Seismic Behavior of Steel Structure with Buckling-Restrained Braces (BRB)

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ABSTRACT

One of the main purposes of designing buckling-restrained braces is that the entire lateral load is wasted by braces and the entire gravitational load is moved to the foundation by beams, and the columns can be moved to the foundation. In other words, braces are designed for lateral load. It should be noted in the implementation of the structure that the implementation of various parts of the structure must be such that the buckling-restrained braces would not bear the gravitational load. Also, this type of brace was investigated under impact loading, and the design goals of designing method (direct motion) are controlled under impact loading. The results of dynamic analysis are shown as relocation charts of floors and switching between floors, and the results are compared.

KEYWORDS: Buckling-restrained braced frame (BRBF), Energy-dissipating, ABAQUS, SAP2000, Impact load.

INTRODUCTION

Steel moment frames are subjected to large lateral displacements during strong earthquakes. For this reason, special attention is required to limit the movement between floors, so that the potential problems due to non-linear geometrics and brittle or soft failures of beam-to-column connections are dropped and high damage to non-structural components is prevented (FEMA, 2000). In response to the most practical and economic issues, most of the engineers have found a tendency towards the use of steel structures with concentric braces as a lateral load resisting system. Thus, the frequent damage to the steel structures with concentric braces in the past earthquakes, such as those in 1985 in Mexico (Osteraas and Krawinkler, 1985), 1989 in Loma Pryta (Kim and Goel, 1992), 1994 in Northridge (Tremblay et al., 1995; Krawinkler et al., 1996) and 1995 in Hyogo-Knabv (AIJ, 1995; Hisatoku, 1995; Tremblay et al., 1996), have increased the concerns about the ultimate deformation capacity of this class of structure.

Several reasons were presented for the poor performance of bracing structures. For example, braces often have the energy dissipation capacity or limited ductility under cyclic load (Tang and Goel, 1989), and

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most connections are subjected to vulnerable behavior. Hysteresis behavior of the braces is quite complicated and shows asymmetric characteristics of stretch and strain and a great reduction in the resistance, while there is uniform loading at the pressure or intermittent load in the inelastic range. This complex behavior can lead to large differences between the distribution of internal forces and predicted deformations using conventional design methods based on very realistic elastic behavior models and non-linear analysis processes (Jain and Goel, 1979; Khatib and Mahin, 1987). Consequences of such behavior differences are twofold:

The selected braces for some floors are often much stronger than the required extent, while the braces of other floors have capacities very close to the design goals, and the distribution of design forces in the columns and beams is often different from the expected rate of real earthquakes. These differences lead to earthquake damage on several poor floors. Some damages occur a little more than the ductility capacities of usual braces and their connections. It should be noted that the lateral buckling of conventional braces may cause great damage to the adjacent non-structural components. Seismic design requirements for bracing structures have considerably changed during the 1990s and the concept of concentric bracing structures has been proposed (AISC, 1997; ICBO, 1997). Considerable studies have begun to increase the performance of bracing structures by providing a new structure or the use of special braces including (Ku, 1999) the braces using the flow of metal (Kamura et al., 2000; Ohi et al., 2001), high-performance materials (Aiken et al., 1992), friction and viscous damper (Yashino and Karino, 1971). Therefore, a systematic review of the general characteristics of the seismic performance of concentric bracing structures designed to current standards is required. Some of the main structures are shown in Figure 1.

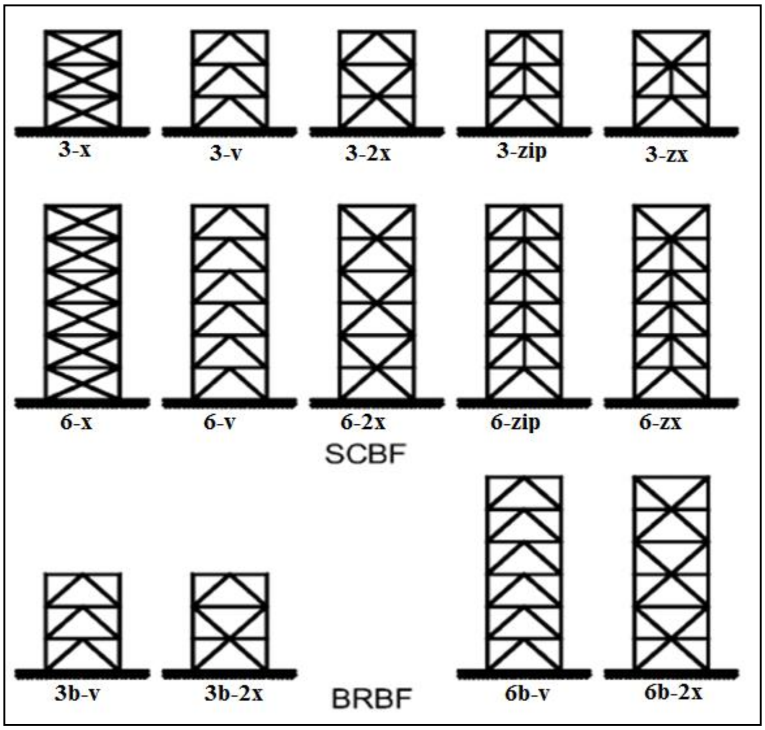

Figure (1): Some structural formations under study (AISC, 1997)

Some studies have been conducted on BRB by Yashino and Karino (1971).They performed some alternative experiments on two samples called "shear wall or bracer", which consisted of a flat metal plate covered with reinforced concrete. Wakabayashi et al. (Xie, 2005) made a bracing system consisting of a flat metal plate with a layer of reinforced concrete by reducing the friction between them.

Another experiment was conducted by Kimureat on metal bracer covered with mortar inside the metal tube. Tube filled with mortar showed its effectiveness against core buckling. In a subsequent study, 4 samples with real sizes were tested under seismic load. It was concluded that if the ratio between the outer sheath of elastic buckling strength and yield strength of the bracer core is larger than 1/9, no buckling would occur at the bracer core and the prototype shows good behavior of the hysteresis cycle. Iwata et al. (2000) investigated the periodic performance of some anti-buckling braces available in Japan. Three large braces were tested at Berkely University to help design and build a structure with BRB.

