

Research Paper

Determining and Investigating the Coefficient of Behavior of the Eccentrically Braced Frames (EBF) Equipped with a Simple Replaceable Fuse (SRF) by Young, Priestley-Pauli and Energy Methods

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ABSTRACT

One of the methods to reduce damages caused by earthquakes is to concentrate damages in predetermined members. During an earthquake, these members minimize the damage caused to other members of the structure by absorbing a major part of the earthquake force. These members act like an electrical fuse that is blown under an unauthorized current of electricity and prevents damage to the wiring and electrical appliances used; Therefore, these members are called "structural fuse" and sometimes abbreviated as "fuse". EBFs are one of the most important resistant systems for bearing lateral loads. Researchers, paying attention to the simple replace ability approach, have introduced a shear fuse with joint connection in the connection area, which is named as a simple replaceable fuse. In this project, a series of numerical and laboratory researches are carried out in full scale, and this article refers to the comparison of design parameters. In this article, a brief explanation of Young's, Presley-Pauli's and energy methods is presented in each section, and then the design parameters are determined and compared. ETABS software was used for frame design and ABAQUS was used for numerical analysis. The comparison results show that the use of this fuse in EBF has significantly improved the design parameters. Due to the simplicity of manufacturing and installing this fuse, using this method optimizes the design and saves the costs of construction and seismic improvement.

Keywords:

Eccentrically braced frames; EBF; Fuse; Replaceable link beam; Articulated connection

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1. Introduction

One of the common and earthquake-resistant systems are lateral bracing frames. Among these systems, bending frames, especially the special bending frame, can be considered as a seismic bearing system that has high ductility and low lateral stiffness. On the other hand, the concentrically braced frames, although they have high stiffness and are suitable, but they do not have enough ductility. In other words, in special bending frames, the displacement criterion is usually the controller, while in EBFs, the ability to absorb and dissipate the induced energy of an earthquake is not so high. EBFs, in fact, are a suitable combination of bending frames and concentrically braced frames, which have both stiffness and ductility (Kasai et al., 1986).

In the early 1970s, EBFs were used for the first time in Japan with the aim of having both the characteristics of flexural frames (high ductility) and concentrically braced frames (high stiffness), and now research on this system continues (Arce, 2002; Hjelmstad & Popov, 1983; Hjelmstad & Popov, 1984; Kasai & Popov, 1986; Kasai & Popov, 1986; Malley & Popov, 1983; Malley & Popov, 1984; Popov & Bertero, 1980; Popov & Engelhardt, 1988; Popov et al., 1987; Popov & Malley, 1983; Ricles & Popov, 1987; Ricles & Popov, 1987; Roeder & Popov, 1978; Vetr & Ghamari, 2019).

According to AISC criteria, diverging braced frames are frames in which the braces are attached to the beam at each span with a small distance from each other on the longitudinal axis of the beam or with a small distance from the beam-to-column connection node. In these frames, the lateral seismic behavior of the structure is a combination of the bending-shear function of braced span beams and columns and the tension-compression function of braces.

In structures that are designed in accordance with modern seismic codes, it is necessary to enter the behavior of the structure into the non-linear region. It is obvious that if the behavior of the structure enters the non-linear area, the occurrence of damage and permanent changes in some elements of the structure is inevitable. Experience has shown that after moderate and strong earthquakes, it is necessary to repair and replace the

damaged elements of the structure for reuse. Therefore, it is very important to use structural systems that concentrate many deformations in a certain element and at the same time have easy replaceability.

In this research, the behavior of EBFs with replaceable link beams has been studied. Unlike other past researches, in addition to paying attention to the proper function of the fuse in the structure, the researchers have put the simplicity of its construction and easy replacement on the agenda. In this system, the link beam is connected to the out-of-link beam using a pin connection, so that it can be easily replaced while exhibiting a complete shear behavior. The results showed that the system that leads to the yielding of the horizontal beam between the diverging braces has high ductility and energy dissipation against seismic loads. In these systems, severe inelastic deformation occurs in the shear panel (link beam) and the internal energy is lost by this member. Figure (1) shows yielding and plastic joint formation in EBFs.

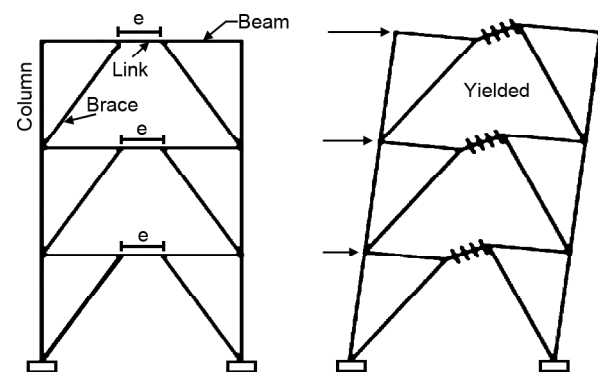


Figure 1. Yield process and plastic joint formation in horizontal shear bond beams.

Since all inelastic deformation occurs in the shear panel (horizontal link beam), the main frame members are not damaged. Of course, despite the high seismic energy dissipation, EBFs with horizontal bond beam (H-EBF) also have disadvantages. The main weakness of this system is that it is very difficult and time-consuming to repair or replace the link beam after an earthquake (Bouwkamp et al., 2016).

To solve this problem in the (H-EBF) system, a new system called EBFs with Simple Replaceable Link Beam (SRF) is proposed. In this system, the

joint beam is connected to the beam outside the joint area with a full joint connection, in the form of a tongue and a tongue, and using a pair of pins (Figure 2).

