acceleration value is compatible with the area, and the other parameters such as energy, frequency content, etc. may not be compatible with the area spectrum.

In this method, the maximum acceleration of each of the accelerograms is scaled to 1g. Then, the response of the structure with one degree of freedom against this accelerogram (accelerogram spectrum) is calculated, and the area under this spectrum between the periodic periods of 0.1 and 3 seconds is obtained. The area under the curve of the design spectrum between these two periodic periods is also calculated. By multiplying the accelerogram scaled to 1g by the ratio of the area of the construction site design spectrum to the area of the acceleration spectrum and finally by the acceleration of the construction site design, the scaled accelerogram is obtained with the area. In this method, the energy of the accelerograms is compatible with the design spectrum.

## 6. Results and Discussion

This section deals with determining the performance level and predicting the behavior of the structures designed in Chapter 3 in accordance with the seismic improvement guidelines and achieving the objectives of Regulation 2800 using the nonlinear static and nonlinear dynamic methods as the base method.

### 6.1. Determining the Target Displacement using the Displacement Coefficient Method

The target displacement determination procedure, which was mentioned in detail in Chapter 2, has been used to estimate the maximum possible displacement during ground motion. Before referring to the process of determining the target displacement, it is necessary to mention one important point. In the FEME356 report, two methods have been proposed to determine the effective period of the structure: first, using an empirical relationship and second, using the period calculated based on the linear dynamic analysis of the structure. However, in the ATC40 guideline, only the period calculated based on the dynamic analysis is considered as a criterion. In Table (4-1a and b), the target displacement values of the studied samples using the displacement coefficient method in accordance with the seismic improvement guidelines are presented.



**Table 6.** Target displacement using the displacement coefficient method in accordance with the improvement guidelines Table (4-1-b): Target displacement using the displacement coefficient method in accordance with the improvement guidelines

Target displacement (cm)	S <sub>a</sub>	C <sub>3</sub>	C <sub>2</sub>	C <sub>1</sub>	C <sub>0</sub>	T <sub>e</sub>	Building type
4	0.75	1	1.1	1	1.42	0.791	(Dual system with 7-shaped braces)
18.25	0.75	1	1.1	1	1.49	1.184	4 floors
27.59	0.54	1	1.1	1	1.58	1.558	6 floors

### 6.2. Determining the Target Displacement by the Capacity Spectrum Method

curve with the response spectrum. In this method, the structure is analyzed under the Pushover lateral load pattern, and the lateral loading occurs at points of the structure with a step-by-step increase in the yield (plastic hinge). Initially, when the system is still in a linear state, the damping ratio of the structure is considered to be 5% as usual, and gradually, as plastic hinges are created at different points, the damping of the structure increases, and the hysteresis curve is drawn based on the plastic joints created at that stage, and the damping hysteresis loop is determined proportionally to the area. Then, based on the obtained damping, the spectrum is corrected, and the status of the two capacity and spectrum curves is checked in the ADRS coordinate system. If the two curves intersect, the target displacement is obtained. For this purpose, SAP2000ver11 software can be used, which has the ability to transfer the spectrum introduced by the user to the ADRS coordinates. It is also possible to intersect the capacity curve of the structure in the EXCEL environment with the spectrum of the 2800 code (Figure 7) and obtain the displacement corresponding to the performance point of the structure. In Table (6), the target displacement values of the studied samples using the capacity spectrum method are presented in accordance with the interpretation of the seismic improvement manual.