Black et al. (2002) performed a different analysis on large earthquakes and the elastic torsional buckling of the core to investigate the stability of the inner core. Chen found that the use of low-resistance metal makes a low flexural deformation at the bracer leading to greater flexibility. In a study conducted by Uang and Nakashima, the advantage of using BRB in dual system for reducing permanent deformation was investigated (Uang and Nakashima, 2004).

Min, Tsai and Hsiao (2005) studied the effect of friction reducers on the periodic response of braces. Sabelli increased the seismic absorption of frames by coating the bracing system.

Kim and Seo (2004) and Kim and Choi (2004) presented a process for BRBF seismic design based on energy dissipation and a direct displacement design process. Their studies aimed to investigate the design of steel structures with buckling-restrained braces as well as to investigate the behavior of the braces. The behavior of these braces was investigated under impact load, while the design goals of the designing method (direct motion) under impact load were controlled. Results of dynamic analysis are shown as the relocation charts of floors and the switching between floors, and the results are compared. To ensure modeling and determining of error rate, modeling was repeated with SAP2000 software, which is explained below.

MODELING AND ANALYSIS

To check the accuracy of modeling and compare the results with the goals of the design, a dynamic analysis was performed using finite element software of ABAQUS (Sommerville et al., 1997). A steel structure frame with three floors and a mouth buckled using buckling-restrained braces went through a dynamic analysis.

Since the design of this type of structure; namely steel frames with buckling-restrained braces, is based on the principle that the beams and columns remain perfectly elastic due to the earthquake and the seismic load is wasted by braces, thus the structure design is limited only to braces design, and the beams and columns designed for gravitational load and component load of braces are identical in all samples. The only difference between the four different models is the size of braces and other structural elements, and the characteristics are the same in all cases.

Structural Modeling

Frame Elements

In the structural modeling, only one frame was modeled at the software. The frame has been selected for analysis, as shown in Figure (2). It has one span and 3 floors. Each floor has the weight of 100 kN. Sections of the beams and columns can be seen in Table 1. The selection of beam and column members is according to the Korean Standard (KS) (Rahai et al., 2008).

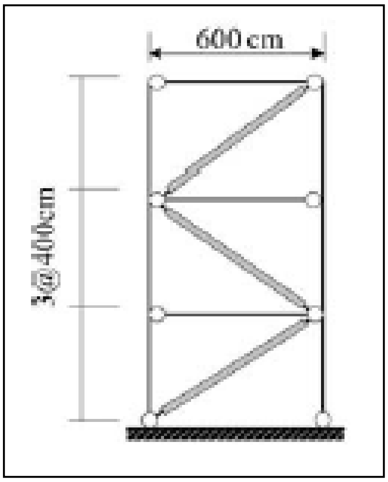

Figure (2): 3-floor frame and a mouth

Table 1. Sections of beams and columns (mm)

Storey	Columns	Beams
1-2	H 250 x 250 x 9 x 14	H 400 x 200 x 8 x 13
3	H 200 x 200 x 8 x 12	

Cross-section of buckling resistant braces, considering the target displacements: 1% (case A), 1.5% (case B), 2% (case C) and 2.5% (case D) height of the roof level, were determined as shown in Table 2.

Table 2. Cross-section of braces (cm^2)

Storey	Case A	Case B	Case C	Case D
1	2.18	1.09	0.97	0.89
2	1.82	0.91	0.81	0.74
3	1.09	0.54	0.49	0.44

As seen in Table 2, with increasing displacement of the target, the cross-section of braces is reduced. According to Figure (3), it can be seen that with increasing target displacement (U_m), S_a value is reduced. Thus, reducing S_a according to equation (1) has a direct relationship with base shear (F_y). In other words, reducing the S_a value reduces the base shear value.

$$F_y = M \times S_a \quad (1)$$

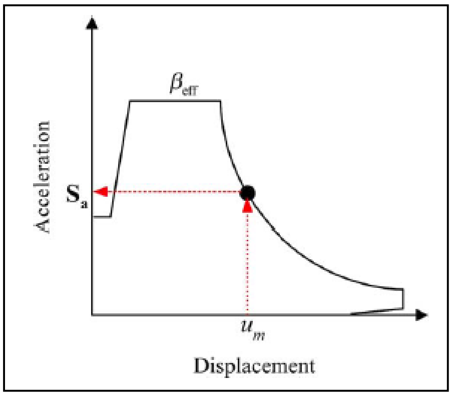

Figure (3): Response spectrum of displacement-acceleration (ADRS)

Thus, by reducing the amount of base shear according to the following equation, the amount of brace cross-section will decrease.

$$A = \frac{F_y}{\cos\theta\sigma_y}. \quad (2)$$

This problem can be explained as the same about the reduction of brace cross-section along the structure height. If we assume that the target displacement of structure is equal to the sum of the target structure displacement of the floors level, thus, the target displacement of the floors level increases by increasing the height of the floors level, resulting in the reduced cross-section of brace for that floor.

As we shall see, the reduced cross-section of brace in higher floors will have some advantages, among which the following points may be mentioned:

1. Preventing the formation of poor floor.
2. Smoothing the displacement between floors.
3. Uniform energy dissipation along the structural height.

However, all the issues above are among the goals of metal frames design with buckling-restrained braces.

Material Characteristics of the Elements

Since the units of N and mm are used for modeling, thus the modulus of elasticity of steel used in beams and columns was considered as E=210000 N/mm^2. Yield stress and failure stress were respectively considered as 240 and 420 N/mm^2. Poisson's ratio of 0.3 was used in the modeling. As previously mentioned, the steel used in buckling-restrained braces is low-strength steel; namely steel with yield stress of 100 MPa.

Creating Elements

Columns

Elements are modeled using Wire in ABAQUS, Figures (4) and (5).

As it can be seen, according to Table 1, the columns of the first and second floor have identical sections and their connections are fixed. These two columns have been created in the same part and are integrated. The total height of this model is about 8000 mm. Since the left and right columns are identical, they have been copied to the other side.