2. Fuse

The intended fuse consists of a set of link beams and a number of steel sheets that are connected to the frame with a pair of pins. Due to the forces that arise in them, the sheets should exhibit completely elastic behavior throughout the earthquake and should not enter the plastic zone in any way; and only the link beam can experience hyper elastic deformations. The main purpose of making such a fuse is to guide and control the force of the earthquake in the area of the connecting beam and to protect other elements of the frame from damage. On the other hand, the most important concern of the researchers has been the easy fuse construction and replace ability at the same time as its optimal performance. In most replaceable models, screw connections are used, and after an earthquake, due to the damage caused when the fuse is replaced, it becomes clear that very small deformations have occurred and caused distortion in the connection sheet. This issue causes when the new fuse is replaced, the previous holes do not

match easily with the new fuse, and this issue is the beginning of a series of problems for replacing screw fuses. One of the distinguishing aspects of this study compared to other studies with replaceable links is in the same way of connecting the fuse to the bracing system. In this research, three steel sheets are used to connect the fuse to the out-of-link beam by maintaining the optimal performance of the link beam. The ends of the connecting plates are curved for free rotation, and this makes it easy to replace the fuse. This set of connection (palm and tongued sheets) is connected to a vertical end plate, and in this way it is connected to the beam outside the short link and from the other side to the beam outside the long link.

According to Figure (3), the longitudinal axis of the braces (dashed line) and the longitudinal axis of the fuse (continuous line) intersect at the starting point of the connecting beam. Determining and choosing the location of this collision has been one of the main points in determining the behavior of the system. For example, the intersection of the longitudinal axes of the brace and the pin of the connection area causes instability in the structure, and to solve this issue, the connection area is moved outside the place where the brace and the connecting beam meet.

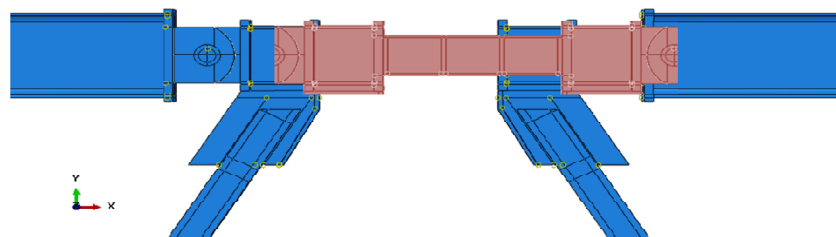


Figure 2. Articulated connection of the bond beam to the out-of-bond beam.

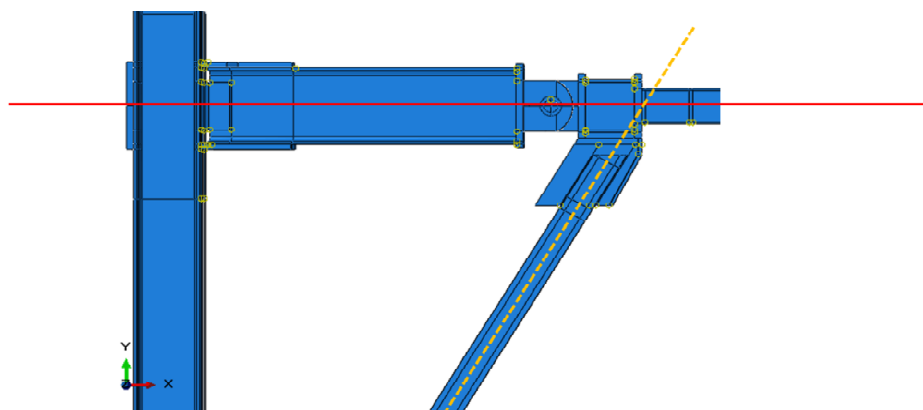


Figure 3. The intersection of the longitudinal axes of the brace and the fuse.

3. Finite Element Modeling (ABAQUS)

For modeling in ABAQUS software, a one-story, one-span frame (which was a part of a ten-story building with educational use in an area with high seismic risk) was subjected to cyclic horizontal displacement. Although the application of gravity load may affect the behavior of the shear bond beam, but because the axial load reduces the capacity of the shear bond beam, the researchers decided not to apply any vertical force to the shear bond beam. The material of the sections is made of 37ST steel with a density considered that this steel is easily available in the market.

This frame is designed using AISC regulations. On this basis, the length of the connecting beam has been chosen in such a way that the life of the connecting beam yields in shear. According to the regulation, to ensure the elastic performance of the members outside the link in the dual systems of special divergent frames and bending frame, the resistance increase factor (ϕ), 2.5, is proposed. Therefore, other members of the divergent frame, except for the link beam, have been designed for a force 2.5 times what was generated in them from a static analysis. Also, one pair of stiffeners are installed at the beginning and end of the joint beam and on both sides of the joint, and two stiffeners are installed along the length of the joint beam on one side of the joint.

The connection of the connecting beam to the out-of-connection beam is provided by a series of curved sheets and pins, and the connection of the out-of-connection beam to the girder-type column is ensured by embedding the sheet in the joist and seat sheet. Also, the connection of the braces is articulated.

Considering that the type of connections is not the subject of this research, for the simulation of welded connections from the Tie clause, and for the curved sheets and pins connecting the link beam to the out-of-link beam, which are in direct contact with each other, from the contact clause and two types of contact vertical and tangential are used. In vertical contact, a Lagrangian equation is used to solve the equations; and tangential contact models the friction between two surfaces using a coefficient of friction (which is user-defined and is about 0.4 for steel). Also, the baseplates are fixed in place

and do not move at all, which is intended to better convey the concept of the baseplates being fixed on the foundation.

The method and element used for meshing is the Structure technique and the solid element called C3D8R, which is an 8-node cubic or brick-shaped, three-dimensional, linear element with a reduced formulation. The reduced formulation is chosen in order to reduce the computation time (because in the case of higher order elements, the computation time becomes very high). At least three members are used in the thickness of the steel wings, plates and stiffeners in order to properly redistribute the bending behavior. To reach the appropriate mesh size, the analysis was done with different mesh dimensions until the analysis results converged. Also, in parts of the model where higher stress concentration is expected, the meshing is finer.

The loading protocol used in this research is based on method B of ASTM regulations. In this loading method, the range of motion is a percentage of the target displacement. These cycles will continue until rupture occurs or a significant drop in resistance is observed. As mentioned, the load application steps are based on the percentage of target displacement; But because each sample has its own final displacement, the loading protocol is different for different samples. But since the purpose of this research is to investigate and compare the behavior of the EBFs with the exchangeable link beam. In this research, instead of the target displacement, the maximum real relative displacement (Δ_m) or drift has been used. The value Δ_m , which is obtained by considering the P - Δ effects in the Δ_m calculation, should not exceed the permissible Δ_a value. In the following relationships, h is the height of the floor. Loading is started with a displacement of ± 0.6 mm in the first stage and the first five stages of loading are repeated once each and the subsequent stages are repeated three times each.