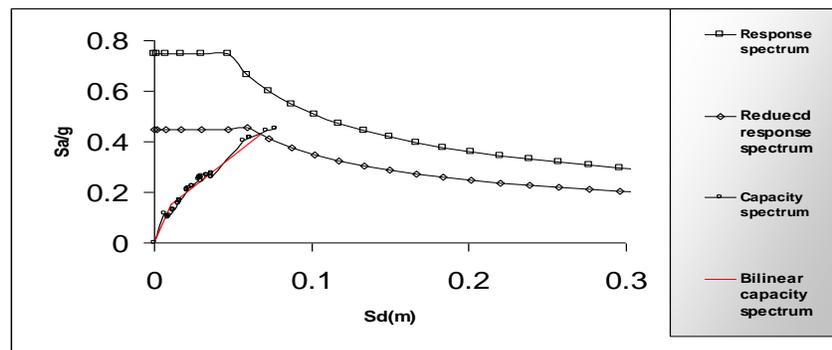


Figure 7. Determining the operating point using the capacity spectrum method

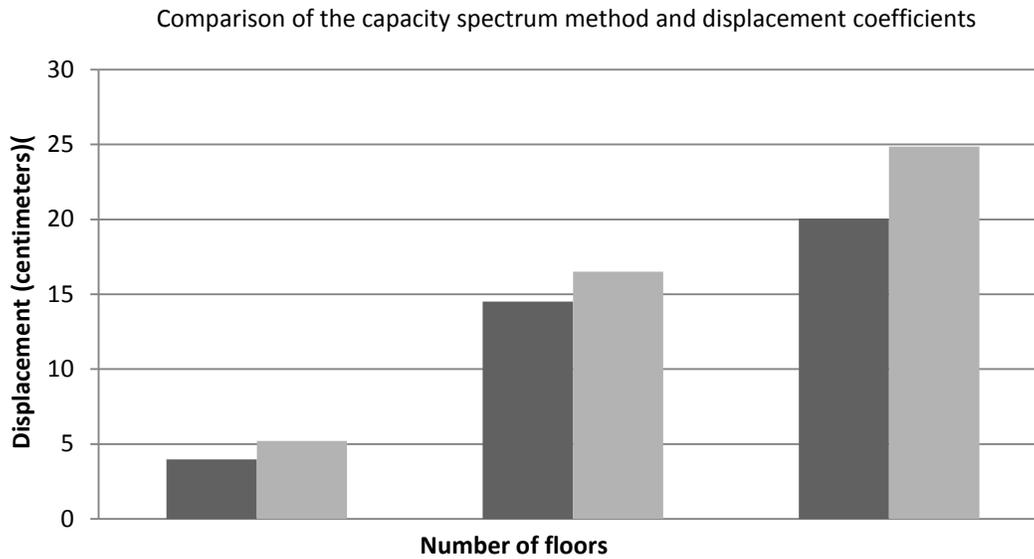


Figure 8. Comparison of target displacement in displacement coefficient and capacity spectrum methods in a dual system building with double-bracing and double-bracing

Table (7) shows the maximum relative displacement values for each model, along with the floor corresponding to this displacement, under different loading patterns. These values will be used in the building performance evaluation section, which will be discussed at the end of this chapter.

**Table 7.** Maximum values of relative displacement in pushover analysis

Improvement method		Number of floors		
		4 floors	6 floors	8 floors
Cross brace system	push2	0.0028	0.0056	0.0063
	push4	0.0028	0.0053	0.0065
System with 7-shaped braces	push2	0.0031	0.0062	0.0068
	push4	0.0032	0.0060	0.0073

### 6.3. Nonlinear Dynamic Analysis

In this section, the models presented in Chapter 3 are analyzed using the nonlinear dynamic time history method described in Chapter 2. The response of the structure model under ground acceleration excitation should also be calculated based on at least three accelerograms. If less than 7 accelerograms are selected for analysis, their maximum effect should be considered to control deformations and internal forces. If seven or more accelerograms are used, their average value can be considered to control deformations and internal forces.



### **6.3.1. Accelerograms used in nonlinear dynamic analysis**

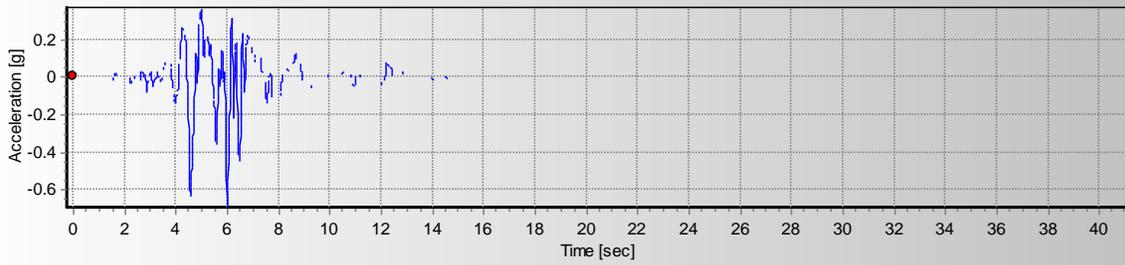
Accelerograms selected for nonlinear dynamic analysis should have characteristics that are consistent with the structure site as much as possible. These characteristics include PGA, frequency content, duration of extreme movements, and consistency with the design spectrum. In order for accelerograms to be used in nonlinear dynamic analysis, the spectrum of this accelerogram should be as consistent as possible with the design spectrum of the structure site. In fact, before using the accelerograms, they must be aligned. The method of aligning the accelerograms is given below.