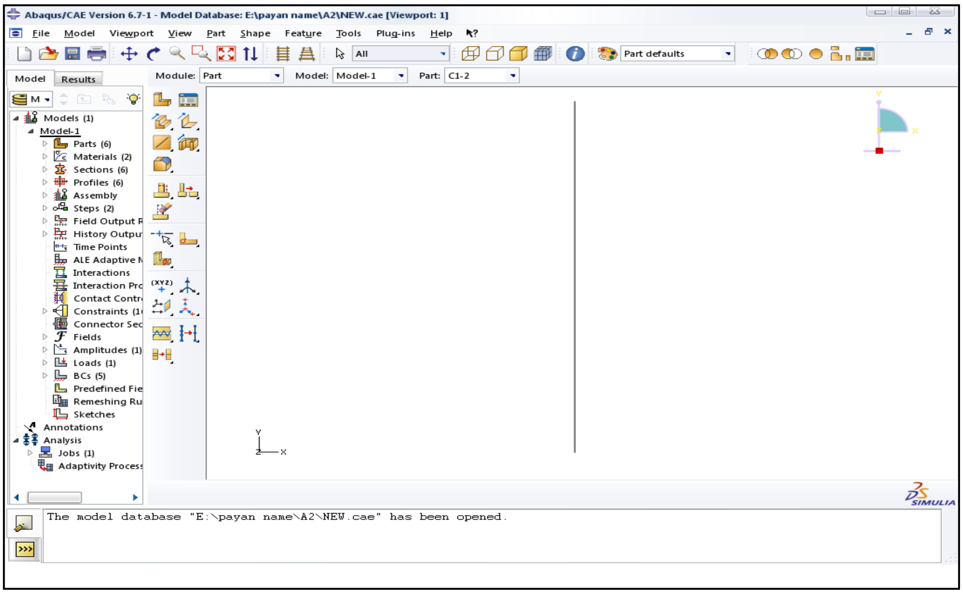

Figure (4): First and second floor columns

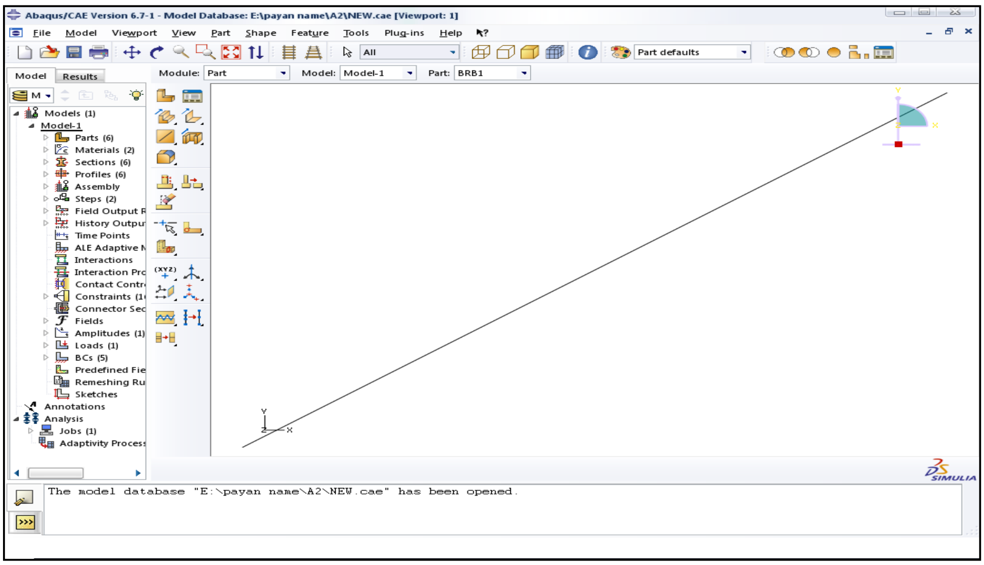

Figure (5): First floor bracing system

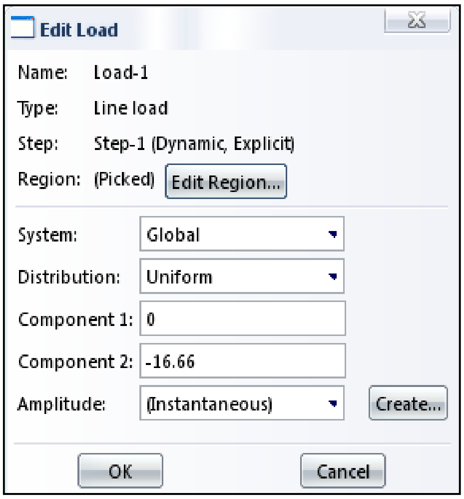

Figure (6): Gravity loading

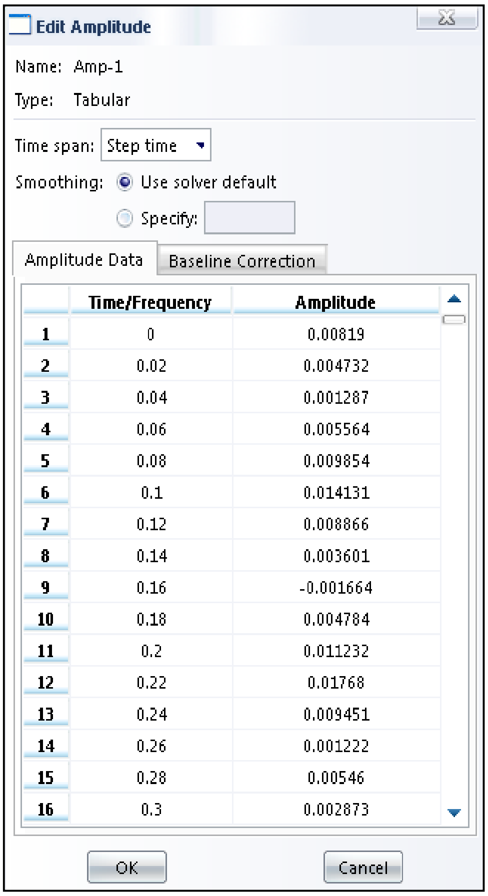

Figure (7): Earthquake records

Bracing Systems

With regard to the properties given in Table 2, the first bracing system has been created and named as BRB1. While the three bracing systems consisted of three different sections and directions, the two other bracing systems, named BRB2 and BRB3, have been created.

