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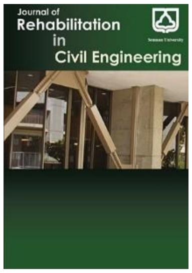

Evaluation on Seismic Performance of Dual Steel Moment-Resisting Frame with Zipper Bracing System Compared to Chevron Bracing System Against Near - Fault Earthquakes

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ABSTRACT

In order to design seismic-resistant buildings, it is necessary to get comprehensive information about their behavior against the forces induced by earthquakes. Seismic design codes have been developed to meet the requirements of a safe and economical structure. According to the structural codes, the designed structures should not be damaged against light or moderate earthquakes so that the members should be had sufficient strength and safety while they should be a ductile complex with a proper structural configuration against severe earthquakes to dissipate the forces caused by ground motions. In the design of steel buildings, the use of moment-resisting frames in combination with braces is a seismic-resistant system. One of these systems is the dual steel moment-resisting frames with zipper braces. In this research, the seismic performance of the moment-resisting frame with the zipper brace system has been studied and its performance has been compared to the performance when the chevron bracing system is used. Three 4-story, 8-story, and 12-story buildings have been selected then they have been modeled by SAP2000 software, and finally, their seismic performances have been evaluated using time history analysis. The structural responses have been compared as comparing the relative displacement of the stories (story drift), the maximum displacement of the roof, and the formation of plastic hinges in the members. The results of the current study have been shown that using a zipper member has been decreased both overall displacement of the structure by about 10 to 30 percent, and also has been reduced the damage index of 4, 8, and 12-story structures by 27, 11, and 12 percent, respectively. The formation of plastic hinges has been directed from horizontal and vertical members toward diagonal members.

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1. Introduction

The first step to develop zipper restraint systems was taken by Khatib et al in 1988. They studied the behavior of the zipper-braced frames using time history analysis response and compared the responses of the zipper-braced, V-braced, Inverted-V braced and X-braced frames and then they concluded that the response of zipper-braced frames has less sensitivity to seismic excitations and it shows more uniform distribution for damage throughout building height. They also showed that zipper-braced frames had a trilinear force and lateral displacement relationship and it can be designed with flexible beams [1]. The research conducted by Kim et al (2008) on the design methods of zipper columns suggested two simple static and dynamic design methods to include the effects of braces' slenderness and higher modes. The results of this study indicate that both proposed static and dynamic methods lead to outperform seismic behavior of the frame in comparison to the lack of zipper columns. This behavior is especially evident in the upper and lower stories of the frame, so that the drifts of 1th and 14th stories was reduced in the studied 15-story frame. Of course, the dynamic method is slightly more effective than the static method in improving the seismic performance of the frame [2]. In 2012, Zhi Chen used belt truss systems at the height of zipper-braced frames to reduce the effects of excessive deformations in tall buildings. For this purpose, two ordinary 12-story and 16-story zipper structures and three zipper frames with belt truss systems were considered in a high-intensity seismic region and their nonlinear responses were compared with each other. The results of the study showed that the drifts of upper stories have been decreased in the zipper-braced frames equipped with belt truss system respect to the drifts in the case without a belt truss system [3]. In 2013, Zahraei et al. evaluated the hysteresis behavior of eccentrically braced frame having a zipper member [4]. In 2013, the seismic behavior of frames braced by the zipper and chevron bracing system has been compared by Razavi and Shaydayi. For this purpose, nonlinear static pushover and nonlinear time history dynamic analyses were performed on various structural models with zipper and chevron bracing systems in different number of stories. Based on the results of nonlinear static pushover analysis, the ductility and behavior factor of zipper-braced frames were better than those braced with a chevron bracing system. The distribution of the story drift ratio and maximum story displacement obtained from nonlinear dynamic analysis showed that the zipper brace has a better performance in uniform distribution of deformation throughout building height and significantly reduces the story drift ratio [5]. Evaluation of the using zipper member in eccentrically braced frame was done by Amiri et al.in 2014. They concluded that as the length of the link beam increases, the load-bearing capacity of the frames decreases while the damage concentration increases in the connections. Also, by using a zipper member in a certain length of the link beam, the stiffness and load-bearing capacity of the frame are increased due to the change of the location of the inelastic hinge [6]. In 2015, Farahani and Mirza Goltabar investigated the rehabilitation details of various frames equipped with zipper braces in regular and irregular frames and then they determined behavior factor for this system [7]. In 2016, Ozcelik et al. did a two-dimensional pushover analysis on 3-story and 9-story models in order to compare the seismic performance of Inverted-V-braced and suspended zipper-braced frames. The results of this study showed that the behavior of the

9-story suspended zipper-braced frame was better than that of the Inverted-V-braced frame. The suspended zipper-braced frame can be a suitable alternative in comparison to the Inverted-V-braced frame in low-rise buildings while the seismic performance of the Inverted-V-braced frame is better than the suspended zipper-braced frame for mid-rise buildings [8]. Sajedi and Mehrabi (2016) indicated that the performance of the zipper brace system was better than the chevron brace system and by increasing the number of frame stories, it revealed the performance improvement of the behavior of the frame [9]. Dashtbani et al. (2016) used the nonlinear static and dynamic analyses for increasing the IDA of each structure using OpenSees Software, the factors of ductility, over strength, and behavior were measured and investigated using the Uang method. According to the results, multistory X braces had less ductility, over strength with very small difference, and less behavior factor in comparison to chevron braces [10]. In the study, ABAQUS Software was used for element analysis by Qodrati Amiri et al. (2017). The advantage of damage concentration in systems with chevron braces prevents the structural members from entering the nonlinear stage, so that they remain plastic. The results of studying the modeling indicate the acceptable ductility of the proposed element and high ability of energy dissipation [11]. Evaluating the analytical results of the study conducted by Kiyainejad and Meshkateddini (2017) indicated that seismic response parameters such as drift, the axial force of columns, the absolute acceleration of stories, and base shear in the model with zipper braces in comparison to the model without zipper elements under selected accelerograms have lower and more invariable values [12]. Vaseghi et al. (2017) developed a simple method for evaluating the performance level of zipper-braced frames (ZBF) using damage indices according to the results of nonlinear static and dynamic analyses [13]. Shah and Dalal (2018) studied the seismic performance of zipper-braced frames of 5, 10, and 15 stories was evaluated using the methods of performance-based practical design (PBPD) and force-based design (FBD), the results indicate the better seismic performance of PBPD in terms of deformation and capacity of frames [14]. In the study were done by Taiyari et al. (2019), a probability-based design of friction dampers in multistory chevron braced steel frames has been proposed. The slip force of the device and the stiffness ratio of the system have been analyzed as two important components. Dynamic analysis to indicate the efficiency of the proposed method includes three structural models of steel moment-resisting frames with friction damper systems (including chevron braces and damper devices). The optimal range of slip force for the three-building frames is between 40 to 55 percent of the total. The weight of structures and the recommended value for the stiffness ratio is 2 [15].The performance level was designed by Nezami Savojbolaghi (2020) using nonlinear static analysis in concrete buildings with moderate ductility and according to the latest regulations. According to the results, the zipper brace is used for reinforced concrete structures without reinforcing the beams and columns [16].Irani Sarand and Jalali (2020) found that the proposed RZBF (rocking zipper-braced frame) has better performance among the others and zipper columns can improve the behavior of rocking systems [17].

In addition to the mentioned methods for the structural numerical analysis, recently some newly developed artificial intelligence-based approaches were successfully implemented by researchers in the civil and construction materials engineering field which can be further employed in future studies [18-23].

In the present study, after selecting the near-fault seismic records, modeling the structures in SAP2000 software and adding zipper member to the chevron bracing system, the relative lateral displacement (drift), damage index and formation of plastic hinge have been compared in performance and seismic levels. The linear and nonlinear dynamic methods were used corresponding to the seismic characteristics of structures. The behavior of zipper and chevron bracing system were investigated in term of inelastic behavior to improve the seismic response of steel frames. Improvement of performance levels in structures equipped with zipper members and its effects on displacement of stories and roofs as well as the number and locations of plastic hinges in near-fault earthquakes have been considered.

2. Earthquake and seismic design of steel structures

Earthquake is one of the natural phenomena inducing dynamic forces to buildings and if buildings are not able to withstand these forces, they will collapse causing financial losses and mortalities. Therefore, seismic rehabilitating and retrofitting of buildings is the best solution to survive these events. In structural design, reinforcement of seismic performance is necessary. Selected structural systems play an important role in seismic design to prevent energy loss and resist earthquakes. The quality of response and survival of building under a seismic event depends on the type of lateral resisting system and its behavior against the vibrations. The lateral resisting system must be able to absorb and dissipate the energy induced by the earthquake without causing permanent damage to the building. In seismic design of a structure, the designer has to design the structure in such a way that its resistance is less than the required resistance when an earthquake occurs, while it must have the capability to undergo deformations and relative displacements through entering to inelastic area in order to dissipate the energy caused by the earthquake. The use of braced frames provides an effective system to resist lateral loads in steel buildings. Steel braces can be used whether as a single resisting system for lateral loads or along with moment-resisting frames. The configuration of restraint systems is generally described as centric or eccentric and they are utilized to whether provide stiffness or control lateral displacement.