### **6.3.2. PGA Alignment**

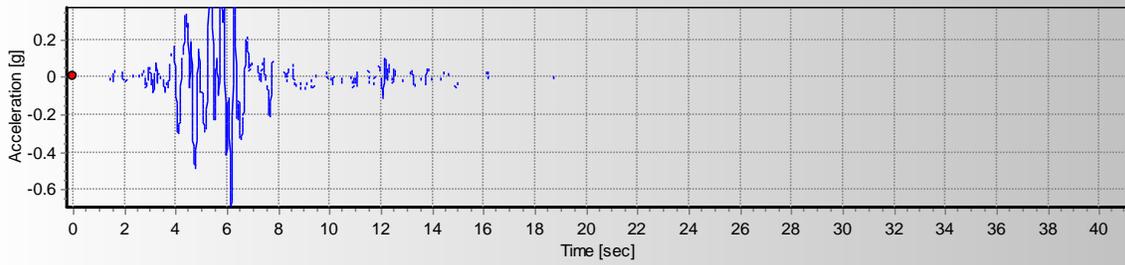
This method is the simplest method of aligning the accelerograms. The method is that all the accelerograms are divided by their PGA and all are multiplied by the acceleration value of the base area plan. In this way, we will have accelerograms that have the same PGA and are equal to the acceleration value of the base area plan. As mentioned, the selected accelerograms must also be compatible with the area plan spectrum, but in this method, only their acceleration value is compatible with the area, and the other parameters such as energy, frequency content, etc. may not be compatible with the area spectrum.

### **6.3.3. Spectrum Alignment**

In this method, the maximum acceleration of each of the accelerograms is scaled to 1g. Then, the response of the structure with one degree of freedom to this accelerogram (accelerogram spectrum) is calculated and the area under this spectrum between the 0.1 and 3 second periods is obtained. The area under the design spectrum curve between these two periods is also calculated. By multiplying the scaled accelerogram to 1g by the ratio of the site design spectrum area to the accelerogram spectrum area and finally by the site design acceleration, the scaled accelerogram with the area is obtained. In this method, the energy of the accelerograms is adapted to the design spectrum. As mentioned earlier, the design of the initial models was based on soil type 2, so the accelerograms were selected in such a way that they are compatible with the desired site. In Figures (4-5, 4-6, and 4-7) and Table (4-4), the accelerograms and their related specifications are given. To use these accelerograms in nonlinear dynamic analysis, the desired accelerograms have been scaled using the spectrum matching method using the Seismosignal software.

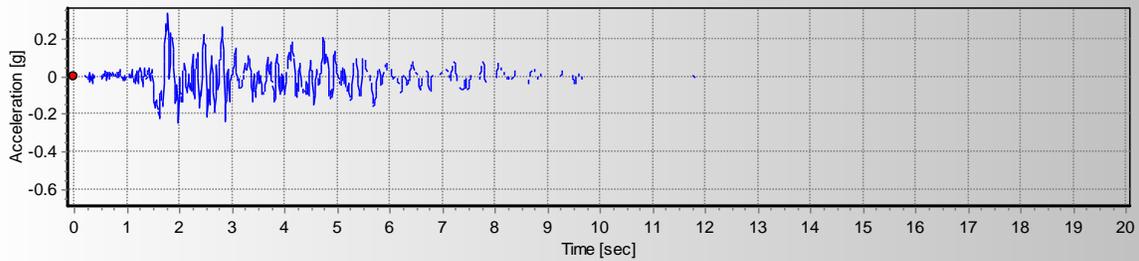


A) Accelerogram of the Kobe earthquake in the X direction

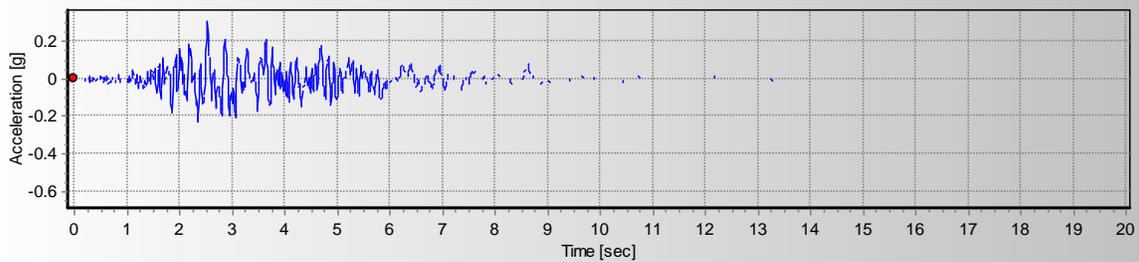


B) Accelerogram of the Kobe earthquake in the Y direction

Figure 9. Accelerogram of the Kobe earthquake.