Loading

The following steps have been implemented in the modeling:

1. Gravity loading.
2. Lateral loading (dynamical load applied).
3. Amplitude created for dynamic loading.
4. Boundary conditions applied to supports.
5. Boundary conditions applied to braces.

While the weight of each floor is 100,000 N, in order to determine the columns stress, a linear gravity load has been applied to the beams (16.67 N/mm). Lateral load has been applied to the structure as a dynamical load. Then, earthquake loading has been applied with regard to the accelerograms (time *vs.* acceleration), Figures (6) and (7).

According to steel structures, all connections of supports are joint. For this reason, movements are constrained in U2 direction, but rotations and movements are free in UR3 and U1 directions, respectively, Figure (8).

In order to model the buckling resistant braces system, tie has been used in order to let the bracing systems not be buckled under the load applied. As it is clear from Figure 9, movements are free in U1 and U2 directions, but rotation has been constrained in UR3 direction.

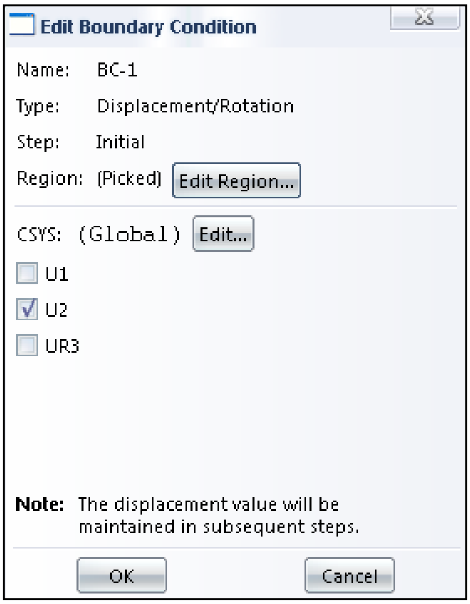

Figure (8): Boundary conditions of supports

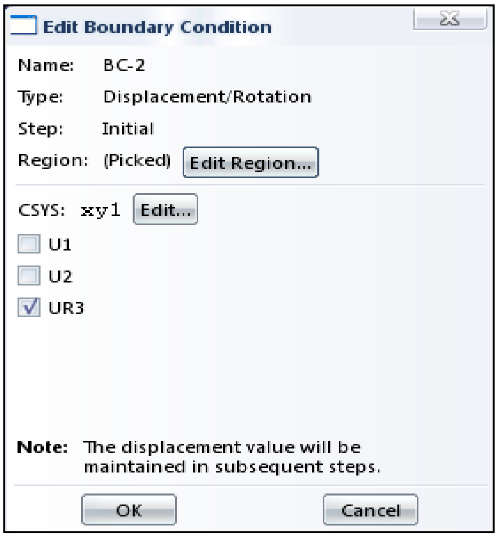

Figure (9): Boundary conditions of bracing system

Meshing

Approximate size of the meshes has been chosen with regard to the size of the elements, Figures (10) and (11).

The overall shape of the structure after meshing is in the form shown in Figure 12.

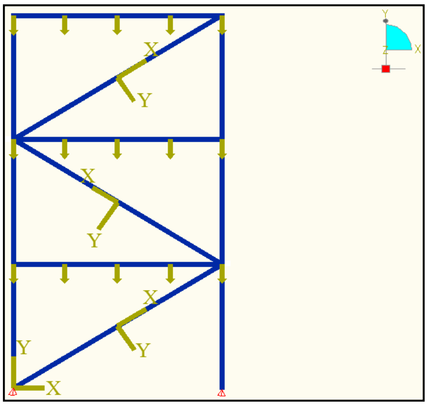

Figure (10): Schematic view of the frame

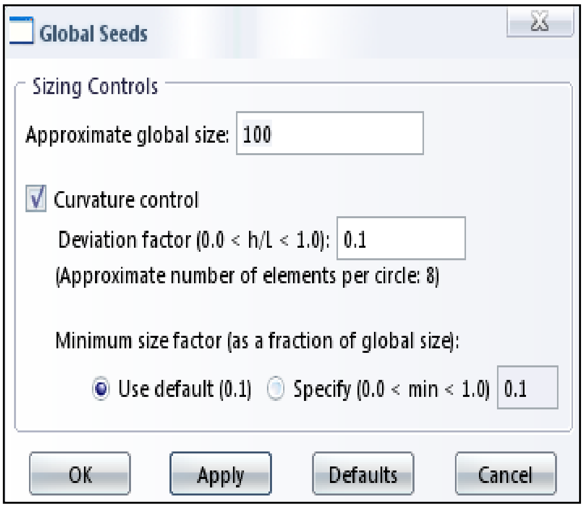

Figure (11): Meshing size

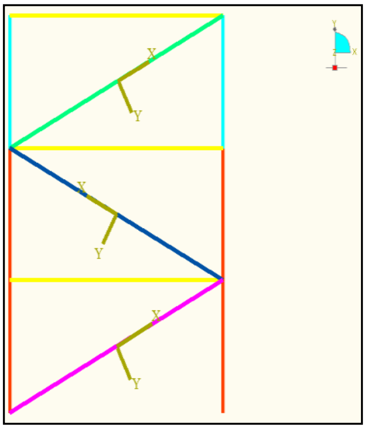

Figure (12): Overall shape of the structure after meshing

Results of Dynamic Analysis and Comparing Them with Design Goals

In this section, the results of the dynamic analysis will be examined as the graphs of the displacement of floors, and a comparative study with the design goals will be conducted.

After the dynamic analysis for each case (A to D), the maximum displacement of floors level can be received as output from ABAQUS software. In this section, for example, the maximum displacement of floors level related to case B designed for a maximum displacement equal to 1.5% will be presented as in Figures (13) and (15).