One of the lateral resisting systems in steel structures is the chevron centric bracing system. During an earthquake, for each of the bracing zones, one member acts in the tension, and the other member acts in the pressure in order to resist against lateral force. When the lateral load increases, the members buckle and plastic hinges are created within them. Due to the inability to distribute plastic hinges in a wider length, buckling is observed in the braces of lower stories so that only these members are involved in decapitating energy caused by the earthquake while braces of upper stories remain in the elastic area. By buckling the members of the lower stories, the shear capacity of the whole frame is reduced, and therefore, these structures will not have desired behavior factors. In the lower stories, an unbalanced vertical force is exerted to the intersection of the braces in the upper beam due to the buckling of the compressive member which causes excessive displacement in the middle of the beam. As a result, large beam sections without any appropriateness with other members have been obtained. In order to compensate for these shortcomings, a new model called the zipper-braced system is proposed. Since the stiffness and ductility of the structure are usually two contradict concepts, there should be a reasonable balance between these two factors in the structural system considering economic principles. In this regard, the chevron bracing system has been introduced together with a vertical member (zipper), which has the sufficient ability to absorb energy. The main issue in the current research is to evaluate the seismic performance of dual steel moment-resisting frame - zipper brace system in comparison to the chevron bracing system.

3. Chevron-braced frames

The inverted-V-braced frame (chevron bracing system) is one of the centric bracing systems in which a vertical truss is counteracted to resist against lateral forces such as earthquakes. Chevron bracing systems have high stiffness and strength, but they show poor post-buckling behavior and they are unable to redistribute the large vertical unbalanced forces created due to buckling of the braces [1]. To solve this problem and control the story drifts as well as to have stable behavior, zipper braces have been suggested.

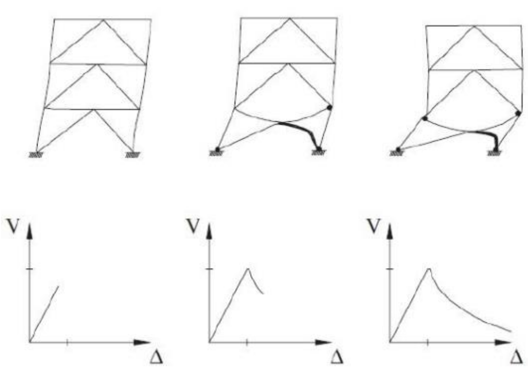

Fig. 1. Chevron bracing frame system and its failure mechanism.

4. Zipper-braced frames

In this system, a vertical element called the zipper column is added to the brace-beam intersection to withstand the upward unbalanced vertical forces created as a result of buckling of the braces. This new structural configuration decreases the damages in the structure and improves the distribution of damage throughout building height. In recent decades, North American research centers have extensively analyzed the following systems analytically and experimentally: CBF systems with a weak zipper column (inelastic behavior), CBF systems with a strong zipper column (elastic behavior) [24].

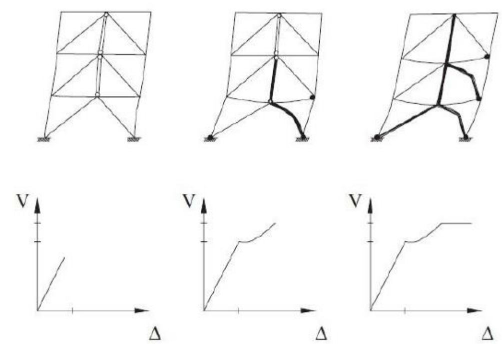

Fig. 2. Zipper bracing frame behavior and its capacity curve under lateral load.

5. Analysis and design

The governing nature of No. 2800 Standard of Iranian Code of Practice for Seismic Resistant Design of Buildings is based on dynamic analysis [25]. Dynamic analysis methods are consistent with the seismic characteristics of the buildings. The dynamic analysis method in the time domain called time history dynamic analysis has valid results for both elastic and inelastic areas. On the other hand, the nonlinear behavior of the structure is inevitable against strong earthquakes. It is more appropriate to use nonlinear dynamic analysis to evaluate the nonlinear behavior of the model under different seismic records. In this regard, the characteristics of the structural members must be determined both in the linear and nonlinear stages under heavy loads. In the nonlinear dynamic analysis method, the deformation, internal forces, and the response of structure are calculated considering the nonlinear behavior of the materials and the non-geometric behavior of the structure under certain accelerograms. In the present study, this analysis method has been used by selecting 7 different seismic records [26].

Three structures with the number of floors 4, 8 and 12, respectively as a symbol of short buildings (according to the criteria of Article sixth of the National Building Regulations of Iran, paragraph 6-10-6-7) [27], Medium (height between two short and high structures considered as medium structure), high-rise (according to the book Basics of Design of High-rise Buildings 4) [28] designed.

The structures have been analyzed and designed using SAP2000 v20 Software. Sections are analyzed and designed in three-dimensional models and their seismic performance is measured in a two-dimensional way. The difference between the three models is in the number of stories.

The buildings are considered as building of moderate importance in the high-intensity seismic region. The soil category is type II soil according to No. 2800 Standard and the structural system is a moment-resistant steel frame with chevron and zipper bracing system. According to the Iranian National Building Code (INBC), Part 6: Loads, the dead load of the stories, live load of the stories, and the live load of the roof are considered 600 kg/m^2, 200 kg/m^2, and 150 kg/m^2, respectively [27]. The height of all stories is 3 m. The stories are typical and their weights are the same. Also, a rigid diaphragm is provided for the floors and ceilings of all stories. The plan of the building is regular, symmetrical, and square in shape, so that in each direction, The structures have 6 spans including two 4-meter spans and four 5-meter spans. The former is located in the middle of the structure and the latter is located in pairs on both sides of the two 4-meter spans. The braces are located in two 5-meter spans. The models developed in the software have been included in the following figures 3,4,5,6,7,8.

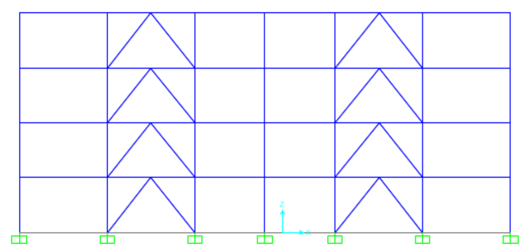

Fig. 3. Model of the 4-story structure with chevron bracing.

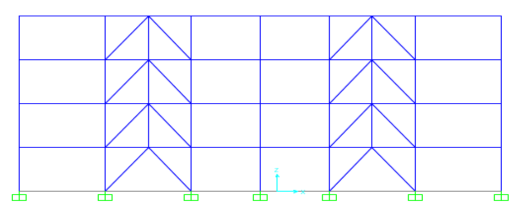

Fig. 4. Model of the 4-story structure with zipper bracing.

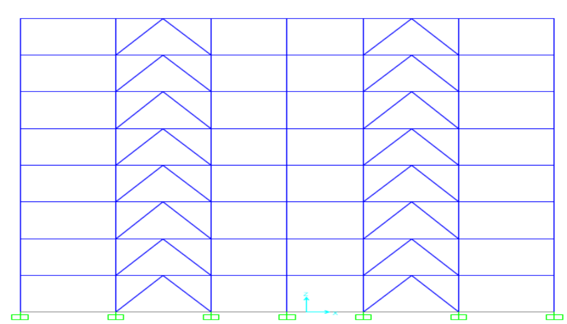

Fig. 5. Model of the 8-story structure with chevron bracing.

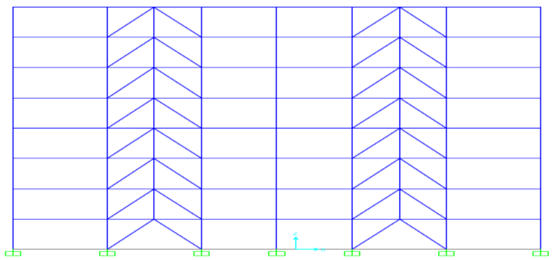

Fig. 6. Model of the 8-story structure with zipper bracing.

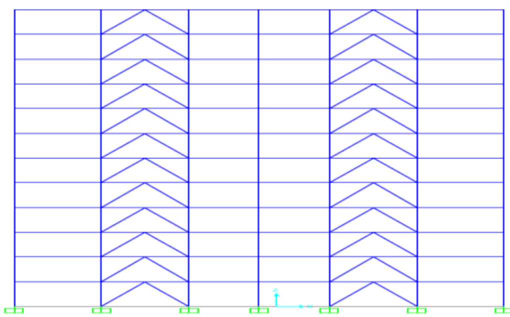

Fig. 7. Model of the 12-story structure with chevron bracing.

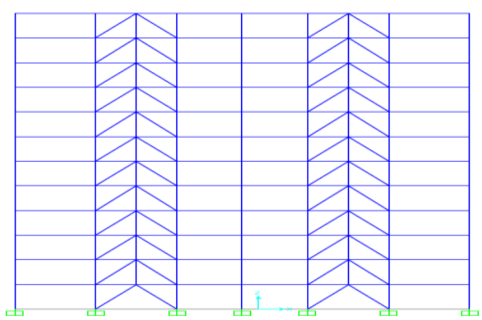

Fig. 8. Model of the 12-story structure with zipper bracing.

Table 1. Properties of used materials (ST37steel).