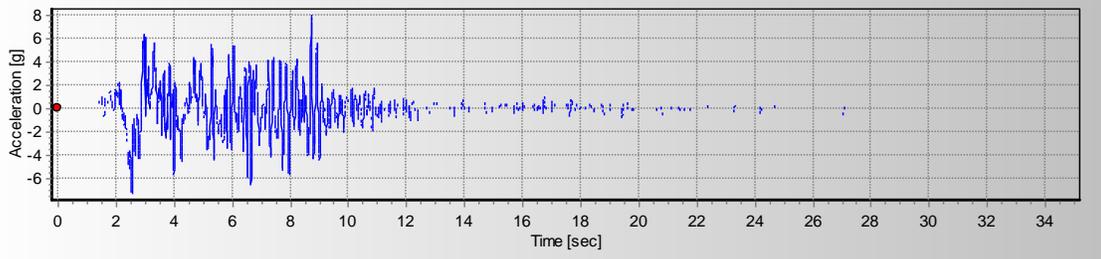


A) Northridge earthquake accelerogram in the X direction

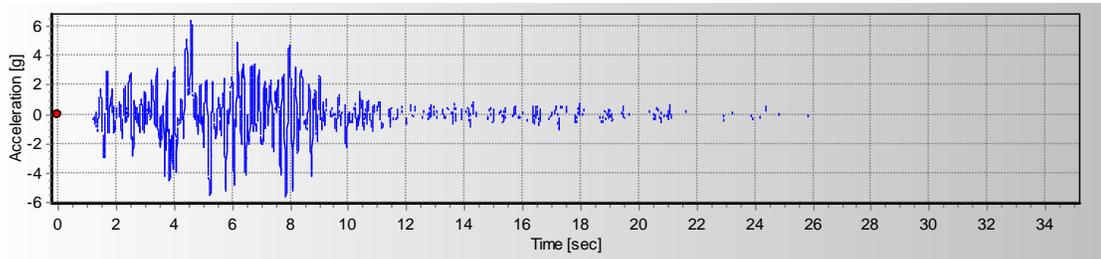


B) Northridge earthquake accelerogram in the Y direction

Figure 10. Northridge Earthquake Accelerogram.



A) Accelerogram of the Bam earthquake in the X direction



B) Accelerogram of the Bam earthquake in the Y direction

Figure 11. Bam earthquake accelerometer

#### **6.4. Relative displacement of floors in nonlinear dynamic analysis**

One of the tangible and measurable parameters for assessing the vulnerability of structures is the maximum relative displacement of floors. Most earthquake codes consider this parameter as a factor controlling the damage of buildings. In general, the limit of relative displacement of floors depends on factors such as the seismic hazard of the location, the number of floors of the building, and the degree of importance of the structure. To determine the relative displacement in the nonlinear dynamic analysis method, the response history of a point from each floor is obtained as a representative of the floor for each earthquake. By subtracting the response history of this point from the response history of the upper floor, the relative displacement history (Drift) of the upper floor is determined. The maximum value of this history is considered as the relative displacement of the floor in absolute value. By dividing this value by the height of the floor, the Drift Ratio is determined.



**Table 8.** Maximum values of relative displacement for each earthquake and building

			<i>KOBE</i>	<i>NORTHRIDGE</i>	<i>Barr</i>
Dual-frame system with double-brace	4 floors	Relative Position Change	0.0087	0.0074	0.009
	8 floors	Relative Position Change	0.0135	0.0124	0.0118
	16 floors	Relative Position Change	0.0158	0.0132	0.0129
Dual-frame system with V-shaped brace	4 floors	Relative Position Change	0.0092	0.0085	0.0097
	8 floors	Relative Position Change	0.0148	0.0133	0.0115
	16 floors	Relative Position Change	0.0168	0.0131	0.0155

### **6.5. Determining the level of performance in nonlinear static analysis**

The level of performance in nonlinear static analysis for the three buildings studied is given in Tables (9, 10, and 11).