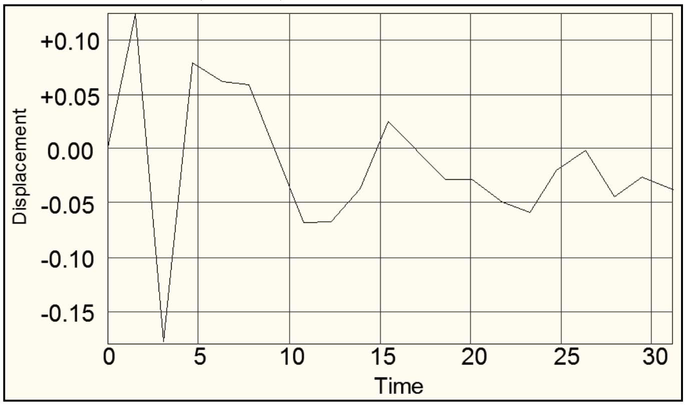

Figure (13): Displacement of the third floor level (m)

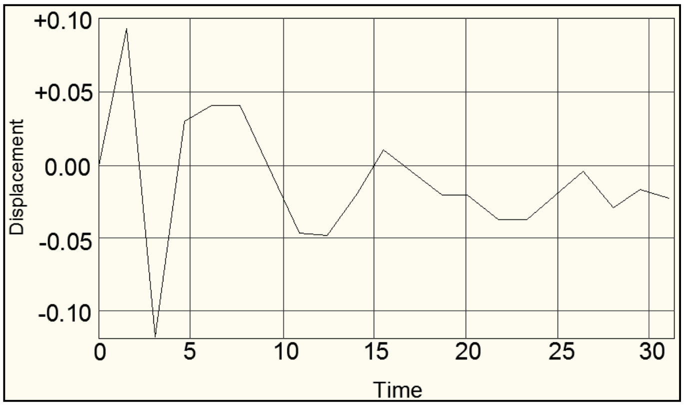

Figure (14): Displacement of the second floor level (m)

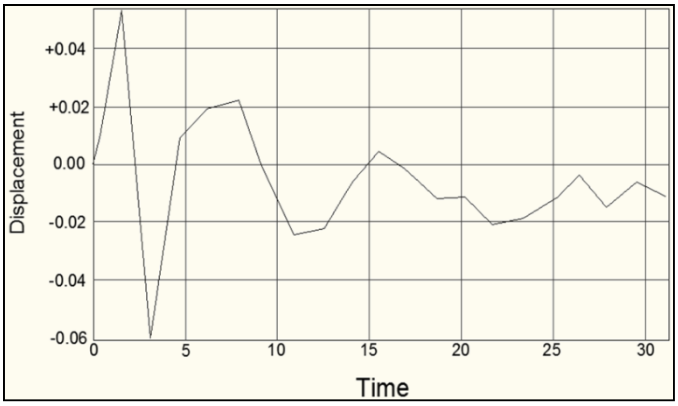

Figure (15): Displacement of the first floor level (m)

Table 3. Maximum displacement of floors level (mm)

CASE A	CASE B	CASE C	CASE D
TD = 1%H	TD= 1.5%H	TD = 2%H	TD= 2.5%H
39.8	60	78	98.1
79.1	116.7	157.4	196.7
117	176.4	236.6	296.7

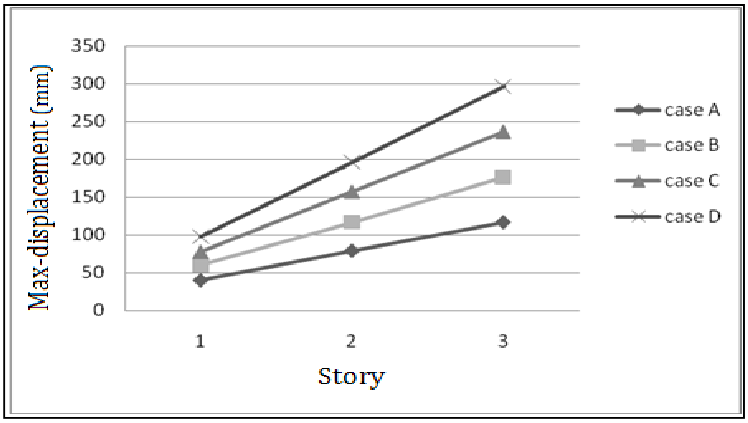

Figure (16): The maximum displacement of floors level

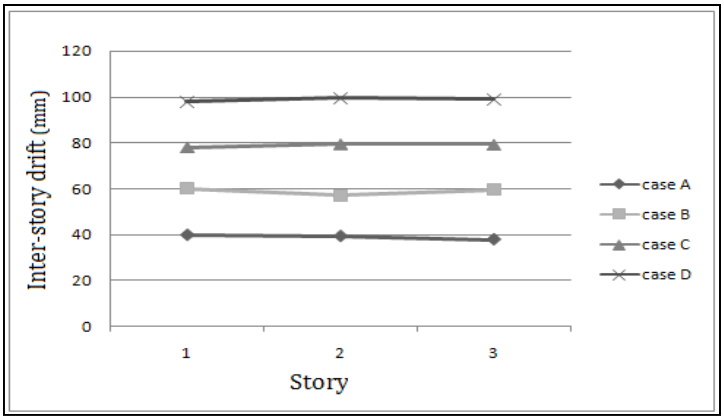

Figure (17): Displacement between floors

Figures (16) and (17) and Tables (3) and (4) show the maximum displacement and the displacement between floors. It can be seen that the maximum displacement of floors and the displacement between floors correspond to the design goals. From Figure 7, it can be seen that the graph of the maximum displacement between floors is close to the line. On the other hand, Figure (17) shows that the displacement between floors is uniform in all floors, showing the lack of damage concentration in floors.

As previously mentioned, one of the fundamental problems in the design of braces is that in some floors the braces capacity is very close to the required capacity, and in some floors the capacity is greater than the required capacity. So, the earthquake condition is different, due to power distribution in floors which is different from power distribution considered in the design; thus, due to the focus of damage on the poor floor, the entire structure will be damaged. In the design method, the direct displacement of the brace cross-section is determined so that the problem is prevented. As the diagram in Figure (16) shows, the uniform displacement between floors is a sign of decentralization of the damage in a particular floor. This means that the braces are designed in such a way that the harm focus does not occur in any of the floors.

