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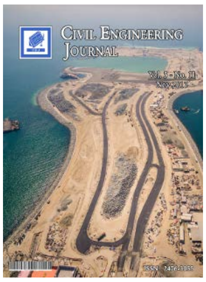

Comparison Study of CBFs and EBFs Bracing in Steel Structures with Nonlinear Time History Analysis

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Abstract

Steel concentrically braced frames (CBFs) and Steel eccentricity braced frames (EBFs) are frequently used as efficient lateral load resisting systems to resist earthquake and wind loads. This paper focuses on high seismic applications where the brace members in CBFs and EBFs dissipate energy through repeated cycles of buckling and yielding. The present study evaluates in detail the design philosophies and provisions used in the United States for these systems. The results of a total of 176 analysis of nonlinear history of seismic behavior of CBFs and EBFs braces have been presented. Notable differences are observed between the performances of the CBFs and EBFs designed using American provisions. The similarities and differences are thoroughly discussed.

Keywords: Steel Structures; CBFs; EBFs; Parameters Studied; Time History Analysis; American Provision.

1. Introduction

Currently, moment resisting frames, concentrically braced frames, eccentrically braced frames, knee braced frames steel plate shear, and zipper braced frames are being commonly used as lateral (i.e. seismic and wind) force resisting systems for steel structures. While new systems such as buckling restrained braced frames are gaining popularity, Moment resisting frames (MRFs) and concentrically braced frames (CBFs) are considered as two of the most popular systems among these alternatives. Although MRFs provide more architectural freedom, compared to the CBFs, they are expensive. CBFs have been quite popular since the 1960s mainly because of their economic advantages over MRFs particularly in cases where the drift requirements govern the design. Furthermore, beam-to-column connections of MRFs suffered premature fractures in the 1995 Kobe and the 1994 Northridge earthquakes [1-2]. In the aftermath of these earthquakes, considerable research and development projects were conducted in the US, Japan, Europe, and elsewhere to develop new moment connections that have sufficient strength, stiffness, and ductility to perform satisfactorily during future strong seismic events. However, the new MRF connections and the modifications made to then existing moment connections have caused their cost of construction and inspection to increase significantly, making the use of CBFs even more economical. More recently, the 2011 Christchurch earthquake in New Zealand resulted in fracture of several eccentrically braced¬ frames (EBFs), further adding to the popularity of CBFs.

The CBF system is currently one of the most widely used seismic load resisting systems in steel structures; it is easy to design and the most efficient especially in controlling lateral drifts of buildings. In recent decades, a significant amount of research has been conducted on the seismic behavior and design of CBFs. A major portion of these studies has focused mainly on the response of bracing members and their connections [3-4]. Extensive experimental [5-7] and numerical [8, 9-24] investigations have also been undertaken to study the behavior of single-story and multi-story CBFs under severe

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loading scenarios, assessing both the system level and component level responses. In the United States, steel CBFs are designed according to the AISC Specification for Structural Steel Buildings [10], hereafter referred to as AISC360, as well as the special seismic design rules of the AISC Seismic Provisions for Structural Steel Buildings [11], which is referred to here as AISC341. In Europe, CBFs are designed according to the regulations of the Eurocode3: Design of Steel Structures– Part 1-1: General Rules and Rules for Buildings [12], hereafter referred to as EC3, as well as the seismic provisions of the Eurocode 8: Design of Structures for Earthquake Resistance – Part 1: General Rules, Seismic Actions and Rules for Buildings [13], which is referred to here as EC8. Due to rapid globalization, engineers are now faced with the challenge of being competent with several design provisions. Owners may require the use of widely accepted design codes regardless of the location of the structure. Nevertheless, in some cases, there can be substantial dissimilarities between the regulations of different provisions which might significantly affect the behavior of the designed structure..

2. Objectives and Scope

Seismic design provisions in AISC341 and EC8 on MRFs and EBFs are quite similar. However, the rules on the seismic design of CBFs in these provisions have evolved separately and have some significant philosophical as well as procedural differences. The main objective of this paper is to study the similarities and differences between CBF and EBF braced Characteristics in the United States code. Section 5 has the data and results of the designed archetype structural models subjected to a large set of ground motions as well as the observations made on the responses. Clear comparisons between the seismic performances of the CBFs and EBFs designed according to the provisions are made with the aid of graphical presentation of the results. Similarities and differences between the behaviors are highlighted and recommendations are developed for practicing engineers.