Yield Strength of Steel (FY)	2400 kg/cm^2 (For force-controlled members)
Ultimate tensile strength of Steel (f_u)	3700 kg/cm^2 (For force-controlled members)
Elastic Modulus of Steel (E_s)	2.1e6 kg/m^2
Elastic Modulus of Concrete (E_c)	2.1e5 kg/m^2
Poisson's ratios of Steel	0.3
Poisson's ratios of Concrete	0.2
Density of Steel	7850 kg/m^3

5.1. Modeling requirements

The building model for analysis should include all the structural components that affect the mass, strength, stiffness, and ductility of the building in the vicinity of the performance point. The members and components that are expected to be able to provide lateral resistance after several cycles of seismic loads must be included in the model. According to FEMA356 and the Iranian Seismic Rehabilitation of Existing Buildings, if the sum of the lateral stiffness of the non-structural components is more than 10% of the lateral stiffness in each story, their effects should be considered in the modeling process [29]. In linear analysis, only the primary members are modeled and secondary members are controlled only for the deformations caused by the analysis while in the nonlinear analysis, all the primary and secondary members are modeled and the effects of reduction in strength and stiffness of the components is included in the models. In this research, the effect of non-structural stiffness and infilled frames has not been considered. The properties of the materials used in the modeling have been provided in Table 1.

In nonlinear analyses for deformation-controlled members, the expected strength of the materials should be used instead of the lower bound of material strength used in linear analyses. According to the Iranian Seismic Rehabilitation of Existing Buildings (Journal 360), this amount for steel is as follows [30]:

$$F_{ye} = 1.1F_y = 1.1 \times 2400 = 2640\, kg/cm^2 \quad (1)$$

$$F_{ue} = 1.1F_u = 1.1 \times 4000 = 4400 \frac{kg}{cm^2} \quad (2)$$

In this modeling, BOX sections,IPE sections, and a pair of channel sections have been used for columns, beams, and zipper columns and braces, respectively. Due to the similarity of the sections used in 4-story structures with 8-story and 12-story structures, the sections in the 4-story have been shown structures shown in Figures 9 and 10.

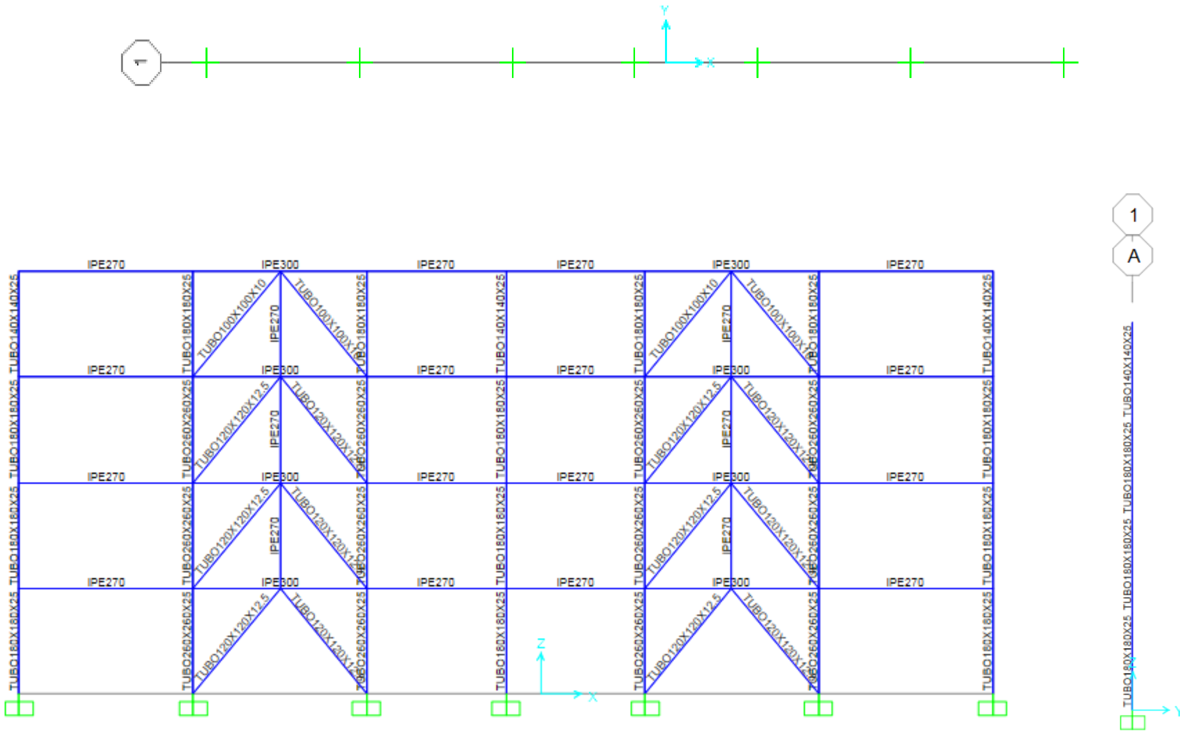

Fig. 9. View the specifications of selected sections for a 4-story structure with zipper bracing.

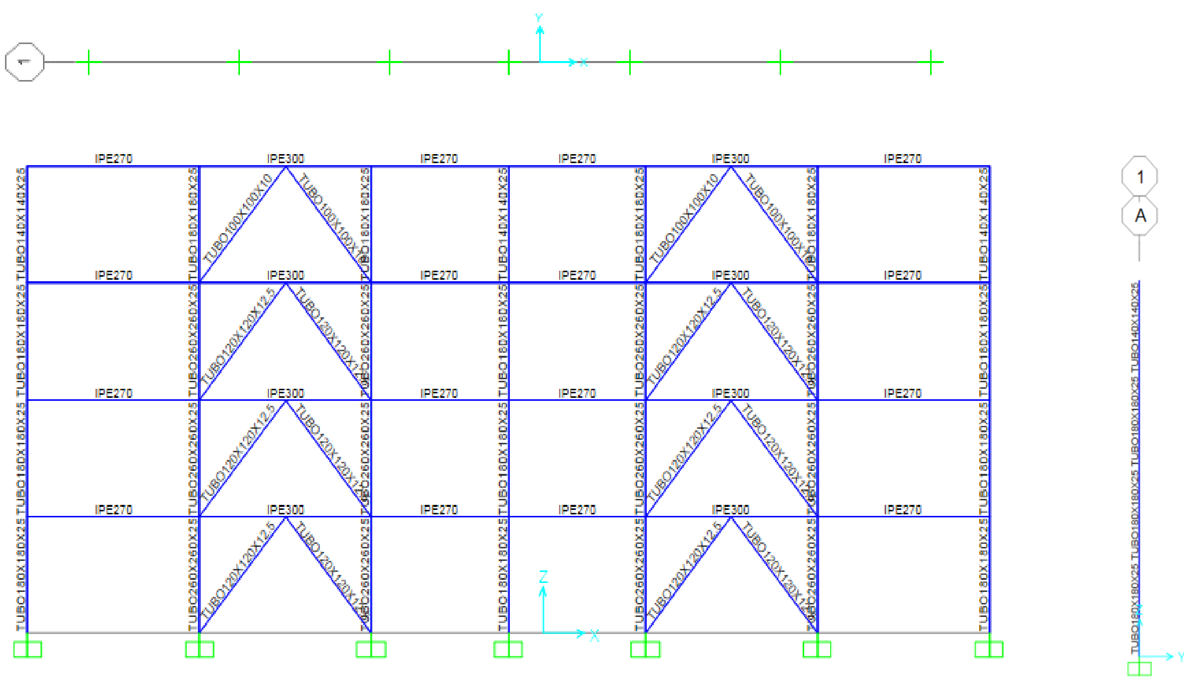

Fig. 10. View the specifications of selected sections for a 4-story structure with chevron bracing.

5.2. Gravity and lateral loads combination

In nonlinear methods, the forces and maximum displacements of each member are calculated regarding nonlinear behavior of the structure and the effects of both gravity loads and lateral loads must be considered. Gravity loads must include all dead loads and applied live loads (as a percentage of all live loads). FEMA356 considers two upper and lower bound method for combining dead and live loads [30].

$$Q_G = 1.1[Q_D + Q_L] \qquad (3) \quad \text{Upper bound}$$

$$Q_G = 0.9Q_D \qquad (4) \quad \text{Lower bound}$$

Where Q_D and Q_L refer to dead loads and live loads, respectively. Since the plans are symmetrical and hence the centers of mass and rigidity are the same for the structures,

accidental torsion is neglected and loads combinations are considered as follows:

$$\begin{aligned} &1.\ 1.4D \\ &2.\ 1.2D + 1.6L \\ &3.\ 1.2D + L + E \\ &4.\ 0.9D + E \end{aligned} \qquad (5)$$

5.3. Control of relative story displacement

According to No. 2800 Standard of Iranian Code of Practice for Seismic Resistant Design of Buildings, the relative story displacement (Drift) is obtained according to the period of the structure and based on the following relation as follows:

$$\Delta_M = C_d . \Delta_{eu} \qquad (6)$$

Where Δ_M = Relative lateral displacement

C_d= Displacement amplification factor

Δ_{eu}= Relative lateral displacement of the story under the effect of the design earthquake

5.4. Selection of earthquakes for time history analysis method

The criteria for selecting accelerograms in order to use in time history analysis have been provided in Section 2-5-3 of No. 2800 Standard. The characteristics of the selected accelerograms have been given in Table 2. In this table, PGD, PGV, and PGA are the maximum displacement, maximum velocity, and maximum acceleration of the earth during an earthquake, respectively. Accelerograms used to determine the motion of the earth must indicate the actual motion of the earth in the construction site as far as possible. Accelerograms applied to time history analysis must be selected in such a way that the magnitude, distance from faults, and source mechanism of their earthquake are proportional to the earthquake occurring on the construction site of the structure. Due to producing diversity and less dependency on results on the selected earthquake, earthquakes with different seismic characteristics (duration of severe shaking) have been selected. Due to the fact that the soil type in this study is type II, according to Regulation 2800, Table 2-3 (land type classification) which has determined the shear wave velocity for different types of soils. In this table of code 2800, for soil type

II, the shear wave velocity in the range of 375-750 has been determined that all selected accelerometers have their shear wave velocity in the same range. [25].