Table 4. Displacement values between floors (mm)

CASE A	CASE B	CASE C	CASE D
39.8	60	78	98.1
39.3	57	79.4	99.5
37.9	59.7	79.2	99.1

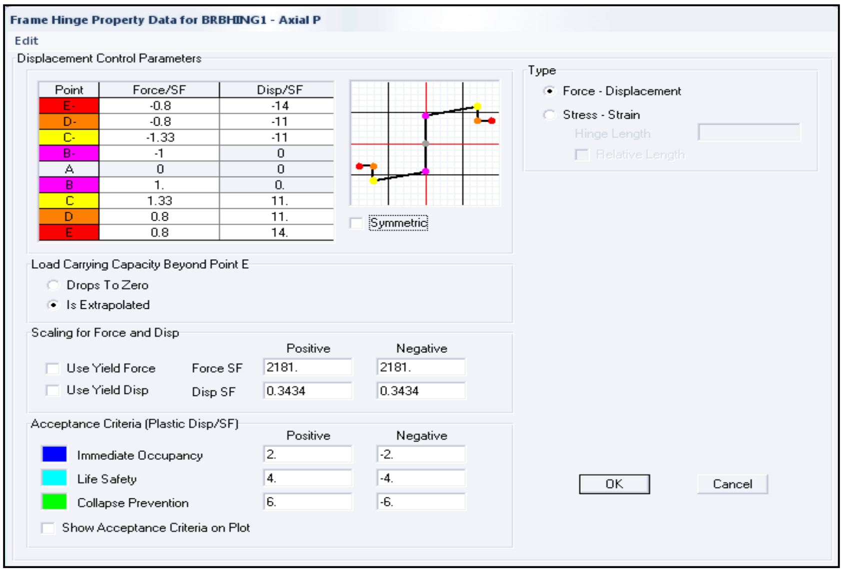

Figure (18): The brace plastic joint of the first floor

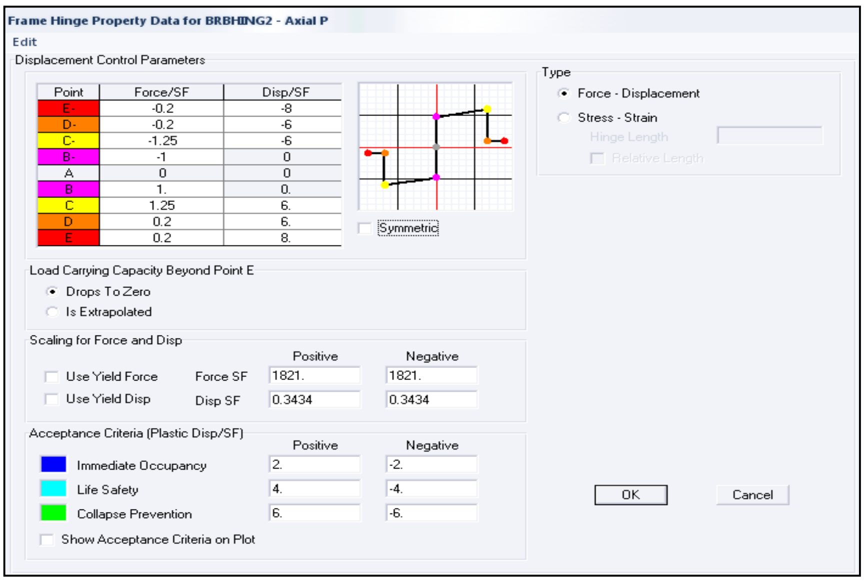

Figure (19): The brace plastic joint of the second floor

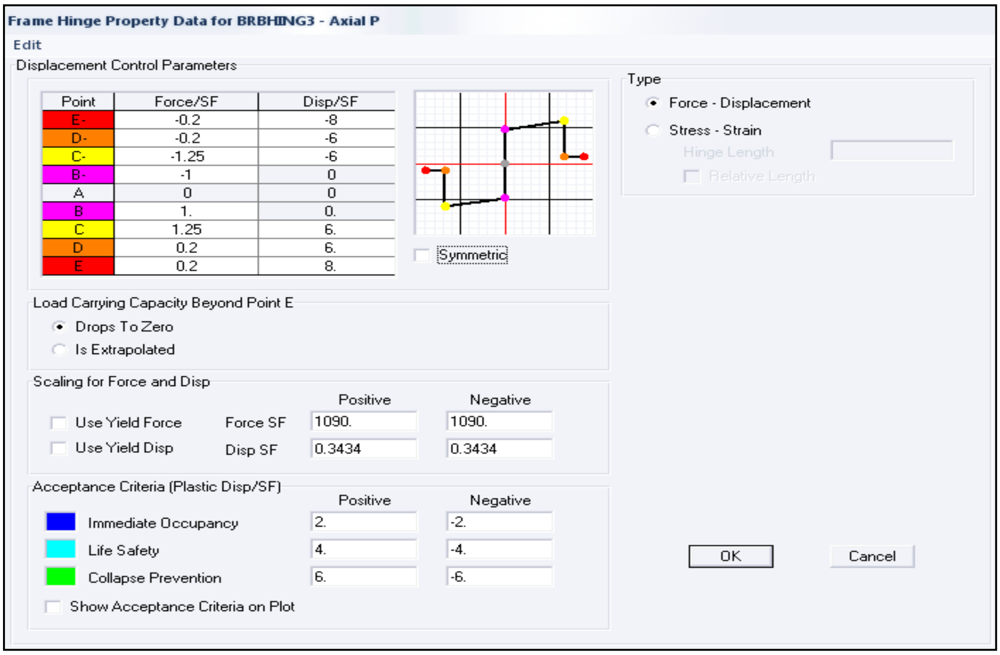

Figure (20): The brace plastic joint of the third floor

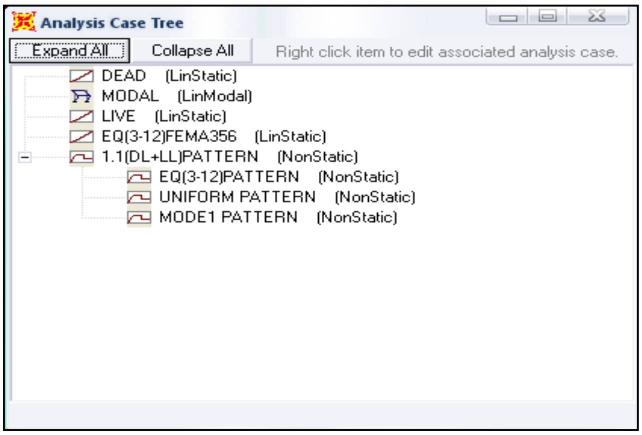

Figure (21): The analysis steps

Control of Results by SAP2000 Software

To ensure the obtained results, modeling was repeated once again by SAP2000 software. Then, a summary of modeling and results analysis is presented. Clearly, modeling the steel frame is easily done in SAP2000 software, but modeling of buckling resistant brace has some tips that should be considered. Buckling resistant brace modeling was carried out using the definition of the plastic joints. But, the remarkable thing here is that the plastic joints of the buckling resistant braces have been defined at the two ends of the brace like the plastic joints of columns. In the buckling resistant brace, the joints can be formed at the two ends like the columns, because the buckling is not given to the brace, and it will not be like the common braces where the plastic joints are defined at the center of the element.