3. Design Provisions in AISC 341

3.1. Definition and Geometries

AISC341 provisions define CBFs as systems where horizontal forces are mainly resisted by members subjected to axial forces. The centerlines of adjoining columns, beams, and braces should be concentric. However, the AISC341 provisions allow eccentricities less than the beam depth if the resulting member and connection forces are addressed in the design. Astaneh-Asl [25, 26] has shown that such relatively small amount of eccentricity, if introduced correctly, can improve the ductility of the gusset-plated connection without increasing the size of the gusset plate or the beam.

3.2. Seismic Demands

To make a fair comparison, seismic demands (i.e. the load effects) should also be considered. In the United States, seismic demands on structures are calculated based on the regulations of the Minimum Design Loads for Buildings and Other Structures [14], hereafter referred to as ASCE7-10. The provision defines a design response spectrum to be used for determining the design base shear force. In ASCE7-10, two spectral acceleration values, S_s and S_1, are considered which are established using acceleration maps and depend on the location of the structure. The S_s and S_1 parameters are based on risk targeted maximum considered earthquake (MCER) ground motions and are defined as mapped MCER, 5% damped, spectral response acceleration parameter at short periods and at a period of 1 s, respectively. These acceleration values are modified to arrive at S_{MS} and S_{M1} which are the MCER spectral response acceleration parameters adjusted for site class effects. These parameters are finally multiplied by a factor of 2/3 to arrive at S_{DS} and S_{D1}, which represent design spectral response acceleration parameters. For the case of ASCE7-10, the values of S_s and S_1 are considered to be equal to 1.5 g and 0.6 g, respectively. Furthermore, for a site class D (stiff soil), the design spectral accelerations, S_{DS} and S_{D1}, are equal to 1.0 g and 0.6 g, respectively. Figure 1 shows that the response spectra developed based on ASCE7-10 for a high seismic region with stiff soil.

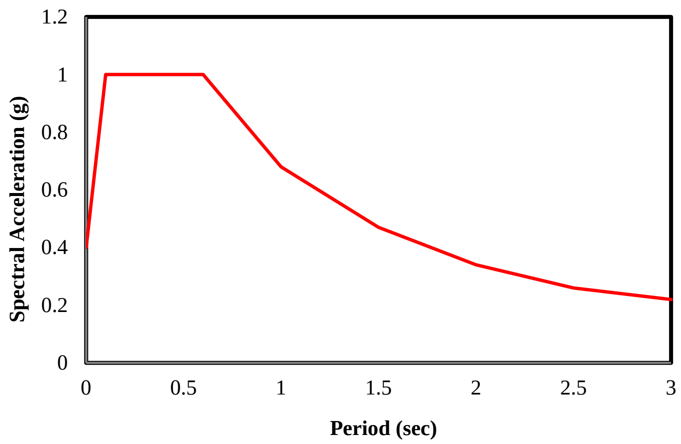

Figure 1. design response spectra (ASCE7-10)

4. Design of Archetypes

4.1. Selected Geometries, Materials and Member Sizes

A total of four 15-story CBF archetypes were designed considering the following configurations:

- Two-story X-braced; referred to hereafter as the split X-braced configuration.
- Single-story X-braced; referred to hereafter as the X-braced configuration.
- Inverted V-braced; referred to hereafter as the V-braced configuration.
- EBF-braced; referred to hereafter as the EBF-braced configuration.

The designs assumed the plan geometry of the SAC 15-story building [15], with X- or split X-braced frames in one direction, and V-braced or EBF-braced frames in the other direction. In all cases, the braced frames were placed at the perimeter of the building as shown in Figure 2. The elevation view depicted in this figure shows the split EBF-bracing system as a representative. The studied building consists of a basement and fifteen stories above the ground level. The heights of the basement and the first story are 3.6 and 5.4 m, respectively, while other stories are 3.4 m high. Dimensions of the floor plan are 24 m by 24 m and the bay widths are 6 m. The mass values used in the SAC building [15] were directly used in the designs. For the US designs, the building was assumed to belong to the seismic design category D_{max} according to FEMA P695 [16], which represents the highest seismic hazard level with $S_{DS} = 1.0\ g$ and S_{D1} = 0.6 g. Consequently, the utilized design spectra are identical to those shown in Figure 1.