Table 2. Characteristics of selected accelerations.

Earthquake	R.S.N	Station	PGA(g)	PGV($\frac{m}{s}$)	PGD (m)	Vs30 (m/s)	Magnitude	Distance from fault (Km)
Coalinga (1983)	368	Pleasant Valley P.P. - yard	0.608	0.61	0.2215	257.38	6.36	8.41
Chalfant (1986)	549	Bishop - LADWP South St	0.547	0.432	0.1631	303.47	6.19	14.38
Kobe (1995)	1106	KJMA	0.484	0.531	0.146	312	6.9	0.96
Landers (1992)	836	Baker Fire Station	0.529	0.549	0.402	324.62	7.28	8.79
Mammoth Lakes (1980)	230	Convict Creek	0.784	0.444	0.0772	372.12	6.63	6.63
Manjil (1990)	1634	Abhar	0.283	0.443	0.185	302.64	7.37	7.55
Tabas (1978)	139	Dayhook	0.504	0.581	0.236	354.37	7.35	13.94

5.5. Distribution of seismic lateral force throughout building height

The shear force calculated according to the proposed relations is distributed according to the following relation throughout building height as follows.

$$F_i = \frac{W_i h_i^k}{\sum_{j=1}^{n} W_j h_j^k} v \qquad (7)$$

Where F_i is the lateral force applied to the ith floor, the h_i is the height of the ith floor calculated from the base level and the value of k is equal to:

$$k = 2.5T + 0.75 \qquad (8)$$

For the periods less than 0.5 seconds, the value of k is considered 1 while it is chosen 2 for periods greater than 2.5 seconds.

5.6. Performance-based design

Performance-based design means that a building is designed according to the expected performance from the beginning. First, we thoroughly design the structure according to the uploaded by-laws and selected design, then we control the building according to the desired performance level.

5.7. Modelling through plastic hinges

This type of modeling refers to the allocation of plastic hinges over the elastic elements; in other words, in sections of the element where it is probable that the element reaches its plastic resistance, a plastic hinge is allocated. Generally, plastic hinge refers to two points in the structure:

1. A plastic hinge in deformation-controlled members represents the nonlinear force-deformation relationship.
2. In force-controlled members, it represents members' yield limit.

Forces (forces and their related displacement) in a structure are divided into force-controlled (in force-controlled forces, which the member must remain in the elastic scope, its sufficiency is determined by its resistance) and deformation-controlled (deformation-controlled members have inelastic behavior and their sufficiency is determined through their ductility). The members are categorized according to forces imposed on them (for example column in bending and column in shear); thus each member is categorized as primary and secondary and force-controlled or deformation-controlled. It can be stated that under FEMA-356 or Journal 360, different types of plastic hinges in nonlinear modeling are as follows:

The first type includes deformation-controlled hinges determined by force-deformation relationship.

The second type is defined according to the maximum tensile and compressive forces or interaction or bending moment.

Plastic hinges in the deformation-control mode may have different performance levels such as IO, LS, CP depending on the rotation of plastic in Figure 11.

5.7.1. Bending plastic hinge (M) and bending-axial plastic hinge (P-M):

Generally, these hinges are deformation-controlled. Internal forces in beams and columns of bending frames are bending and bending-axial respectively. The deformation of plastic in these members due to the lateral loads of the earthquake is usually represented as plastic hinges at the beginning and end of columns and beams.

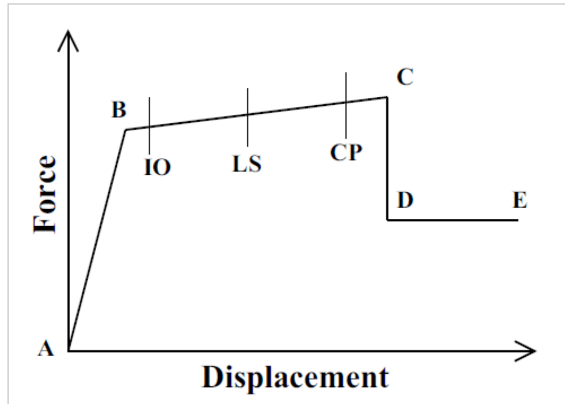

Fig. 11. Plastic hinges in the structure and the formation of plastic hinge [11].

Therefore, at the beginning and end of the beam, plastic hinges are as bending plastic hinge (M). Furthermore, at the beginning and end of the column, abending-axial hinge (P-M) is considered.

5.7.2. Axial plastic hinge (P):

This type of hinge is applied to braces since braces are under axial tensile and pressure forces. It is usually located in the center of braces.

5.8. Assigning plastic hinges in software

In SAP2000, the location of the hinges is expressed as proportional to the length of the member. This indicates the point at which the likelihood of occurring nonlinear behavior is at the highest level. The description of the assigned hinges has been shown in Table 3.

As it was stated earlier, in SAP2000 Software, plastic hinges can be allocated automatically to the members of the structure such as beam, column, and brace. However, if there are special regulations on columns or beams or according to the conducted research, there must be more conformity to the instruction of reforming the existing buildings (Journal 360), they can be amended and changed [30].

Table 3. Description of joints assigned to the structure in modeling.

Member	Location of Hinge	Control Mechanism	Type of Hinge	Description
Column	Beginning and end	Deformation-controlled in tension and force-controlled in pressure	P-M3	Interaction of axial force and bending moment
Beam	Beginning or end	Deformation-controlled	M3	Bending moment around the strong axis
Beam in braced frame	Beginning and end	Deformation-controlled	M3	Bending moment around the strong axis
	Center	Deformation-controlled	V	Shear force
Brace	Center	Deformation-controlled	P	Axial force

In Table 4, which is a part of Table 3.5 in Journal 360, modeling parameters and acceptance criteria are included to define and amend plastic hinges in beams under bending and shear [30]. Modeling parameters include factors a, b, and c, and acceptance criteria include IO (immediate occupancy), LS (life safety), and CP (collapse prevention). According to 4, to define shear hinge in the middle of the span beam with brace, the values of modeling parameters and acceptance criteria are as follows:

a = 5 b = 7 c = 1
IO = 0.25 LS = 6 CP=7

In Table 5, which is a part of Table 4.5 in Journal 360, modeling parameters and acceptance criteria are included to define and amend plastic hinges in columns under tension [30].

According to Table 5, to define tension in the axial-bending hinge in columns, the values of modeling parameters and acceptance criteria are as follows:

a = 9 b = 11 c = 0.6
IO = 1 LS = 9 CP=11

According to Journal 360, θ_y is the rotation of the yield limit of the member and is achieved through the following equation [24]:

$$\theta_y = \frac{zF_{ye}L_b}{6EI_b} \quad (9)$$

z: plastic section modulus (cm^3)
L_b: length of beam
I_b: inertia of moment of beam

The obtained coefficients are entered into the software manually. Due to the similarity of the place of allocation of plastic joints in 4-story structures with 8 and 12-story structures, the only place of allocation of plastic joints have been shown in 4-story structures in Figures 12 and 13.

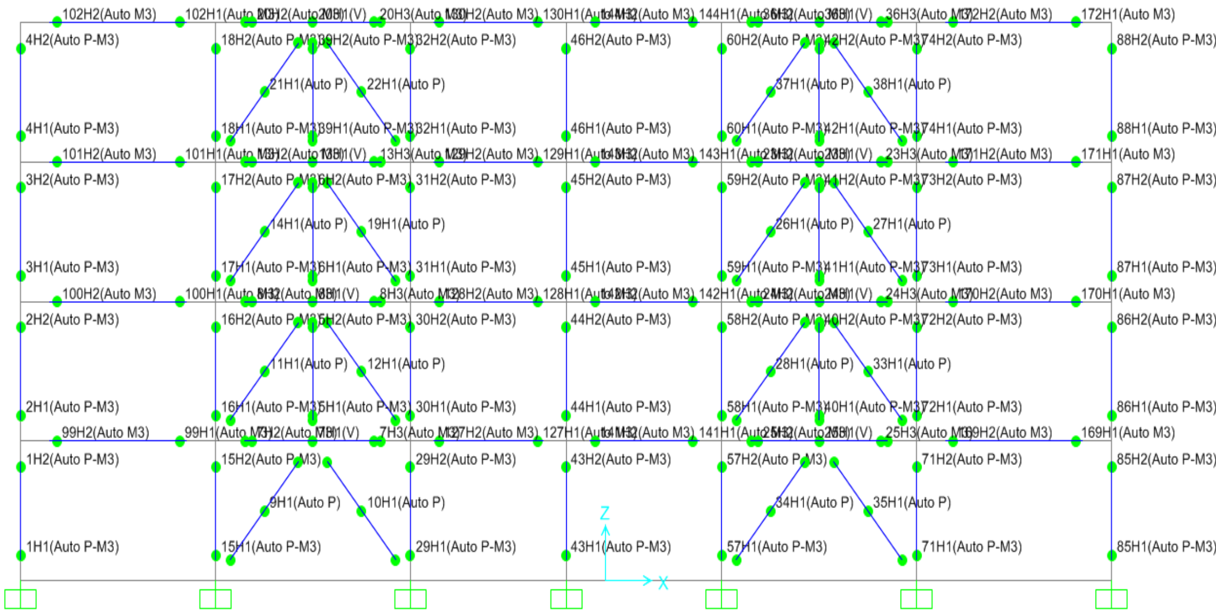

Fig. 12. Location of plastic joints in a 4-story structure with zipper bracing.