Braces' Plastic Joint

Here is a glimpse into the A modeling.

Characteristics of braces' plastic joint at the ends of the brace were entered according to Figures (18) to (20). As can be seen, the coefficients of load and displacement are the same for both pull and push modes. In the Scaling for force and Disp section, the entered coefficients according to the amount of cross-section were as same as the stretch and pressure modes.

Values of Scaling for force and Disp were entered according to the cross-section of brace and the yield stress of the brace. For example, for case A, the cross-section of brace is 2.18 cm^2 on the first floor and the yield stress of brace is 1,000 kg/cm^2.

Scaling for force and Disp = 2.18 X 1000 = 218 kgf.

SAP2000 Analysis Results and Comparison with Design Goals

With the analysis observed in Figure (21), the maximum displacement made in floors and the performance of braces were obtained. According to Figure (22), it can be seen that the displacement of floors corresponds with the design goals. In this section, modeling of metal frame with buckling-restrained braces was performed using SAP2000 software. The purpose of this modeling is to compare the maximum displacements, especially the maximum displacement of the first floor level corresponding to Case A. This comparison is done due to the difference between the results of ABAQUS and design goals with the results of Drain2dx software. The modeling confirmed that the results of ABAQUS software correspond with the design goals.

One important point according to Figure (23) is that the beams and columns remained at the elastic stage and no plastic joint was made within them, and only the braces entered into the plastic stage. As mentioned earlier, one of the design goals has been achieved.

U3	U2	U1	Step Num	Step Type	Case Type	Output Case	Joint
cm	cm	cm	Unitless	Text	Text	Text	Text
0.000000	0.000000	0.000000		Max	Non Static	Uniform pattern	1
-0.028910	0.000000	4.068352		Max	Non Static	E q(3-12)Pattern	2
-0.046885	0.000000	8.070608		Max	Non Static	E q(3-12)Pattern	3
-0.061162	0.000000	12.016055		Max	Non Static	E q(3-12)Pattern	4

Figure (22): Values of maximum displacement of floors (cm)

Table 5. Comparison of the results for the maximum displacement of floors for case B (cm)

	STOREY 1	STOREY 2	STOREY 3
DRAIN 2DX	6	12	18
TD	6	12	18
ABAQUS	6.01	11.67	17.64

In Table (5), the comparison of the results of the reference paper (modeling in Drain2dx software) with the results of modeling in ABAQUS software and the values of the design goals are presented. This table, for example, is set for case B.

Steel Frame Behavior with Buckling-Restrained Braces under Impact Load

Great force that works in a very short time is called the impact force (Chopra). An important class of dynamic forces is studied under impact loads. A good practical example of this force is the wave from a surface blast of a high building to its adjacent short building. Dynamic response of structures against such forces was worked on by studies between 1950 and 1960. In this study, the steel frame with buckling-restrained braces previously designed is placed under the impact load and its behavior is investigated. The desired steel frame can be given for easy access as shown in Figure (24). Characteristics of the cross-sections related to beams, columns and braces are visible in Tables (1) and (2). Steel frame under study is in case B and has been designed for the target displacement of 1.5% of the structure height. In this section, the structural frame is modeled as two-dimensional and the impact load enters into the third floor roof.

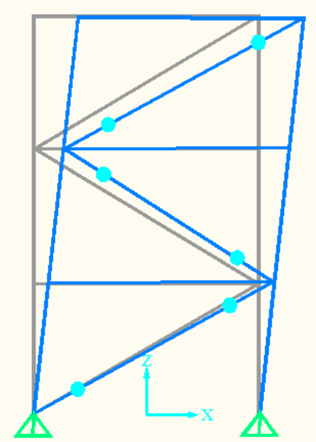

Figure (23): Formation of plastic joints in the structure

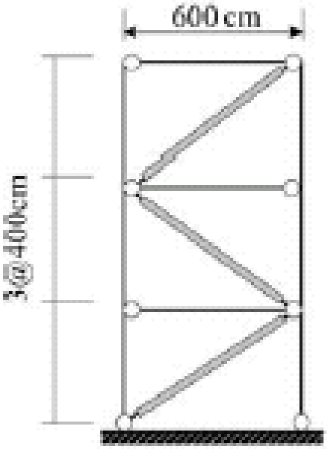

Figure (24): Steel frame with buckling-restrained braces

Impact Load for Steel Frame

In this study, the impact load is applied in two triangular and rectangular forms. Although the two forms of impact load have been used, the values of maximum load time and amount are as such that the area under the time-load curve is fixed for all load cases. The objective of choosing two different forms of impact load is to investigate the effect of the impact load on the structure behavior. The load values in both triangular and rectangular forms are observed in three different effects in Figures (25) and (26) (Chopra).

As Tables (6) and (7) show, in both cases, the load value is fixed, but the time of load effect of the rectangular form is half of that of the triangular load effect. The reason for this difference is that the area under the load-time curve is constant for all cases.

Table 6. Load values of triangular impact

	Case 1	**Case 2**	**Case 3**
T(s)	0.3	0.4	0.5
P(N)	16000	12000	9600

Table 7. Load values of rectangular impact

	Case 4	**Case 5**	**Case 6**
T(s)	0.15	0.2	0.25
P(N)	16000	12000	9600

Study of Design Goals under Impact Load

By applying the impact load according to Tables (6) and (7), the maximum displacement and displacement between floors are visible in Figures (25) to (28).