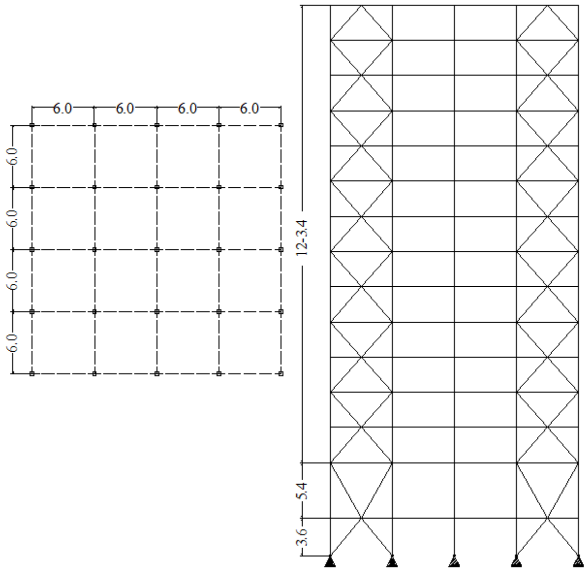

Figure 2. Floor plan and elevation view of the archetype building (all dimensions are in meters)

Since the current study is focused on the seismic design and performance of CBFs, only the braced bays were designed considering their share from gravity and earthquake loads. For the US designs, ASCE7-10, AISC360, and AISC341 documents were utilized. Beams and columns were selected from American wide flange sections while hollow structural sections (HSS) were considered for braces. Braces were assumed to be from ASTM A500 Grade B steel with a yield strength of 317 MPa. On the other hand, beams and columns were assumed to be from ASTMA992 steel with a yield strength of 345 MPa. The equivalent lateral force procedure was used in the design of plane CBFs and the inter story drift ratios were limited to 0.02 in all cases. Second-order effects were also incorporated in the design considering a leaner column concept. The selected member sizes of the designed split X-braced archetypes are depicted in Figure 3.

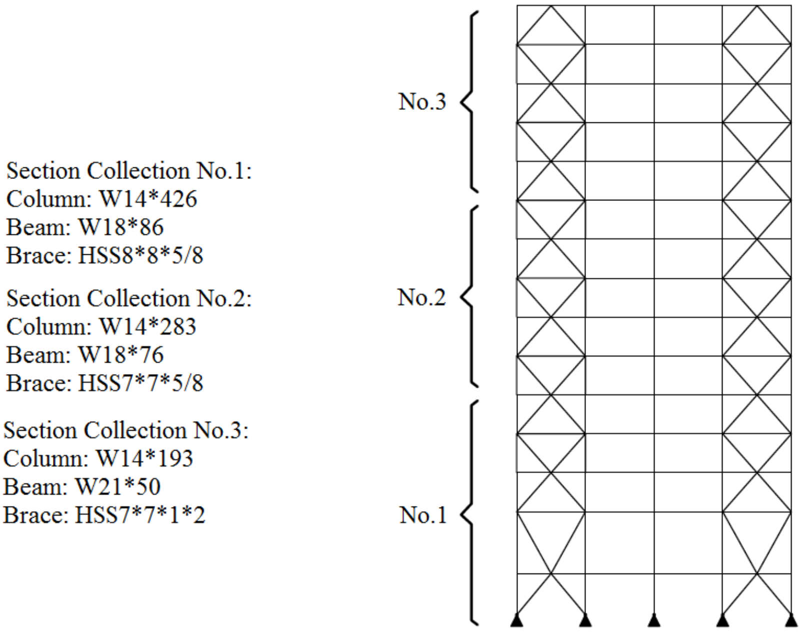

Figure 3. Member sizes for the designed original split X-braced (US-2X) archetypes

4.2. Split X-braced System

The US design for the split X-braced frame (designated as US-2X in Figure 3) has a fundamental period of 1.63 s according to an eigenvalue analysis. This value is, however, higher than the upper bound on the fundamental period, as per ASCE7-10, which is $1.42\ s$. Therefore, the base shear (for one frame) was determined as $12000\ kN$.