4. Cyclic Displacement Results

The results obtained from the finite element modeling are caused by the simultaneous application of horizontal cyclic displacement to the outer faces of both columns (at a distance of about 1510 mm from the center of the connecting beam, at the level

of the floor). The deformed form of the target frame under the applied displacement is shown in Figure (4).

$$n \text{ buildings with more than five floors} \quad (1)$$

$$\Delta_a = 0.020h$$

Considering that the desired frame is located in a 10-story building, therefore, the maximum real relative displacement will be equal to:

$$\Delta_m \leq \Delta_a = 0.020h = \quad (2)$$

$$0.020h = 0.020 \times 2400 = 48 \text{ mm}$$

According to the results obtained from the

analysis of the frame under cyclic displacement, the main concentration of stresses has occurred on the web of the link beam and while the other parts of the structure remain in the elastic region, the web of the link beam has undergone many deformations upon entering the non-linear region and caused failure. The dissipate energy has become significant. Color graphs of von Mises stress, resulting from horizontal cyclic displacement, are presented in Figure (5). In order to specify more precisely the behavior of the link beam with joint connection in the frame, in Figure (6), color diagrams of shear stress (S, S12) are presented. For shear yielding we must better to use γ_u, γ_y .

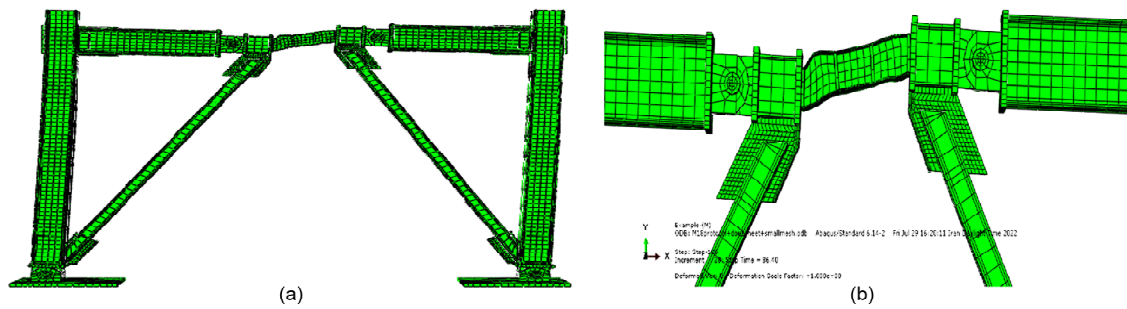


Figure 4. (a) Deformation of the frame and (b) Deformation of the link beam.

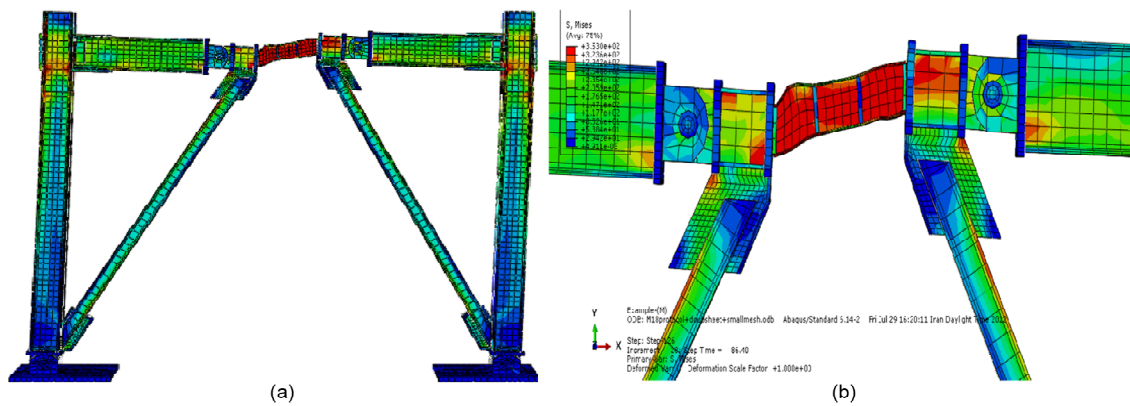


Figure 5. Von Mises stress color diagram, (a) in the frame and (b) in the link beam.

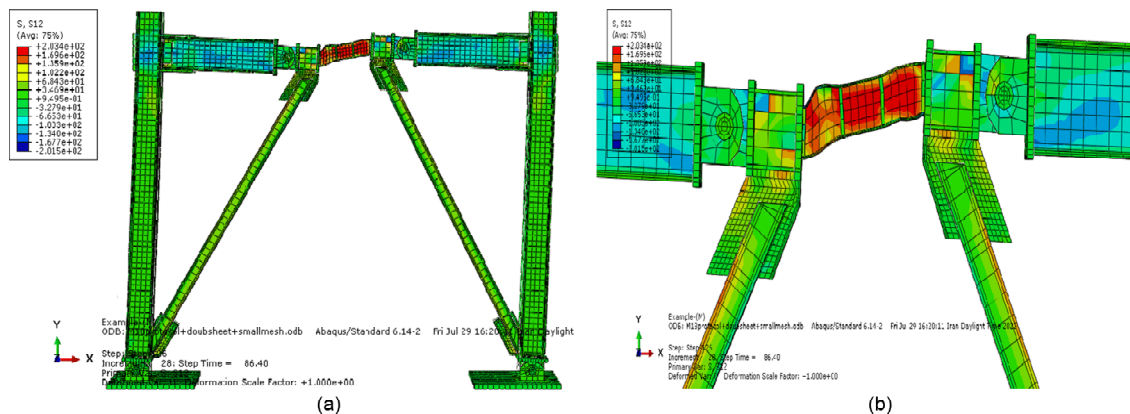


Figure 6. Colored diagram of shear stress (a) in the frame, (b) in the link beam.