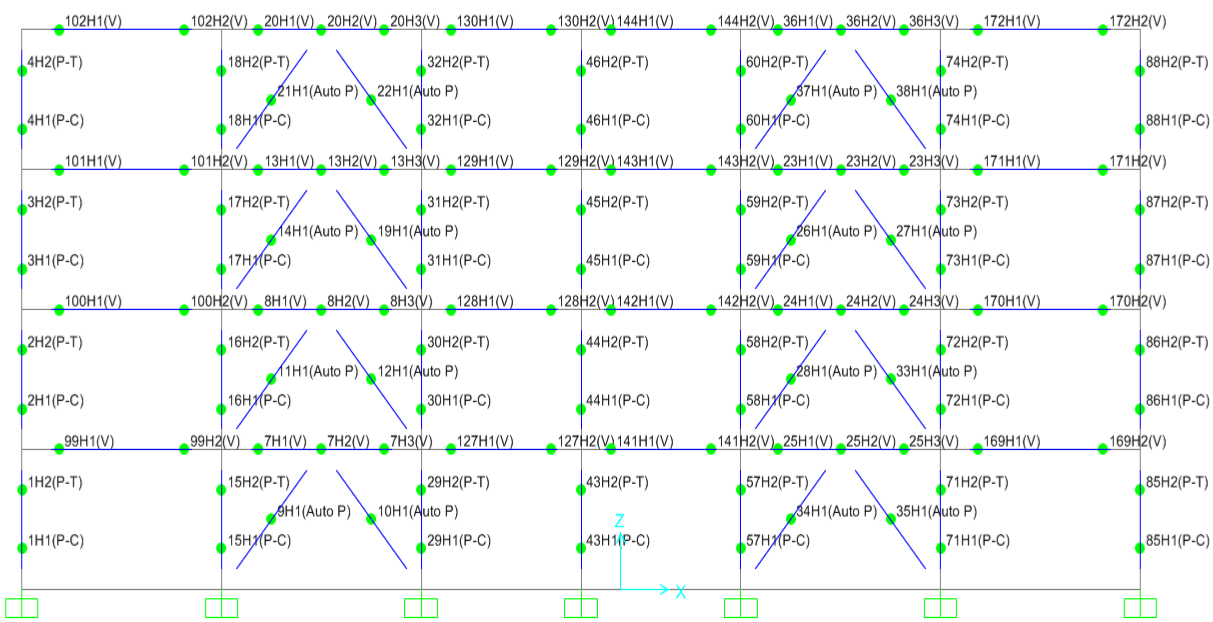

Fig. 13. Location of plastic joints in a 4-story structure with Chevron bracing.

5.9. Modifying and preparing accelerograms for time history analysis

There are different ways to equalize the data to make their related earthquakes to reach similar intensity. The simplest way to equalize accelerograms according to the seismic by-law of uploading buildings is to select three or seven accelerograms that each pair includes two horizontal components related to the earthquake similar to the earthquake of the design first. If three earthquakes are selected, the

maximum of the response of the structure must be considered and if seven pairs are used, the average of the response must be applied to the design [17]. In this research, seven accelerograms have been used. Accelerograms with spectrums according to the spectrum of the design are called accelerograms compatible with the spectrum of the design and applied to time history analysis of the structure. First, all accelerograms are equalized to reach their maximum value, so that their maximum acceleration is equal to g; in each accelerograms, PGA is calculated and equalized to reach g with a particular coefficient.

Table 4. Modeling parameters and acceptance criteria in steel structures to bend and shear beams [31].

Component/ force	Modeling parameters			Acceptance criteria				
	Plastic rotation angle, radian	Ratio of residual stress		Plastic rotation angle, radian				
	A	B	C	All members	Primary members		Secondary members	
				IO	LS	CP	LS	CP
Beams, in bending								
$\frac{h}{t_w} \leq 2.45\sqrt{\frac{E}{F_{ye}}}, \frac{b_f}{2t_f} \leq 0.3\sqrt{\frac{E}{F_{ye}}}$	$9\ \theta_y$	$11\ \theta_y$	0.6	θ_y	$6\ \theta_y$	$8\ \theta_y$	$9\ \theta_y$	$11\ \theta_y$

Table 5. Modeling parameters and acceptance criteria in steel structures for columns in tension [31].

Component/ force	Modeling parameters			Acceptance criteria				
	Plastic rotation angle	Ratio of residual stress		Plastic rotation angle				
	A	B	C	All members	Primary members		Secondary members	
				IO	LS	CP	LS	CP
Beams and columns in tension (except for beams and columns of eccentrically braced frames)	$5\ \Delta_T$	$7\ \Delta_T$	0.1	$0.25\ \Delta_T$	$3\ \Delta_T$	$5\ \Delta_T$	$6\ \Delta_T$	$7\Delta_T$

Then, the desired accelerograms are processed in Seismosignal Software and single-degree-of-freedom (SDOF) is calculated for the pair of accelerograms. The acceleration response spectrum of each pair of equalized accelerograms was determined in Seismosignal with considering the damping of 5%.

$$S_a = \sqrt{S_{a_x}^2 + S_{a_y}^2} \tag{10}$$

$$S_{average} = \frac{S_{a_1} + S_{a_2} + \ldots + S_{a_n}}{n} \tag{11}$$

After calculating the above, we measure them using the design spectrum of No. 2800 Standard and according to No. 2800 Standard, the obtained values must be higher than the diagram of the design spectrum standard. [25].

As the intensity of the selected earthquakes affects the results of the time history analysis, the accelerograms of different earthquakes must be normalized in order to compare the results of this type of analysis so that all of these accelerograms represent the same intensity for an earthquake. By dividing the average value of No. 2800 Standard by the average value of each accelerogram, a ratio less than unit is obtained called scale factor [25]. If the scale factor is multiplied by original accelerograms, the new values are achieved for accelerograms to be used in the structure. Normalized accelerogram should be 1.4 times more than that of the standard design spectrum for periods ranging between 0.2T to 1.5T. The accelerogram used in the present study have been illustrated in Figs. 14 to 20 as follows

Table 6. Scale factors of different earthquakes to normalize them.

Earthquake	Scale factor
Coalinga	0.3075
Chalfant	0.2475
Kobe	0.1845
Landers	0.2318
Mammoth Lakes	0.4845
Manjil	0.2824
Tabas	0.3041

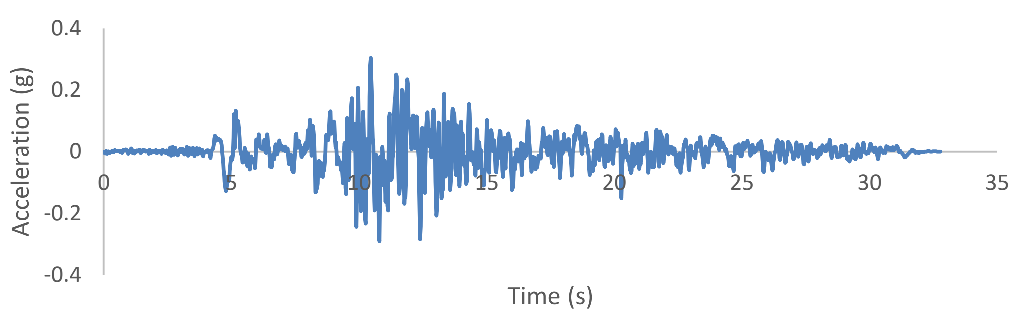

Fig. 14. Tabas accelerogram.

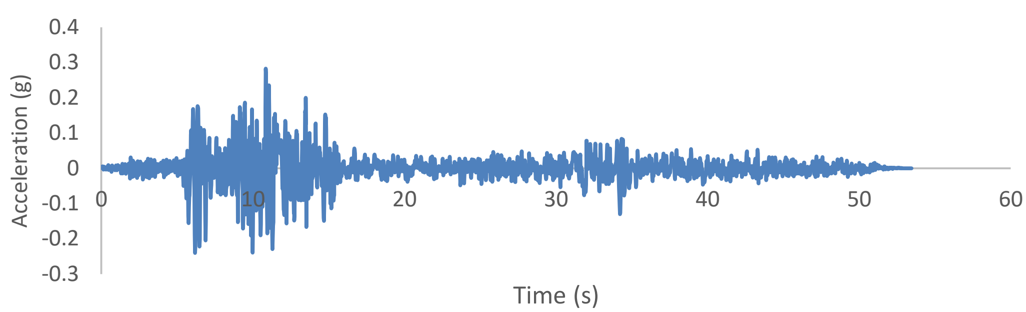

Fig. 15. Manjil accelerogram.