4.3. X-braced System

For the X-braced configuration, the US design (designated as US-X in Figure 3) has a fundamental period of $1.21\ s$ which is again higher than the upper bound defined by ASCE7–10, resulting in a base shear of $13591\ kN$ for frame.

4.4. V-braced System

The fundamental period of the US design for the V-braced frame (designated as US-V in Figure 3) is $1.58\ s$ which is once more higher than the upper bound defined by ASCE7-10. Therefore, the base shear for one braced bay of US-V was again found to be $11308\ kN$.

4.5. EBF-braced System

For the EBF-braced configuration, the US design (designated as US-EBF in Figure 3) has a fundamental period of $2.54\ s$ which is again higher than the upper bound defined by ASCE7–10, resulting in a base shear of $11004\ kN$ for frame.

5. Comparison of Seismic Behaviors

5.1. Details of Numerical Simulations

A comprehensive study has been undertaken to investigate the seismic behavior of the archetypes designed according to AISC341. Numerical models of these structures were subjected to a large set of ground motions and the simulation results of a total of 176 analyses were examined to highlight the similarities and differences between the responses. All of the analyses were conducted using the finite element package, ABAQUS 2017 [17], considering two-dimensional models of the structures. Both material and geometric nonlinear effects were included in the analyses. Since the study is focused on the seismic performance of CBFs and EBFs. A von Mises plasticity constitutive model with kinematic hardening considering bilinear material behavior was used in the numerical simulations. All members of the frame and the leaner column were modeled using B21 two-node linear beam elements, each having one integration point at the mid-length and five integration points through the height of the section. A rather small mesh size was employed with 14 elements in each brace, 16 elements in each beam, and 10 elements in each column.

All of the beam-to-column, brace-to-column, and brace-to-beam connections are pinned and were modeled in the finite element models using the pin-type multi-point constraint (MPC Pin) technique available in ABAQUS. Furthermore, at each story level, the horizontal displacement of the leaner column was constrained to that of the CBF column using the equation technique of ABAQUS. Based on the recommendation of FEMA P695 [16] the reactive seismic mass of each floor was determined as 1.05DL + 0.25LL using the dead and live load values reported for the original SAC building [15]. It is worth noting that 25% of this mass was considered in the models since the braced bay under investigation will only receive one-fourth of the base shear. The vertical loads on the leaner column were also calculated in a similar manner. To properly capture the buckling behavior of braces in CBFs, it is necessary to include initial imperfection in these members. The comprehensive studies of D'Aniello et al. [18-19] have shown that the introduced amount of imperfection can affect the seismic response of the CBF structure under consideration. Furthermore, it is suggested by these researchers to use theoretical formulations instead of empirical methods for estimating the proper level of imperfection, since the former depends not only on the brace length, but also on the flexural strength interaction of the brace cross section. Nevertheless, considering the comparative nature of the current study, it was finally decided to use an initial imperfection equal to 0.1% of the brace length (i.e. L/ 1000) in all models.

This amount of imperfection has been previously reported as a reasonable estimate by Deierlein et al.[20] and also used successfully in numerical simulations of many other studies such as Fell [21] and Okazaki et al. [6]. In any case, more advanced and accurate methods for estimating the initial imperfection can also be employed, as thoroughly discussed by D'Aniello et al.[18-19]. Another important issue in nonlinear time history analysis is damping. Although the equivalent viscous damping approach using Rayleigh formulation is commonly adopted, great care should be taken while using this method. Based on this approach, the damping matrix, C, is determined as follows:

$$C = \alpha M + \beta K \tag{1}$$

Where M and K are the mass and stiffness matrices, respectively, and α and β are proportionality factors which are calibrated to result in a predefined percentage of critical damping (ξ) at two desired vibration periods. Many researchers such as Hall [22], D'Aniello et al. [18], and Charney [23] have illustrated that using this method can produce unrealistically large damping forces and unconservative displacement demands in nonlinear analyses. This issue was also clearly observed in the present study during initial analyses. Brace members experienced very high velocities when yielded which in turn created large damping forces in these members, sometimes even comparable to their plastic capacities. As discussed by Deierlein et al. [20], there is no solid consensus on a solution scheme for this problem. Many approaches have been proposed such as using a capped viscous damping formulation or using the tangent stiffness matrix instead of the initial (or elastic) stiffness matrix as K. However, most of these methods cannot be readily implemented in most commercial finite element software. A more convenient approach was consequently chosen in this study, as described by Deierlein et al. [20], where the stiffness proportional part of damping is minimized, and damping is mostly represented by the mass proportional part. This approach reflects the fact that the stiffness proportional part of damping is considered to have a more pronounced effect in producing unrealistic damping forces. To this end, α=0.3 and β=0.00005 were used in all models. The value of β was found to be sufficient enough to reduce high-frequency noises in the response while minimizing the stiffness proportional part of damping. Free vibration analyses of the designed archetypes revealed that the used factors correspond to a ξ of about 5% in the 1st mode of vibration and 2% in the 2nd mode. The developed finite element models were finally subjected to a set of 13 ground motion records presented in FEMA P695 [16]. This document provides two sets of records which are designated as “far-field” and “near-field” sets by FEMA. In the present study, the far-field set was utilized. It should be mentioned that these ground motions are first normalized based on their peak ground velocities and then, considering the fundamental period of the designed archetypes, are scaled by a factor of 0.6012 during the analysis to reach the MCER spectral demand of the seismic design category D_{max}. (Figure 4).