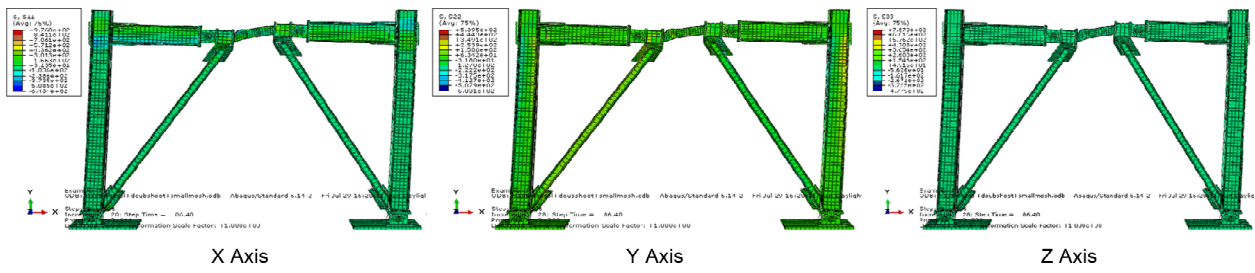


Figure 7. Color diagram of frame axial stresses.

The axial force in the link beam and other frame members is negligible. In Figure (7), the more critical points of these stresses are shown for X, Y and Z directions.

Due to the fact that in this modeling displacement has been applied to the columns of the frame, in order to reach forces such as displacement, the forces applied to both columns at each stage of displacement should be added together according to the direction of the forces. According to the results obtained from the ABAQUS software, the sum of forces and displacements applied in the X direction leads to the drawing of the hysteresis diagram according to Figure (8).

On the other hand, as in Figure (6), it is also clear that if the points presented in the design section

are followed, and the design is done correctly, we will witness the concentration of shear force on the web of the link beam, which means the occurrence of shear failure in the link beam. According to the main idea of the design that for easy exchangeability, there were two bending joints on both sides of the link beam at the beginning, we should never have allowed the formation of a third bending joint in the link beam. Since with the formation of the third bending joint, the failure mechanism was formed in the beam and the frame collapsed without bearing an acceptable amount of shear force. Therefore, yielding has occurred correctly in the joint beam and of the shear type. Figure (9) shows the colored graphs of the equivalent plastic strain (PEEQ) at the maximum displacement.

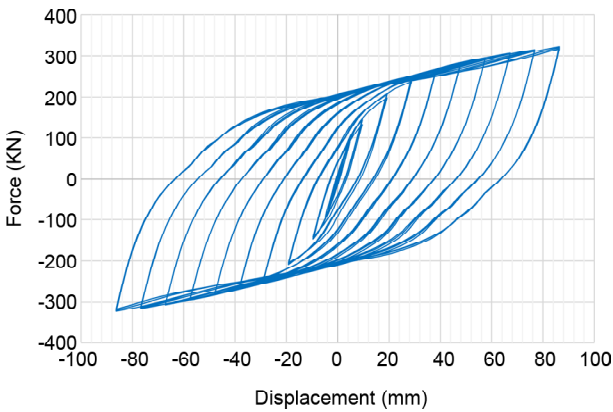


Figure 8. Hysteresis diagram of the frame under cyclic motion.

5. Validation

In another part of this project, which includes full-scale laboratory modeling, the laboratory tests prove the claim of the researchers that the fuse enters the non-linear area and the rest of the frame members are protected during the cyclic loading. In the figure below, the final images of the laboratory test #1, the eccentrically bracing frame system equipped with SRF, clearly show that only the fuse area is failure in shearing. The rest of the EBF is not suffering, Figure (10).

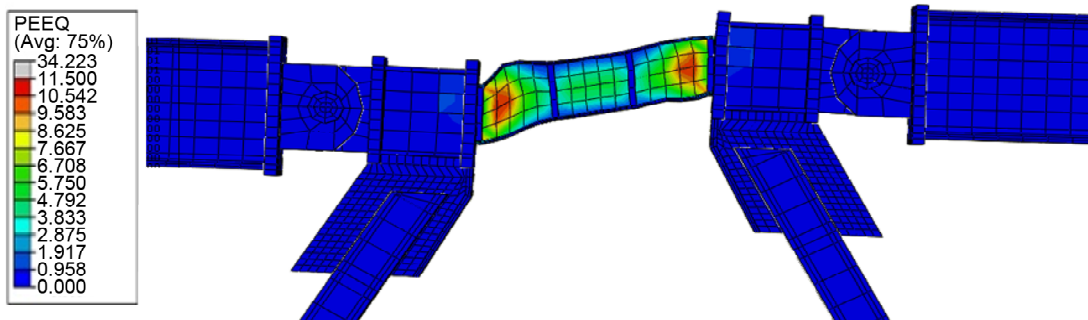


Figure 9. Equivalent plastic strain diagram at maximum displacement by the analysis and tests(a, b, & c).