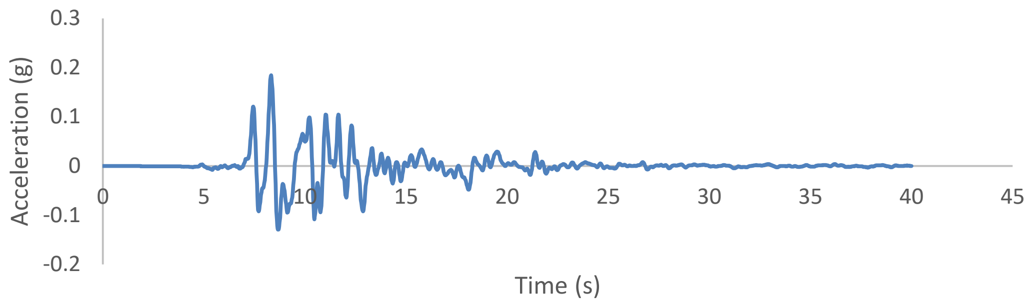

Fig. 16. Kobe accelerogram.

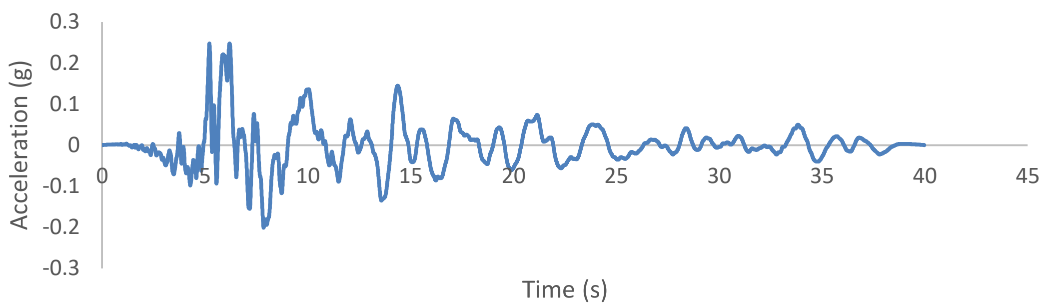

Fig. 17. Chalfant accelerogram.

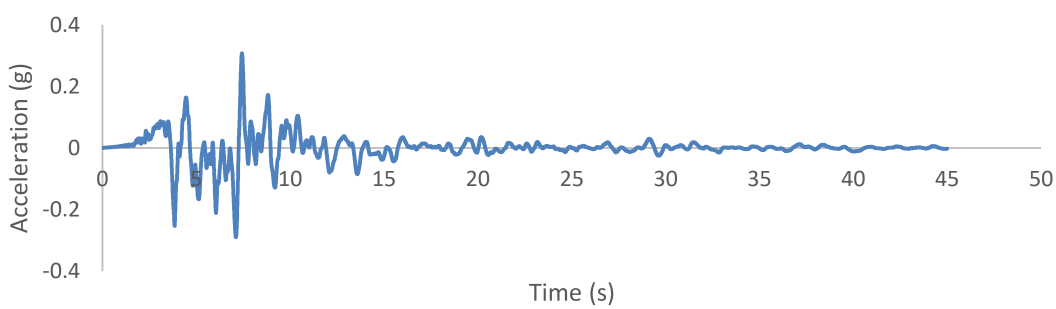

Fig. 18. Coalinga accelerogram.

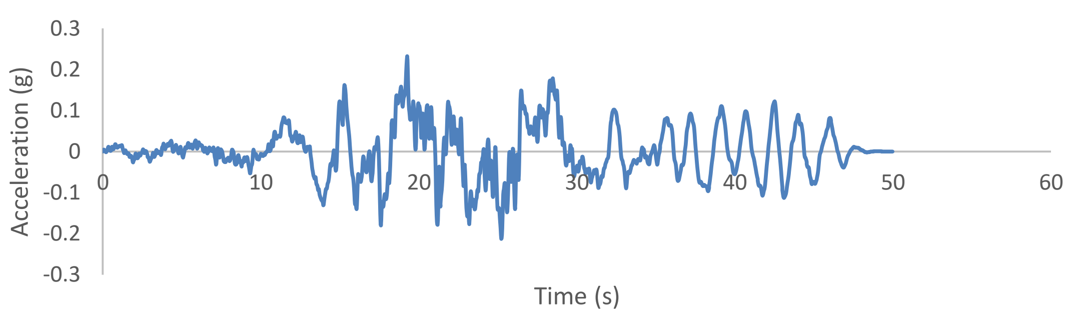

Fig. 19. Landers accelerogram.

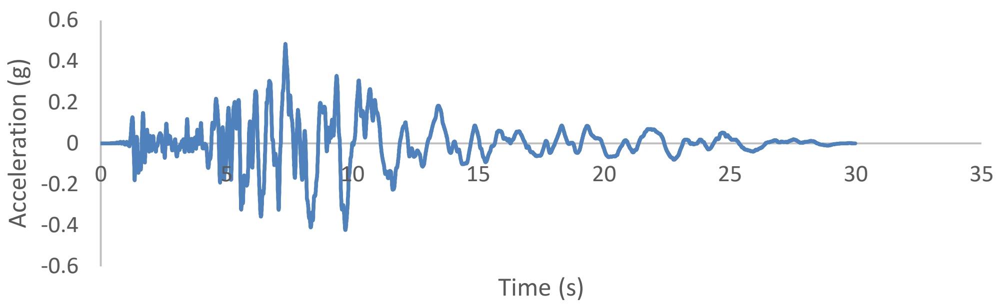

Fig. 20. Mammoth Lakes accelerogram.

6. Calculations and discussing the results

6.1. Comparison of displacement of structure and relative story displacement in the 4-story structure

The results of displacement for 4-story structures braced with chevron and zipper braces have been presented in Figs. 21 and 22, respectively. It should be noted that all displacement charts are extracted and plotted in envelope mode (i.e. maximum value). As can be seen in Fig. 21, the displacement of the structure with the zipper bracing system is less than the corresponding value of structure with the chevron bracing system in all story levels. It is evident from the figures that the displacement of the roof is 31 mm in the structure with a zipper bracing system while this amount is 35 mm in the structure with a chevron bracing system showing an 11% decrease in displacement. Averagely, however, displacement in the structure with the zipper bracing system has been decreased by about 20%. Fig.22 illustrating the average responses of 7 seismic records as the relative story displacement shows that the displacement of the stories relative to each other is better in the structure with the zipper bracing system.

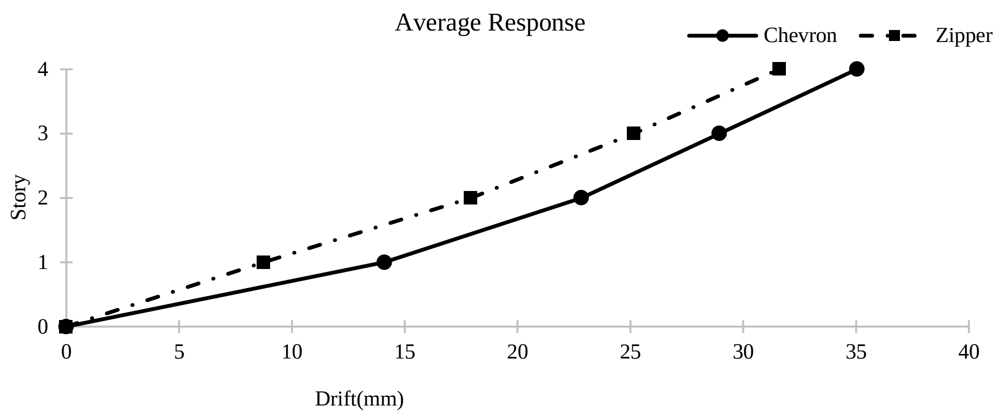

Fig. 21. Comparison of lateral relative story displacement of 4-story structure with chevron and zipper braces (average responses).

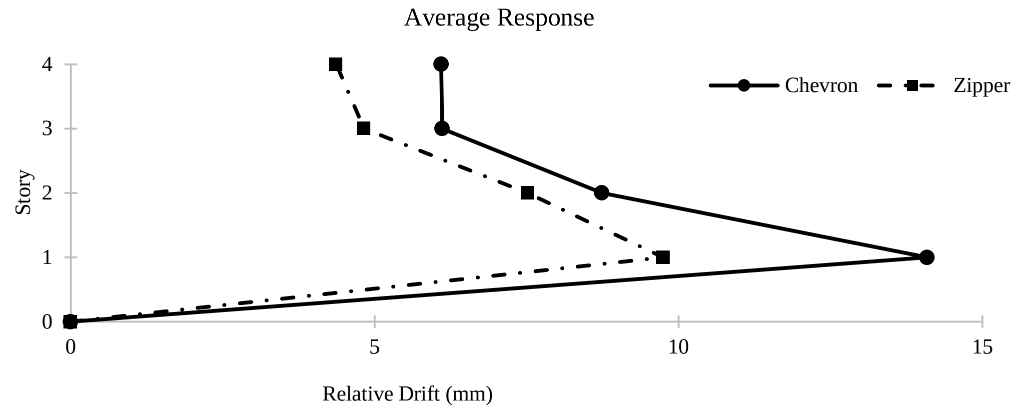

Fig. 22. Comparison of lateral relative story displacement of 4-story structure with chevron and zipper braces (average responses).

6.2. Comparison of displacement of structure and relative story displacement in the 8-story structure

The results of displacement for 8-story structures braced with chevron and zipper braces have been presented in Figs. 23 and 24. It should be noted that all displacement charts are extracted and plotted in envelope mode (i.e. maximum value).