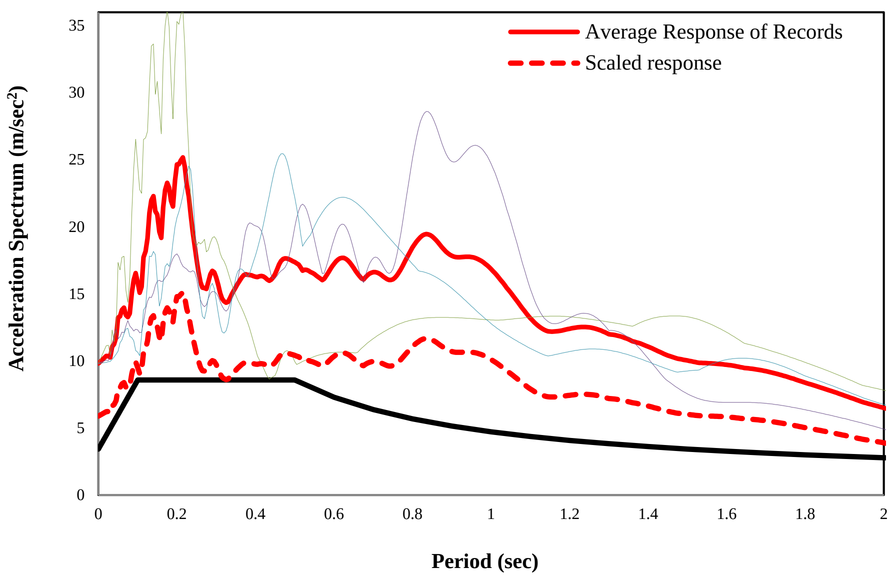

Figure 4. Scale the record spectra according to the design response spectra

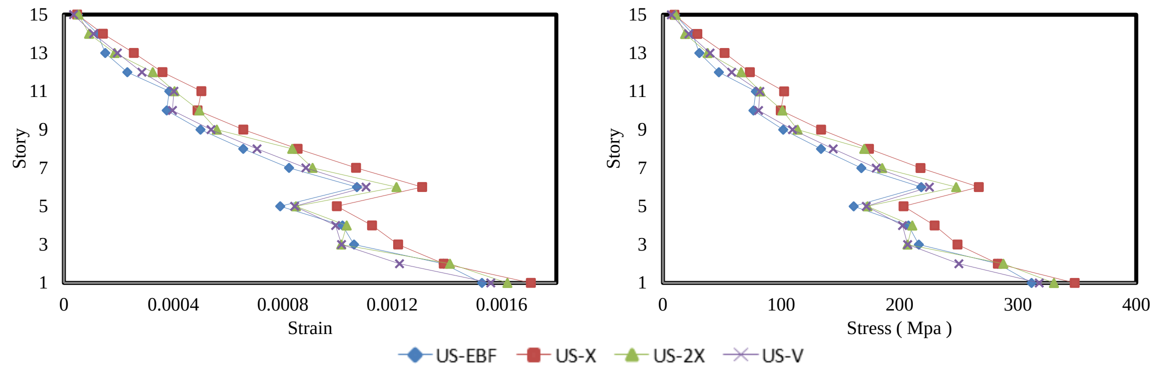

Figure 5. Strain and Stress values in the side columns

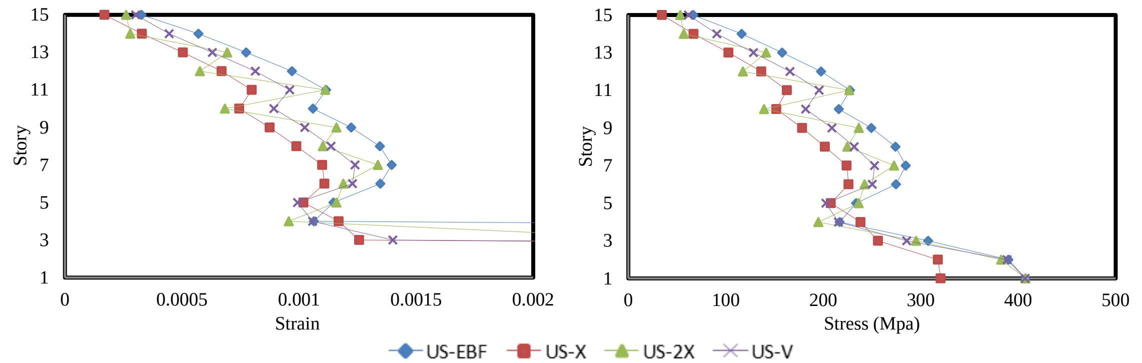

Figure 6. Strain and Stress values in the braces

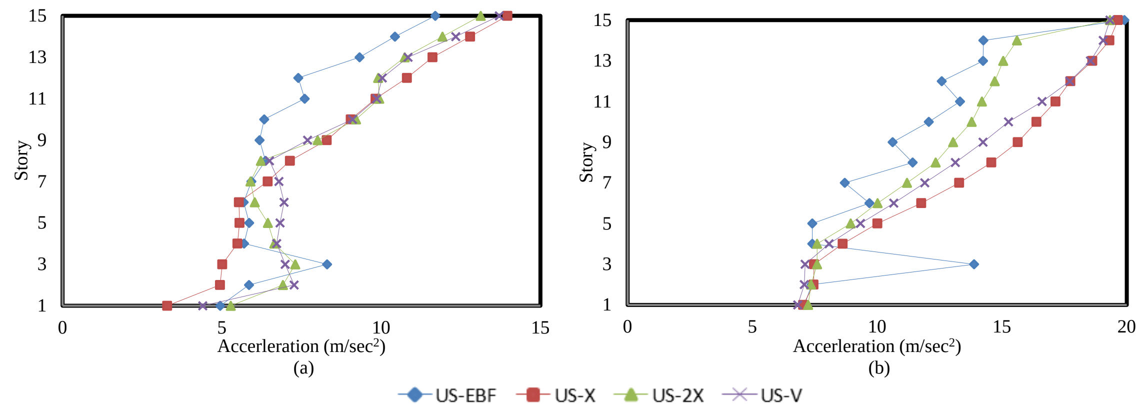

Figure 7. Horizontal (a) and vertical (b) acceleration values

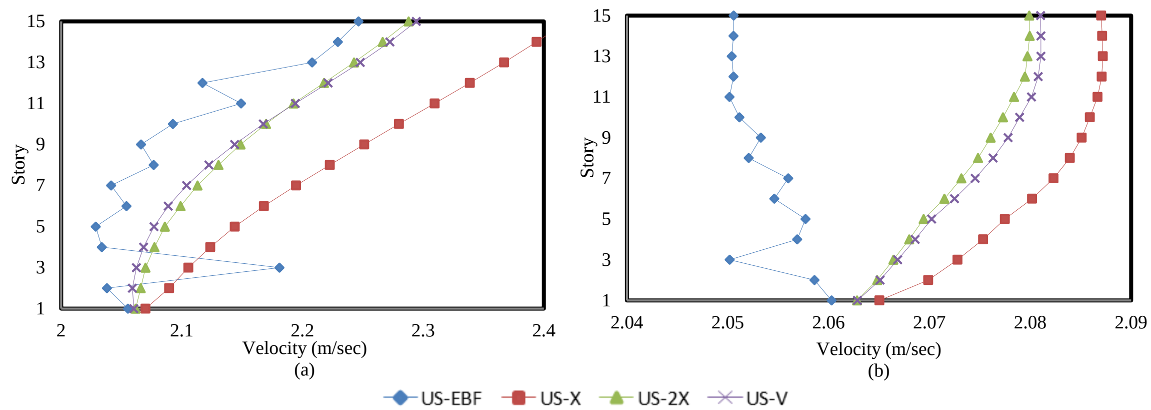

Figure 8. Horizontal (a) and vertical (b) velocity values

5.2. Compare the Results of all Braced Frames

The strains and stress in the side column in the studied frames show that the X-braced frame has the highest stress and strain value and we had that expectation due to the hardness of the X-braced (Figure 5).The strains and stress in the braces in the frames show that the EBF-braced frame has the highest stress and strain value (Figure 6). According to nonlinear time history analysis, the history of acceleration and speed and displacement in maximum points of stories is presented in Figures 7, 8 and 9. The divergence of the frame in the horizontal and vertical direction indicates