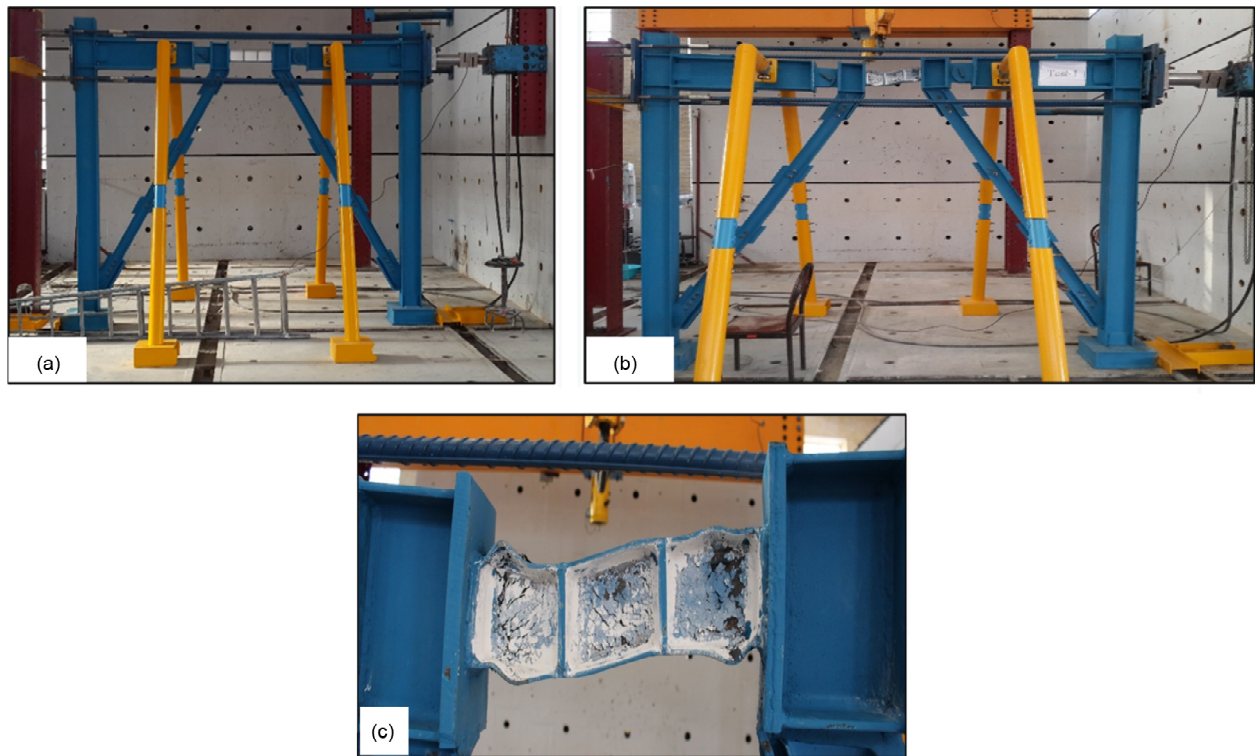


Figure 9b. Continue.

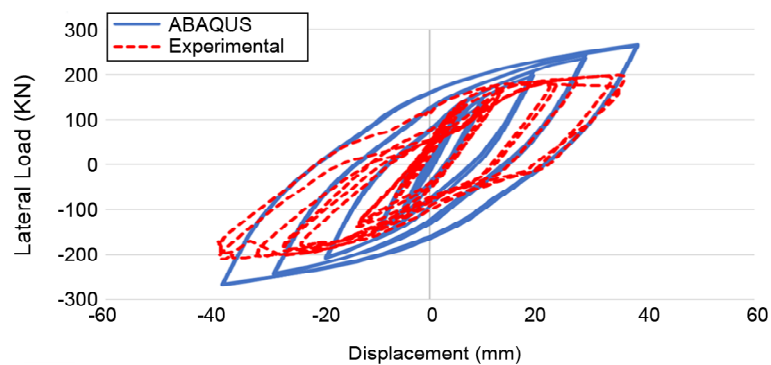


Figure 10. Validation of test (load (KN) -displacement (mm)) with Abaqus software (Vetr & Kordbagh, 2017).

6. Analysis of the Results

So far, many researchers have used different methods to calculate behavior coefficients. By comparing these methods, they can be divided into two main groups: American researchers' methods and European researchers' methods. Generally, the American methods have simpler theoretical bases, but they are more practical. While the European methods have more complex theoretical and analytical bases, it is difficult to use them in practice. In the methods of American researchers, two methods are more prominent than others, and other methods are mostly similar to these methods with slight differences. One of these methods is known as "capacity spectrum", which is the result of

Freeman's research. The second method, which is known as the "ductility coefficient" method, is the result of Yang's researches, which was explained earlier (Uang, 1991). Along with American researchers, European researchers have also researched the estimation of behavior coefficients of structures. Mainly, the methods used by Europeans are divided into two groups of methods based on ductility theory and energy methods. In the following, we determine the design parameters of Young, Priestley-Pauli and energy methods.

6.1. Frame Design Parameters By Young's Method

Young's method has been used to idealize the

cover curve and determine the design parameters of the frame. In this method, a two-line curve is obtained by continuing the elastic range and obtaining the yield base shear and then connecting it to the maximum base shear so that the stored strain energy does not change. In this case, the area under the cover diagram is assumed to be equal to the area under the bilinear curve diagram (Uang, 1991), Figures (11) and (12).

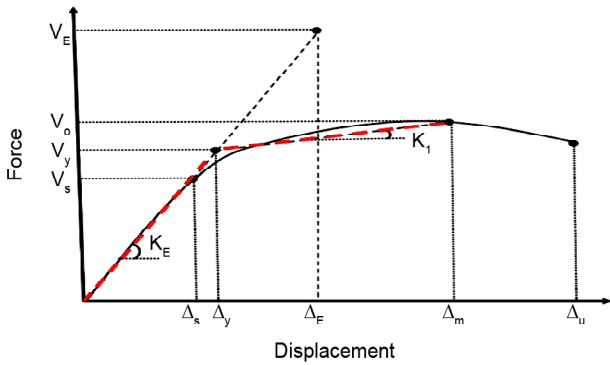


Figure 11. Bilinearization of pushover diagram by Young's method.

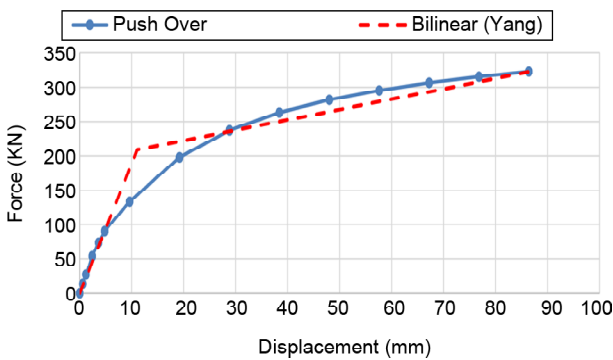


Figure 12. Bilinearization of SRF frame pushover diagram by Young's method.