For an 8-story structure, as a 4-story structure, the structure has experienced certain displacement due to the earthquake. As can be seen in Fig.23, the displacement of the 8-story structure with a zipper bracing system is less than the corresponding value of structure with a chevron bracing system in all story levels. It is clear from the figures that the displacement of the roof is 122 mm in the structure with a zipper bracing system while this amount is 162 mm in the structure with a chevron bracing system showing a 24% decrease in displacement. According to Fig. 24, after the second story, the stories did not have significant displacement relative to each other in the structure with the zipper bracing system. This suggests that the zipper member in the braced structure causes the structure to deform more coherently.

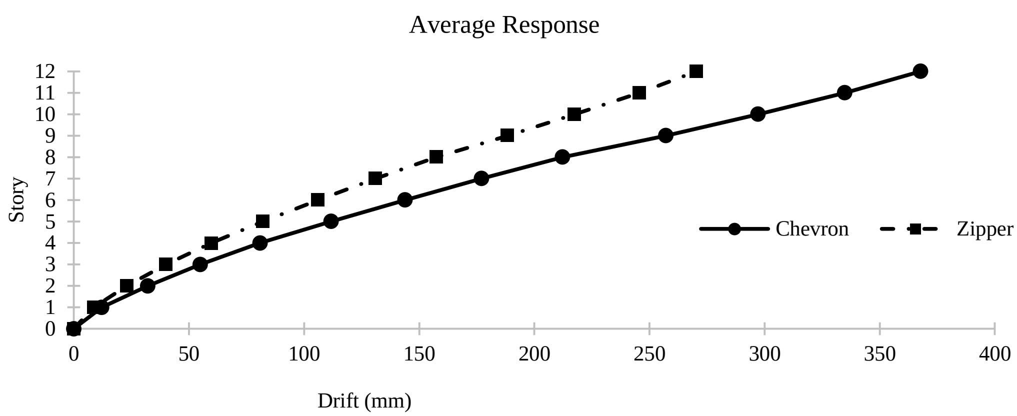

Fig. 23. Comparison of lateral relative story displacement of 8-story structure with chevron and zipper braces (average responses).

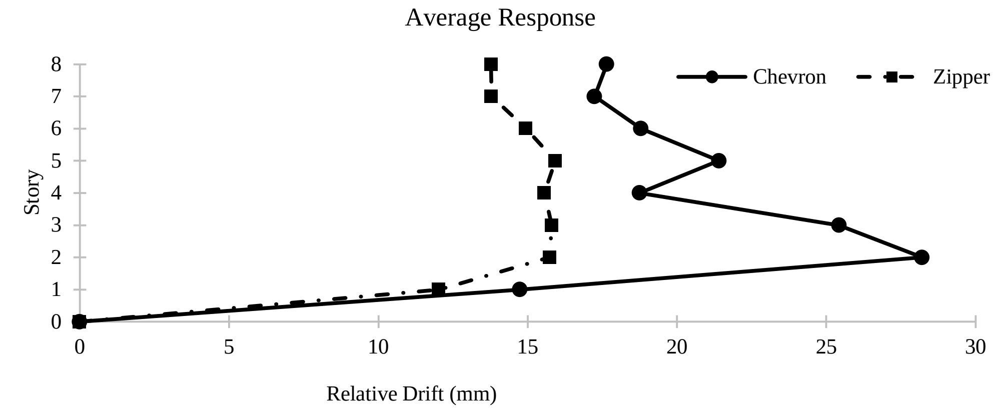

Fig. 24. Comparison of lateral relative story displacement of 8-story structure with chevron and zipper braces (average responses).

6.3. Comparison of displacement of structure and relative story displacement in the 12-story structure

The results of displacement for 12-story structures braced with chevron and zipper braces have been presented in Figs. 25 and 26. It should be noted that all displacement charts are extracted and plotted in envelope mode (i.e. maximum value).

As can be seen in Fig.25, the displacement of the 12-story structure with a zipper bracing system is less than the corresponding value of structure with a chevron bracing system in all story levels. It is clear from the figures that the displacement of the roof is 270 mm in the structure with a zipper bracing system while this amount is 367 mm in the structure with a chevron bracing system showing a 26% decrease in displacement. According to Fig. 26, in the structure with a zipper brace, the relative story displacement is lower and the structure behaves more uniformly. In general, based on the results of 4-, 8- and 12-story structures, it can be concluded that the use of zipper member makes the behavior of the structure more integrated and the relative story displacements are reduced. It also reduces permanent displacements of structures by about 20%.

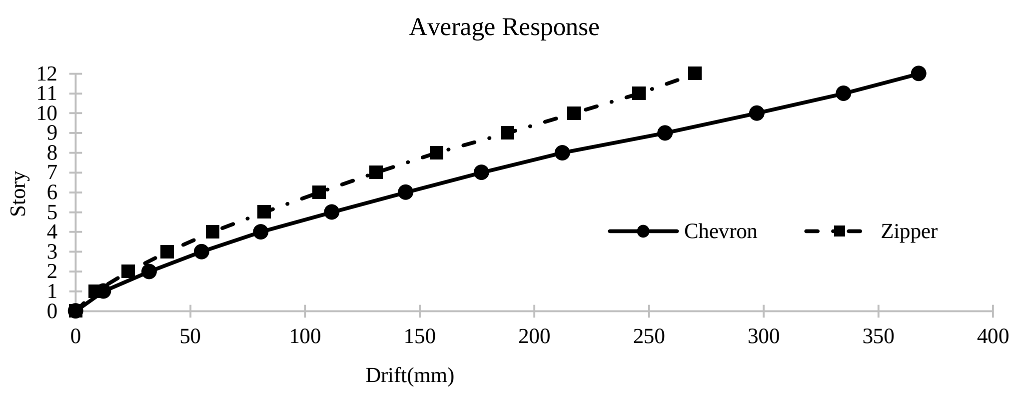

Fig. 25. Comparison of lateral relative story displacement of 12-story structure with chevron and zipper braces (average responses).

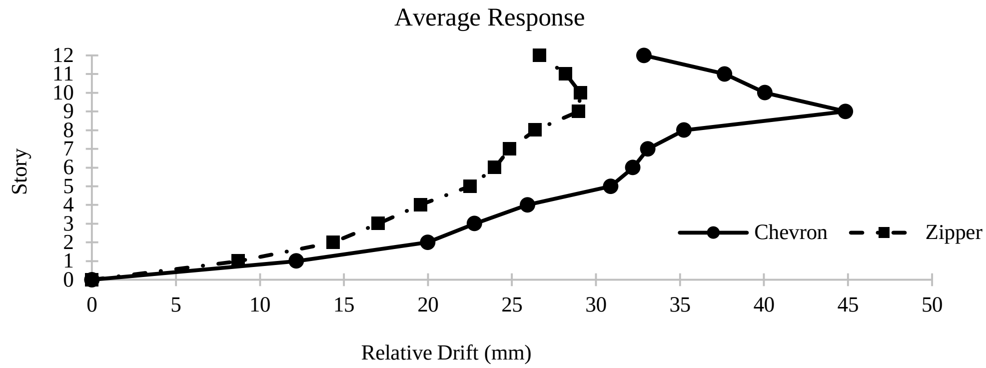

Fig. 26. Comparison of lateral relative story displacement of 12-story structure with chevron and zipper braces (average responses).

6.4. Comparing the maximum displacement of roof

Table 7 shows the maximum displacement of the roof in structures braced with chevron and zipper bracing system. As 7 records have been considered, the final response is obtained as the average value of all responses.

Table 7. Comparing the maximum displacement of the roof.

Number of stories	Structure with chevron bracing system	Structure with zipper bracing system	Difference (%)
4	35	31	11%
8	163	122	24%
12	368	270	26%

6.5. Plastic hinges formed and evaluated based on the performance levels of the structure

In order to evaluate the effect of earthquakes on the strength of dual lateral resisting systems including frames equipped with braces (chevron and zipper bracing system), plastic hinges formed in 4-, 8- and 12-story structures have been evaluated. The results are as follows.

6.5.1. Distribution of plastic hinges in 4-story structure:

As can be seen in Figs. 26and 27, 6 plastic hinges have been formed at the location of braces under the Chalfant earthquake in the 4-story structure with a chevron bracing system, all of which have exceeded the collapse prevention (CP) performance level. In the structure with a zipper bracing system, 6 plastic hinges have been formed at the location of bracing under the Chalfant earthquake and 4 hinges have exceeded the collapse prevention (CP) performance level.

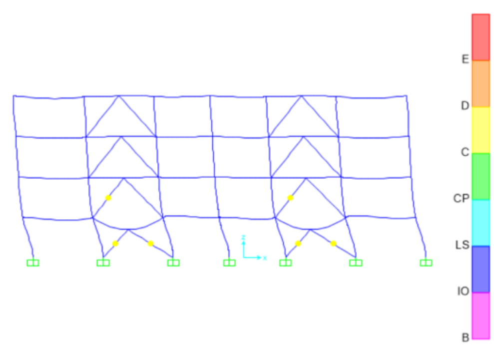

Fig. 26. Distribution of plastic hinges in the 4-story structure with chevron bracing system under Chalfant earthquake.