**Table 9.** Determining the level of performance in improvement design methods (4 floors)

Improvement Design Method	Building with dual system of flexural frame with cross braces		Building with dual flexural frame system with figure 7 braces	
	PUSH2	PUSH4	PUSH2	PUSH4
Maximum Drift	0/28	0/28	0.31	0.32
Control	0.5 < 1.5		1.5 < 0.5	
Structure Performance	IO		IO	



**Table 10.** Determining the level of performance in improvement plan methods (8 floors)

Improvement Design Method	Building with dual system of flexural frame with cross braces		Building with dual flexural frame system with figure 7 braces	
Loading Pattern	PUSH2	PUSH4	PUSH2	PUSH4
Maximum Drift	0/56	0/53	0.62	0.60
Control	0.5 < 1.5		1.5 < 0.5	
Structure Performance	LS		LS	

**Table 11.** Determining the level of performance in improvement plan methods (12 floors)

Improvement Design Method	Building with dual system of flexural frame with cross braces		Building with dual flexural frame system with figure 7 braces	
Loading Pattern	PUSH2	PUSH4	PUSH2	PUSH4
Maximum Drift	0.63	0.65	0.68	0.73
Control	0.5 < 1.5		1.5 < 0.5	
Structure Performance	LS		LS	



**Table 12.** Determining the level of performance in nonlinear dynamic analysis History

	Structure with Zharf Dari bracing system			Structure with V-shaped bracing system		
	4 floors	8 floors	16 floors	4 floors	8 floors	16 floors
<b>Maximum Drift 3 Accelerometer</b>	0/9	1/35	1/58	0/97	1/48	1/68
<b>Control</b>	1.5 >> 0.5	1.5 >> 0.5	1.5 >> 0.5	1.5 >> 0.5	1.5 >> 0.5	1.5 >> 0.5
<b>Structural performance</b>	LS	LS	CP	LS	LS	CP

By comparing the results of nonlinear static and nonlinear dynamic analyses, the methods (displacement coefficients and capacity spectrum) are not sufficiently accurate in estimating displacements. However, they can be relied upon as a baseline method in evaluating the performance level of a structure compared to nonlinear dynamic time history analysis. An important weakness of these methods, which is due to the assumptions governing nonlinear static analysis, is manifested in structures with long period periods, such as tall buildings, where the effect of higher modes has a significant effect on their seismic behavior, and that is the inability of the method to identify the failure mechanism caused by higher modes. This issue could be related to the failure to consider the effect of higher modes in the lateral load distribution pattern used in the nonlinear static method. Finally, according to the above tables, it can be concluded that structures designed based on Iranian Code 2800, third edition, for this type of structural system hardly satisfy the life safety performance level.

## 7. Conclusion

1- Although displacement coefficient and capacity spectrum methods based on nonlinear static analysis are not accurate enough in estimating displacements compared to nonlinear dynamic time history analysis as the base method, they are accurate enough in evaluating the performance level of the structure and can be relied upon.

2- By comparing the analysis results obtained from the designed buildings, it is concluded that the accuracy of nonlinear static analysis decreases with increasing height. This issue could be related to the failure to consider the effect of higher modes in the load distribution pattern used in the nonlinear static method (because in tall buildings, high modes have a significant effect).

3- According to the results of nonlinear dynamic analysis, the short-story building is within the performance level of uninterrupted service of the medium-story building, and the high-rise building is barely within the life safety limit.

4- According to the results of nonlinear dynamic analysis, the Bam earthquake had the greatest impact on short structures and the Kobe earthquake had the greatest impact on tall structures, and



with increasing structure height, the effect of selective accelerometer mapping on the results of nonlinear dynamic analysis increases.

5- According to the nonlinear dynamic analysis, it is observed that nonlinear static analysis for high-rise structures with a dual system of a 7-shaped moderate bending frame does not have the appropriate accuracy and the structure is in the collapse range.

6- The structures that are designed based on the Iranian Code 2800, third edition, for the structural system proposed in this thesis, have a life safety performance level in the evaluation based on the criteria of the Seismic Improvement Guidelines for Existing Buildings.