The research of Berkeley University researchers has shown that the components of the behavior coefficient consist of four parameters, which are mentioned in Equation (3) (Rojahn et al., 1995).

$$R = R_S R_\mu R_R R_\zeta \tag{3}$$

where R_S : over strength coefficient, R_μ : coefficient due to ductility, R_R : uncertain coefficient and R_ζ : damping coefficient of the system, the calculation of each of which is described in detail by Rojahn et al. (1995), and the results are presented for the desired frame in Table (1).

Table 1. Design parameters of EBFs by Young's method.

Parameter	Value	Parameter	Value
V_S (kN)	90.56	V_E (kN)	893.87
δ_S (mm)	4.80	δ_E (mm)	47.37
K_E	18.86	R_S	2.30
V_0 (kN)	322.57	R_μ	4.27
δ_m (mm)	86.40	R_R	1.00
V_y (kN)	208.91	R_ζ	1.00
δ_y (mm)	11.07	μ	7.80
K_1	1.50	R	9.87

6.2. Frame Design Parameters by Priestley-Pauli Method

At this stage, the Priestly-Pauli method was used for idealization and Bilinearization of the push curve, and the Chopra method was used to calculate the design parameters. In this method, the first part of the two-line curve intersects the cover curve at , and then continues up to , and the slope of the second part of the two-line graph is zero. In other words, the second part of the curve is horizontal and is equal to In this case, the area under the push-over diagram is assumed to be equal to the area under the bilinear curve diagram, Figures (13) and (14).

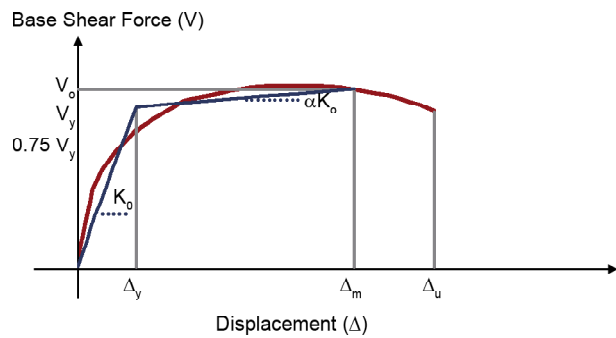


Figure 13. Bilinearization of SRF frame pushover diagram by Young's method.

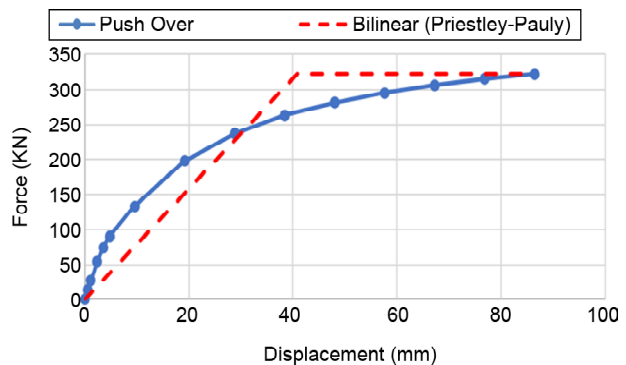


Figure 14. Bilinearization of SRF frame pushover diagram by Priestley-Pauli method.

Table 2. Design parameters of SRF frame by Priestley-Pauli method.

Parameter	Value	Parameter	Value
V_S (KN)	90.56	V_E (KN)	896.35
δ_s (mm)	4.80	δ_E (mm)	47.50
K_0	7.90	R_S	3.56
V_0 (KN)	322.57	R_μ	2.77
δ_m (mm)	86.40	R_R	1.00
V_y (KN)	322.57	R_c	1.00
δ_y (mm)	40.78	μ	2.11
$\delta_{0.75V_y}$	30.58	R	9.89

In Table (2), the design parameters of the frame are presented by the Priestley-Pauli method. It is quantitatively compared the result of validation curve with each other.

6.3. Frame Design Parameters by Energy Method

The energy method is based on the assumption that the maximum kinetic energy caused by a strong earthquake is equal to the maximum energy that a structure can absorb. The energy balance equation in a structure is as follows.

$$E_{ku} = W_0 + D_u - E_{2u} \quad (4)$$

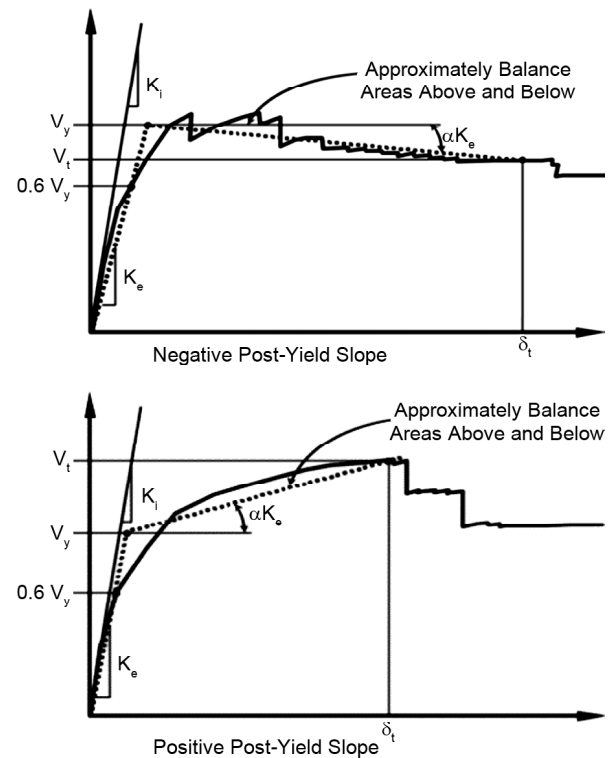
In the above relationship, E_{ku} , the maximum kinetic energy that can be absorbed and dissipate in the structure, W_0 , is the energy stored in the structure during the elastic deformation stage, D_u , the energy stored during the inelastic deformations of the structure and the E_{2u} , work done by the vertical forces in the entire structure deformation process.

If the response spectrum of the ground acceleration in the design earthquake is called S , and the resulting kinetic energy is called E_k , it can usually be related to the most intense design earthquake by applying a coefficient m .