that it differs. The absorbing energy into the frame, according to the analysis, can be seen in Figure 10. The difference between the absorbing energy values of the structures in the studied frames is visible. The values of support reactions in the frames at the base level are presented in Figure 11**.** Note that the horizontal axis of the graph represents the naming of the axes of the frames (Figure 2).

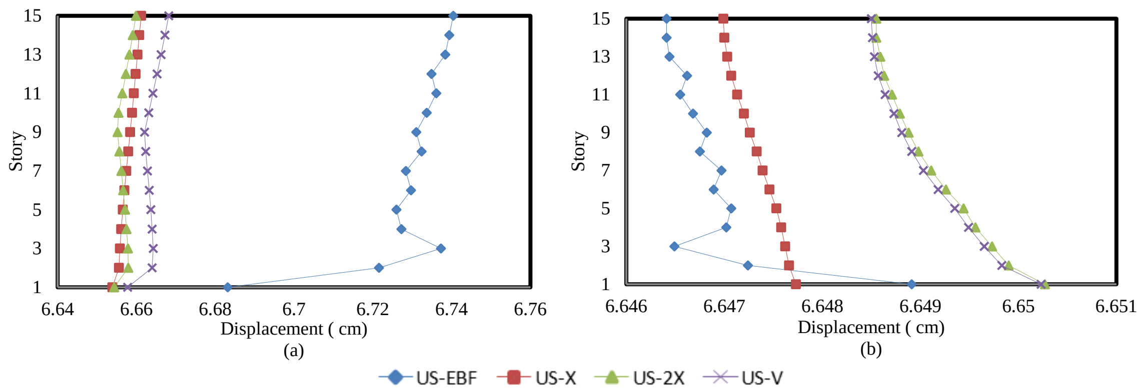

Figure 9. Horizontal (a) and vertical (b) displacement values

The results indicate that US-EBF frames show good seismic behavior under shaking ground motion. The displacement value of the US-EBF frames in the horizontal direction is maximum and in the vertical direction is the minimum value compared to other braced frames. In examining the energy absorption of the frames, it can be stated that the amount of energy absorbed by the US-EBF frame reaches 25,000 KJ But in the US-X frame, this does not exceed 800 KJ. Regarding Figure 11, we can say that the braced columns (1, 2, 4, 5) have greatest internal force in their members and the middle column (3) has smallest internal force.