As Figures (25) and (27) show, the maximum displacement of floors is close to the line in all cases. As the rectangular impact load is applied with greater intensity to the structure, the maximum displacement of a rectangular load is larger than that of a triangular load.

Also, according to Figures (26) and (28), it can be concluded that the displacement between floors for all loads and both forms of load is fairly the same.

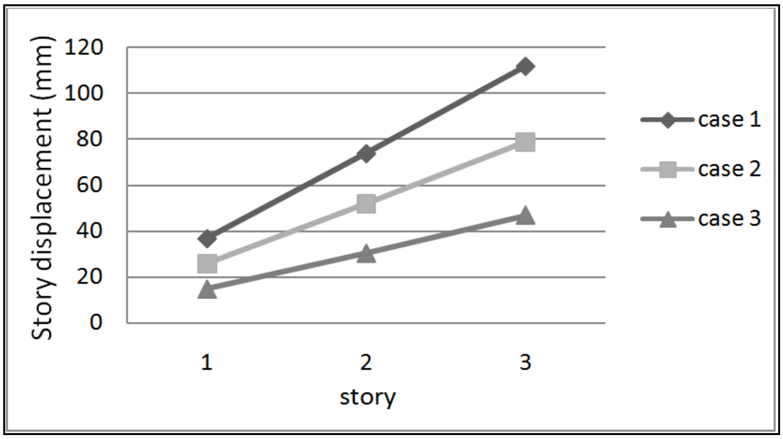

Figure (25): Maximum displacement of floors under triangular impact load

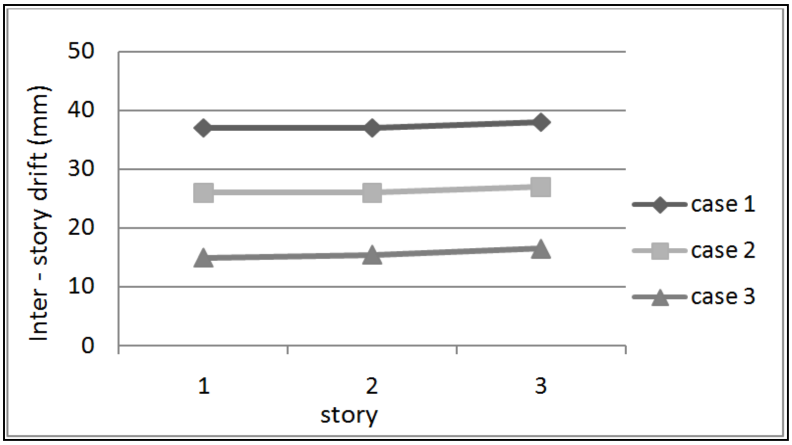

Figure (26): Maximum displacement between floors under triangular impact load

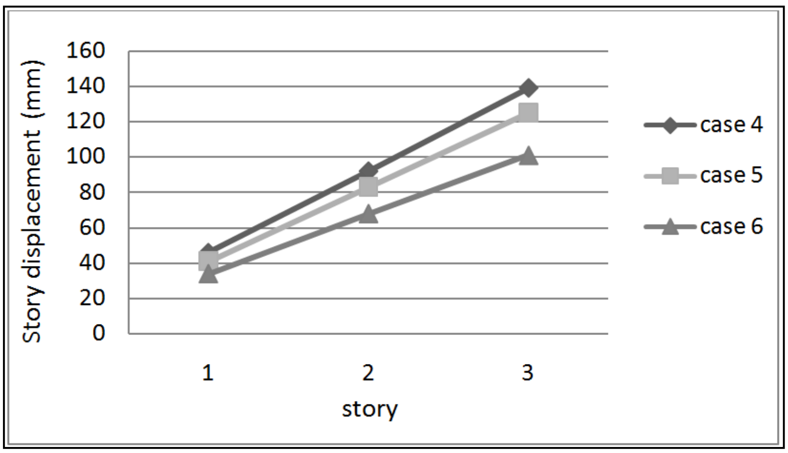

Figure (27): Maximum displacement of floors under rectangular impact load

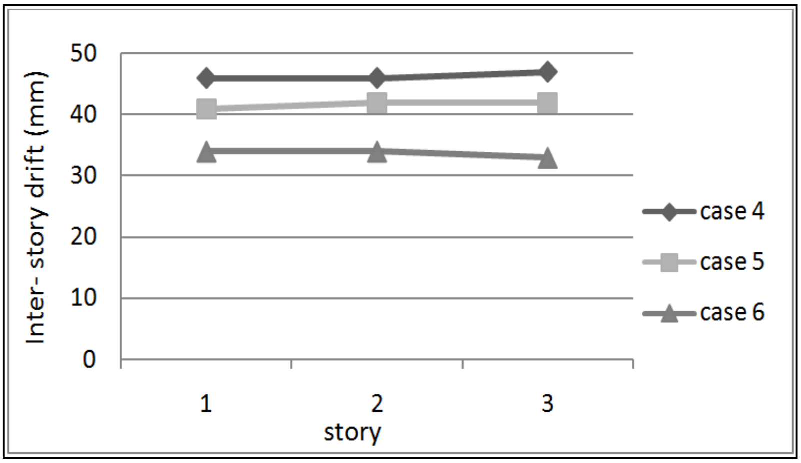

Figure (28): Maximum displacement between floors under rectangular impact load

Thus, it can be concluded that under impact load, as well as under earthquake load, all the design goals of the design method (direct motion) are estimated and the structure under impact load also shows good performance.

CONCLUSIONS

In this study, the seismic design process was investigated for structures with buckling-restrained braces by the joint connection of beam to column. The proposed design process assumes floor disablement, type of shear and the main shape of the mode. The performance of the model structure designed for the target displacement has been investigated under impact load with dynamic analysis to check whether the operational goals are evaluated or not. The following results were obtained:

1. According to the numerical results of the diagrams, the maximum displacement of floors is close to line, the displacement between floors is the same under impact load and the structure under impact load shows optimal performance.
2. Maximum displacement of rectangular load is larger than maximum displacement of triangular load.
3. The displacement between floors, for all load cases and both forms of load, is fairly the same.

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