According to this issue, the kinetic energy caused by this destructive earthquake (E_{ku}), which is determined using the maximum pseudo-velocity estimated from the ms spectrum, and is related to by Equation (5); As a result, according to the principle of energy balance, it is necessary to maintain Equation (6).

$$E_{ku} = m^2 E_k \quad (5)$$

$$E_{km} = E_{ku} \quad (6)$$


Figure 15. Bilinearization of pushover curve by FEMA356 method.

There are complex analytical methods for solving the above equation and extracting the coefficient of behavior from it. Due to the complexity and time-consuming nature of these methods, the explanation of these methods and their details are avoided here. In this section, the proposed FEMA-P450 method is used for idealization and determination of design parameters. In Figure (15), the general response of the structure considering the positive and negative slope after yielding, and the method used to bi-linearize the force-displacement curve fit are shown.

The ideal bilinear curve is drawn using an iterative graphical method and by balancing the area above and below the curve. The initial stiffness (or the slope of the first part of the bilinear curve) is determined using the basic shear force equal to 60% of the ideal yield strength of the structure. The second part of the ideal bilinear curve (slope after yielding) is determined using the line that passes through the true curve and the target displacement point (δ_t). As mentioned before, in this study based on references (Uang, 1991; Gad et al., 1999; Kim & Choi, 2005; Maheri & Akbari, 2003; Park, 1989), it is assumed that the displacement of the target is the maximum drift

of the structure; Unless there is a significant drop (20%) in the resistance of the structure before that. In which case that point will be the target displacement change. Also, the regulation specifies that the effective yield strength should not be considered greater than the maximum base shear force at any point of the true curve. In Table (3) and Figure (16), the results and ideal two-line graph are presented.

Table 3. Main values of bilinear curve of SRF frame by energy method in positive and negative directions.

Parameter	Value (mm)	Parameter	Value (KN)
Δ_s^+	3.52	V_s^+	0.75
Δ_s^-	-3.59	V_s^-	-0.77
Δ_y^+	14.83	V_y^+	2.30
Δ_y^-	-14.74	V_y^-	-2.31
Δ_E^+	52.60	V_E^+	8.17
Δ_E^-	-52.43	V_E^-	-8.21

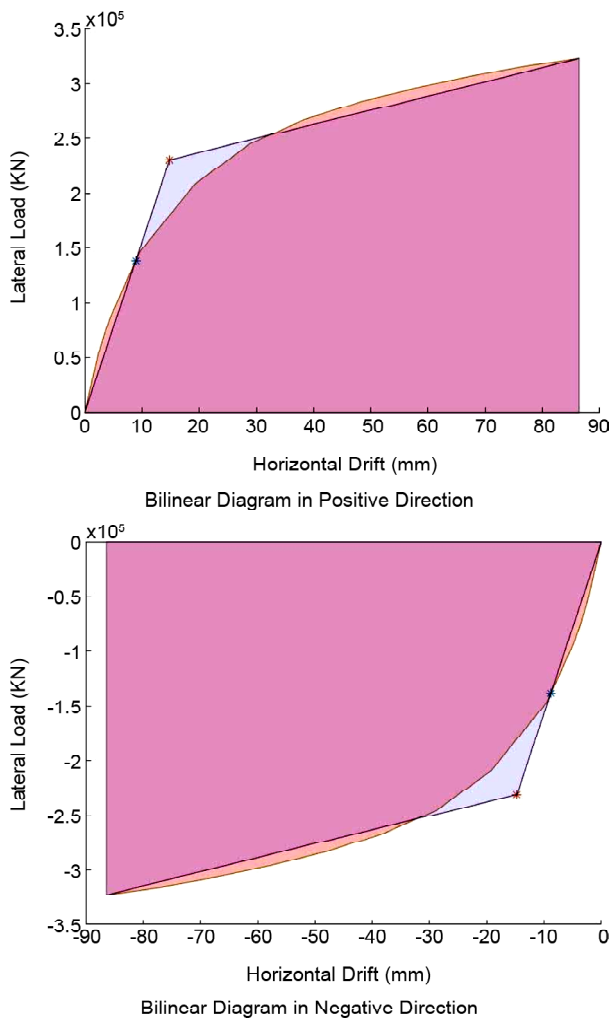


Figure 16. Ideal bilinear diagram in the positive direction of the SRF frame.

6.3.1. Calculation of Behavior and Ductility Coefficients

In this method, the behavior coefficient is expressed by its two main components as follows:

$$R = R_d \times \Omega_0 \tag{7}$$

In which, R_d is ductility coefficient and Ω_0 is over strength coefficient of the structure. Each of these components is determined by using Figure (17), (in which the real and elastic performance of the structure is shown along with the bilinear ideal curve), and equations (8) and (9).

$$R_d = \frac{V_e}{V_y} \tag{8}$$

$$\Omega_0 = \frac{V_y}{V_s} \tag{9}$$

V_e : Elastic response strength of the structure

V_y : Ideal yield strength

V_s : Strength of the first appreciable yield point.

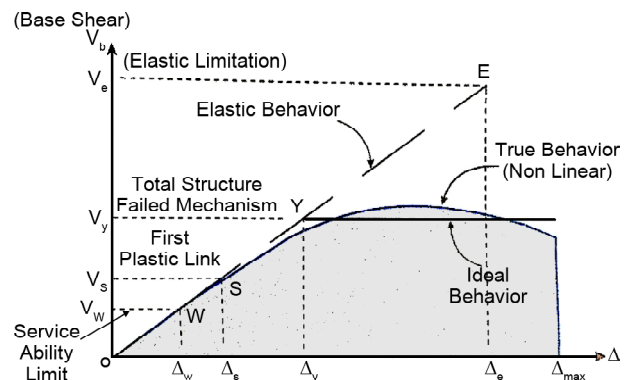


Figure 17. Actual and ideal response curves of the structure.

According to the above relationship, the details of the calculation of behavior coefficient and ductility coefficients are included in Tables (4) and (5), respectively.

In the Figure (18), the real behavior and the elastic response of the structure are presented side by side.