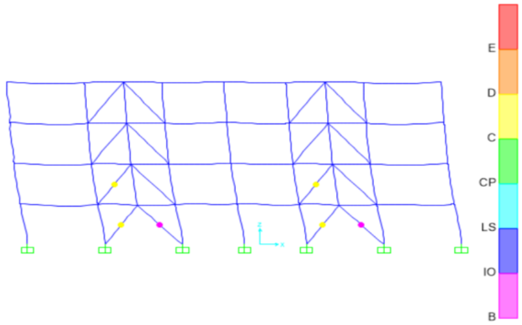

Fig. 27. Distribution of plastic hinges in the 4-story structure with zipper bracing system under Chalfant earthquake.

6.5.2. Distribution of plastic hinges in 8-story structure:

As can be seen in Figs. 28 and 29, 24 plastic hinges have been formed at the location of braces and beam of braced span under the Chalfant earthquake in the 8-story structure with a chevron bracing system. 14 hinges have been operating at immediate occupancy (IO) performance level while 10 hinges have exceeded the collapse prevention (CP) performance level. In a structure with a zipper bracing system, 22 plastic joints hinges have been mainly formed at the location of braces. 4 hinges have been operating at immediate occupancy (IO) performance level, 4 hinges have been operating at life safety (LS) performance level and 14 hinges have exceeded the collapse prevention (CP) performance level. It can be observed that the zipper element has prevented from formation of plastic hinges that have not been formed at the beam (beam of the second story) and has transferred them toward the braces. Also, the displacements and deformations occurred in the second story of the structure braced with the chevron bracing system have not been occurred in the structure braced with the zipper bracing system.

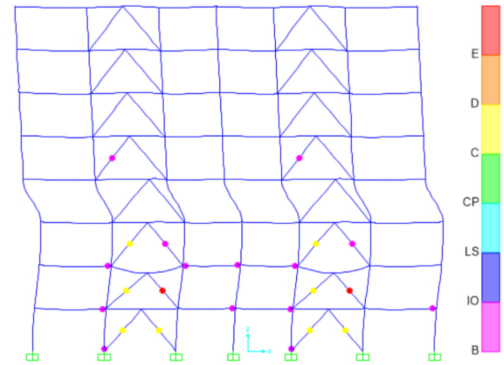

Fig. 28. Distribution of plastic hinges in the 8-story structure with chevron bracing system under Chalfant earthquake.

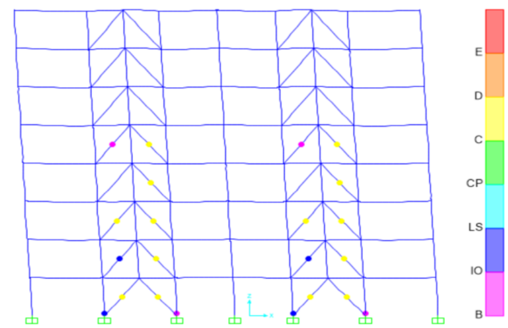

Fig. 29. Distribution of plastic hinges in the 8-story structure with zipper bracing system under Chalfant earthquake.

6.5.3. Distribution of plastic hinges in the 12-story structure:

As can be seen in Figs. 30 and 31, 84 plastic hinges have been mainly formed at the location of braces and beams under the Chalfant earthquake in the 12-story structure with a chevron bracing system. 71 hinges have been operating at immediate occupancy (IO) performance level, 9 hinges have been operating at life safety (LS) performance level and 4 hinges have exceeded the collapse prevention (CP) performance level. In a structure braced with a zipper bracing system, 80 plastic hinges have been mainly formed at the location of braces under the Chalfant earthquake. 69 hinges have been operating at immediate occupancy (IO) performance level and 11 hinges have been operating at life safety (LS) performance level.

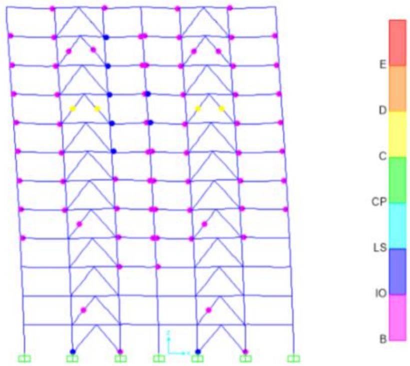

Fig. 30. Distribution of plastic joints in an 12-story structure with a chevron brace under the Chalfant Earthquake.

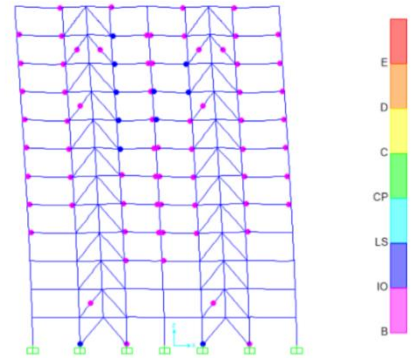

Fig. 31. Distribution of plastic joints in an 12-story structure with a zipper brace under the Chalfant Earthquake.

The zipper member has reduced the number of CP hinges to zero in the 12-story structure, although it is not possible to issue a general rule on this subject because different results have been obtained under other earthquakes.

Tables. 8 and 9 show the number of plastic hinges formed in IO, LS, and CP performance levels for 7 earthquake records, separately.

Table 8. Number of plastic hinges formed in 4-, 8- and 12 -story structures braced with chevron bracing system under different earthquake records.

Earthquake / Stories	CP			LS			IO			Total		
	12	8	4	12	8	4	12	8	4	12	8	4
Coalinga	6	8	2	23	2	0	62	13	2	91	23	4
Chalfant	4	10	6	9	0	0	71	14	0	84	24	6
Kobe	18	0	2	47	0	0	53	0	0	118	0	2
Landers	0	0	4	5	0	0	62	0	4	67	0	8
Mammoth Lakes	0	13	6	0	20	0	32	27	1	32	60	7
Manjil	0	2	2	0	0	0	47	0	2	47	2	6
Tabas	12	0	4	34	0	0	65	0	2	111	0	6

Table 9. Number of plastic hinges formed in 4-, 8- and 12 -story structures braced with zipper bracing system under different earthquake records.

Earthquake / Stories	CP			LS			IO			Total		
	12	8	4	12	8	4	12	8	4	12	8	4
Coalinga	4	7	2	19	3	0	51	12	2	74	22	6
Chalfant	0	14	4	11	4	0	69	4	4	80	22	6
Kobe	18	0	2	31	0	0	42	0	4	91	0	6
Landers	0	0	3	6	0	0	59	0	4	65	0	7
Mammoth Lakes	0	9	6	0	14	0	24	36	4	24	589	10
Manjil	0	0	0	0	0	0	39	2	2	39	2	2
Tabas	8	0	4	26	0	0	60	0	0	94	0	4

6.6. Damage index

The damage index is defined as the sum of the CP plastic hinges in all earthquakes divided by the total plastic formed hinges. According to Tables. 8 and 9, the damage index for structures braces with chevron and zipper is calculated as follows:

6.6.1. Damage index for 4- story structure:

Damage index for structure braced with chevron bracing system $= \dfrac{26}{37} = 0.703$

Damage index for structure braced with zipper bracing system $= \dfrac{21}{41} = 0.512$

In the structure braced with a zipper bracing system, the damage index has decreased by 27% compared to the structure braced with a chevron bracing system.

6.6.2. Damage index for 8-story structure:

Damage index for structure braced with chevron bracing system $= \dfrac{33}{109} = 0.302$

Damage index for structure braced with zipper bracing system $= \dfrac{29}{105} = 0.27$

In the structure braced with a zipper bracing system, the damage index has decreased by 11% compared to the structure braced with chevron bracing system.

6.6.3. Damage index for the 12-story structure:

Damage index for structure braced with chevron bracing system $= \dfrac{40}{550} = 0.073$

Damage index for structure braced with zipper bracing system $= \dfrac{30}{467} = 0.064$

In the structure braced with a zipper bracing system, the damage index has decreased by 12% compared to the structure braced with a chevron bracing system.

7. Conclusion

After analyzing the results in steel moment-resisting frames braced with chevron and zipper bracing systems under near-fault earthquakes and comparing them to the results of SAP2000 software in structures with different number of story, the following results have been obtained:

1. All considered frames enter the nonlinear stage.
2. Plastic hinges are mainly formed at the location of braces firstly and they are formed in other members secondly for both structures braced with chevron and zipper bracing systems.
3. According to the results of 4-, 8- and 12-story structures, it can be concluded that the use of zipper member has decreased permanent displacement of the structures about 10% to 30%.
4. The zipper member has decreased the maximum displacement of the roof in 4-, 8- and 12-story structures by 11%, 25%, and 27%, respectively.
5. The zipper member can reduce the relative story displacements (story drift). In other words, the sequential stories experience fewer displacements relative to each other. Therefore, the presence of a zipper member in the bracing system can make the structure to be more integrated under the lateral forces.
6. The zipper member has prevented the formation of plastic hinges in the beams and it has transferred then toward braces. In a desired design, it is expected that the brace is damaged at the first stage and the beams and columns to be damaged in the next stage, so the use of the zipper member can help get this idea.
7. The displacements and deformations of the beam in the braced spans are significantly less in the presence of a zipper member.
8. Using zipper brace has reduced damage index in 4, 8, and 12-story structures by 27%, 11%, and 12% respectively.
9. The vertical member of the zipper brace connects the beam of the lower

story to the beam of the upper story vertically; in fact, it appears as a column in the middle of the story, where the openings cannot be placed, while they can be placed on its left and right sides. The limitation imposed by the vertical member of the zipper brace is that openings must be placed on two sides of the zipper brace.

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