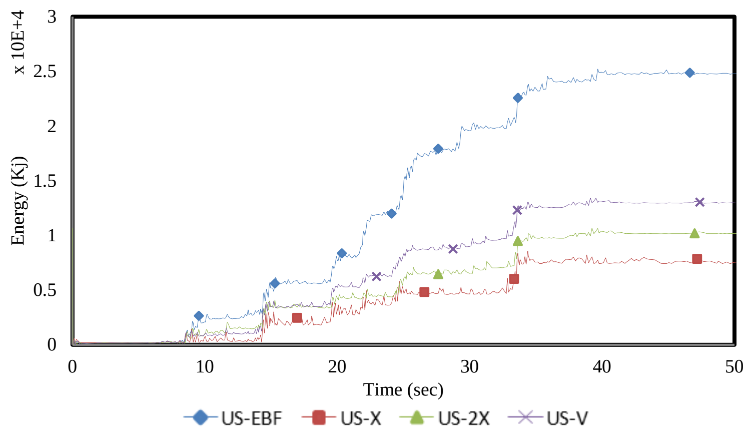

Figure 10. Internal energy value in the frames

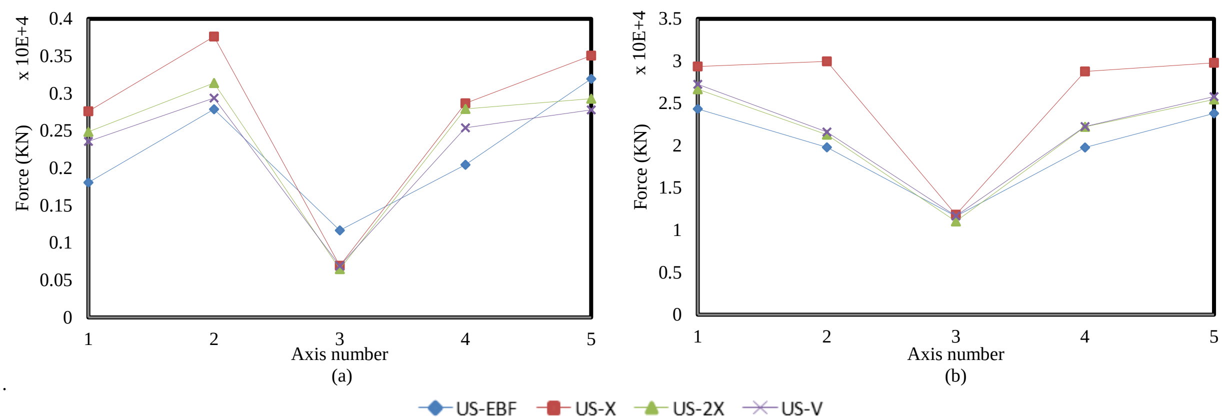

Figure 11. Horizontal (a) and vertical (b) reactions of supports

6. Conclusion

The paper has explored in detail the similarities and differences between the seismic behaviors of steel CBFs and EBFs designed to the provisions of AISC341 developed in the United States. The following can be concluded from this study:

- In comparison, the stress of columns US-EBF and US-V shows that in lower storys, US-EBF braced columns are more than US-V frames and As the height increases in the structures, the amount of stress in the columns increases from US-V to US-EBF.
- Stress and strain in US-EBF are more than other braces, but this value in the lower storys is approximately equal to in the braces US-V and US-2X. also, it is about 25% more than the stress and strain levels in the four braces.
- The difference in stress in the adjacent floors in bracing US-2X is more than the other braces.
- The difference in stress in height in bracing US-EBF is more than the other lower story braces.
- Horizontal displacement in US-EBF frame is greater, but the US-2X, US-X and US-V frames are approximately equal but the vertical displacement is less than US-EBF.
- US-EBF absorbs more energy, respectively, therefore, it can be said that the ductility US-EBF differs greatly from the other frames.
- US-EBF frames improve the performance of the columns and balances force distribution and increases the cross-section of braces and reduces the cross-section of columns compared to other braces.
- US-X and US-V show lower storey displacement indicating that these systems have strength and stiffness.

7. Acknowledgements

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