Table 4. Maximum displacement and ductility coefficients.

Move Direction	Δ_{max} (mm)	Δ_y (mm)	$\mu_s = \Delta_{max} / \Delta_y$	Average
Positive	48	14.83	3.2366	3.2465
Negative	-48	-14.74	3.2564	

Table 5. Details of calculation of positive and negative behavior coefficient.

Move Direction	V_s (KN)	V_y (KN)	V_e (KN)	$R_d = V_e / V_y$	$\Omega_0 = V_y / V_s$	$R = R_d \times \Omega_0$	μ_s
Positive	0.75	2.3	8.17	3.5521	3.667	10.89	10.775
Negative	-0.77	-2.31	-8.21	3.5541	3.0000	10.66	

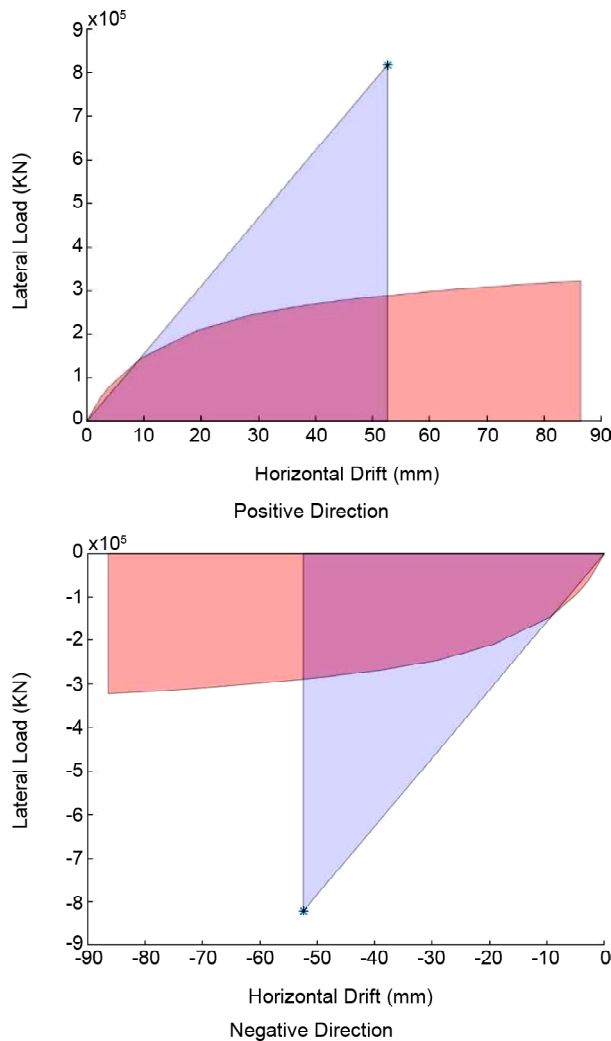


Figure 18. Elastic response compared to the actual behavior of the structure.

7. Comparison of the Methods of Calculating the Coefficient of Behavior

According to the contents stated in a numerical comparison, the coefficient of behavior in Young's method is 9.87, in Priestley-Pauli method it is 9.89 and in energy method it is 10.775. Since the Yang and Priestley-Pauli methods are among the American methods and the energy method is among the methods of European researchers, there is a slight difference in the determination of this parameter in the American methods. The difference of 9% in the behavior factor calculated by the energy method with the Yang and Priestley-Pauli methods is also due to different assumptions

and parameters influencing the calculation of this factor. For example, in the Priestley-Pauli method, the slope of the second part of the two-line graph is zero (.). In other words, the second part of the curve is horizontal and is equal to V_y or V_0 .

However in the energy method, this value is not zero, and α is equal to the slope of the first part of the ideal two-line graph. Comparing other methods with each other, it seems that the definition of over strength coefficient (R_s) in Yang's method is the same as the definition of increased capacity R_C in Freeman's method. The difference between these two methods is in the nature of the ratio of elastic forces to inelastic forces (R_d), the reduction factor due to ductility is (R_μ). Accepting some approximation, Freeman calculates (R_d), for structures with different degrees of freedom, directly from the comparison of elastic and inelastic spectra. While in Young's method, the overall ductility coefficient of the structure (μ_s) is calculated and from the relationships that connect (μ_s) to (R_μ), for structures with one degree of freedom. The reduction coefficient due to ductility (R_μ), in structures with multiple degrees of freedom, with Some approximation is calculated. Also, by reviewing all the proposed methods for calculating the coefficient of behavior, it can be seen that the method of Young's ductility coefficient from the American methods and the method of the theory of ductility from the European methods are very similar to each other and both are based on the results of Newmark's research on single degree of freedom systems are stable. As mentioned, the energy method is a theoretical method and its practical application is difficult.

8. Conclusion

In this article, the design parameters of the EBFs equipped with SRF were compared with Young, Priestley-Pauli and energy methods. In order to achieve the desired results, the design of the steel frame has been done by ETABS software, and modeling and analysis by ABAQUS software.

The results of this study showed that the design parameters and above all the behavior coefficient factor for the target frame, which was calculated from three methods, had a slight difference with each other. In simpler words, the main difference between these three methods is summarized in how to convert the Push curve into an ideal two-line graph. On the other hand, the results indicate that the EBFs is a safe system against lateral loads caused by earthquakes. EBFs equipped with SRF, which exhibit very good behavior during earthquakes due to their high hardness and ductility, require correct and flawless design and implementation for proper operation during earthquakes. The coefficients of behavior and ductility in all three methods showed that SRF fuses show a high potential as an energy dissipating device. Although the fuse was subjected to large inelastic deformations, all other frame members remained fully elastic.

In other words, the replaceable fuse, by accepting a significant shear deformation, enters the non-linear area and with its damage, other parts of the structure remain immune. The biggest advantage of this fuse is its simple replace ability after significant energy consumption. The fact that this fuse can be easily replaced after large earthquakes with a small cost and in the shortest time, and this factor reduces the time of temporary settlement after the earthquake and the structure quickly returns to the state of the settlement frame, a significant advantage for This system is considered.

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