## **8. Suggestions for future research**

- 1- Doing similar work for a structure with a dual system of flexural frames with off-axis bracing
- 2- Doing similar work using 7 accelerograms
- 3- Doing similar work for buildings that are irregular in plan and elevation

## **9. References**

- 1- Ghanbari, Hamid and Rahgozar, Reza, 2009, Evaluation of seismic behavior of existing steel buildings with double structural systems designed according to Standard 2800 using nonlinear static analysis and performance level, an approach to the code of building design against earthquakes (Standard 2800); Present and Future, Tehran,,<https://civilica.com/doc/87469>
- 2- Kohnepooshi, Omid and Salem, Afrasiab, 2019, Investigation of the Behavior of Slotted Damper Structures in Steel Beam-Column Connection,,,,<https://civilica.com/doc/1178347>
- 3- Mahjoob Asl, Farzad and Yasharbi Nia, Yashar, 2015, Evaluation of the Behavior of Steel Flexural Structures Strengthened with Anti-Buckling Braces, Second International Conference on Civil Engineering, Architecture and Urban Economic Development, Shiraz,,<https://civilica.com/doc/457197>
- 4- Chabaki, Mahsa and Aghakouchak, Ali Akbar, 2018, Comparison of the Behavior of Steel Tube Frame and Flexural Frame Structures Against Fire Effects and Assessment of Progressive Failure Potential, Ninth National Conference on Structures and Steel, Tehran,,<https://civilica.com/doc/838145>
- 5- Yousefian Jezzi, Meysam and Jaberzadeh, Erfan, 2019, Investigation of the effect of beam depth ratio on the behavior of concrete-filled steel beam-column (CFST) connections including side beams with different depths, 3rd International Conference on Civil Engineering, Architecture and Urban Planning, Tehran,,<https://civilica.com/doc/1039373>
- 6- Nastarni, Shadi and Mohebbi Aminabadi, Elahe, 2019, The role and performance of crisis management in reducing natural hazards (earthquake) in earthquake-prone areas, 3rd International Congress on Contemporary Civil Engineering, Architecture and Urban Planning, Tehran,,<https://civilica.com/doc/1002395>
- 7- Bayramnejad, Yousef and Hamidi, Peyman and Farokh Qateh, Hamid and Davari Ahranjani, Ghafour, 2019, Evaluation of ductility of reinforced concrete structures with shear walls with openings using nonlinear static analysis method, 6th National Congress on Civil Engineering, Architecture and Urban Development, Tehran,,<https://civilica.com/doc/1003124>
- 8- Naderi Rad, Iman and Fallahi, Hamid, 2019, Investigation of the behavior of steel flexural frame under the effect of nonlinear static load, Second International Conference on Civil Engineering, Architecture and Urban Development Management in Iran, Tehran,,<https://civilica.com/doc/973391>



9- Naderi Rad, Iman and Fallahi, Hamid, 2019, Investigation of the behavior of steel flexural frame under the effect of nonlinear quasi-static load, Second International Conference on Civil Engineering, Architecture and Urban Development Management in Iran, Tehran,,<https://civilica.com/doc/973392>

10- Rafiei, Hadi and Seyidi Birjand, Seyed Reza, 2018, Comparison of the results of nonlinear static and nonlinear dynamic analyses in steel flexural frames with 3D sandwich panel interframes with openings, Third International Conference on Civil Engineering, Architecture and Urban Design, Tabriz,,<https://civilica.com/doc/806420> Dear student, to complete the form, be sure to download and read Guide No. 1 from the university press.

#### Foreign

11- Comeau,G; Velchev,K; Rogers, C.A, Development of seismic force modification factors for cold-formed steel strap braced wall. Canadian Journal of Civil Engineering, Vol37, pp.236-249, 2010.

12- Gad.F.Duffield.L.Hutchinson.S.MansellaG.Stark(1999) Lateral performance of cold-formed steel-framed domestic structures [https://doi.org/10.1016/S0141-0296\(97\)90129-2](https://doi.org/10.1016/S0141-0296(97)90129-2)

13- F A Boudreault, C Blais, and C A Rogers(2007). Seismic force modification factors for light-gauge steel-frame - wood structural panel shear walls. Canadian Journal of Civil Engineering. 34(1): 56-65. <https://doi.org/10.1139/106-097>

14- Y.S.TianJ.WangT.J.Lu(2004) Racking strength and stiffness of cold-formed steel wall frames <https://doi.org/10.1016/j.jcsr.2003.10.002>

15- M. Al-Kharat C. A. Rogers (2007), Inelastic performance of cold-formed steel strap braced walls. <https://doi.org/10.1016/j.jcsr.2006.06.040>

16- Kaustabh V. Raut, Samadhan G. Morkhade, Rushikesh R. Khartode & Dhiraj D. hiwale(2020) Experimental study on flexural behavior of light steel hollow flange beam with various stiffening arrangements<https://doi.org/10.1007/s41062-020-00345-4>