International Journal of Architectural, Civil and Construction Sciences
ISSN: 2415-1734
Vol:9, No:4, 2015

Seismic Behavior of Steel Structure with Buckling-Restrained Braces

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Abstract**—**One of the main purposes of designing buckling-restrained braces is the fact that the entire lateral load is wasted by the braces, the entire gravitational load is moved to the foundation through the beams, and the columns can be moved to the foundation. In other words, braces are designed for bearing lateral load. In the implementation of the structure, it should be noted that the implementation of various parts of the structure must be conducted in such a way that the buckling-restrained braces would not bear the gravitational load. Moreover, this type of brace has been 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 the relocation charts of the floors and switch between the floors. Finally, the results are compared with each other.

Keywords**—**Buckling-Restrained Braced Frame (BRBF), energy-dissipating, ABAQUS, SAP2000, impact load.

I. Introduction

STEEL moment frames are subjected to large lateral displacements during strong earthquakes. For this reason, special care should be taken to limit movement between the floors so that the potential problems resulting from the nonlinear geometric and brittle or soft failures of beam-to-column connections are dropped, and much of the damage to non-structural components are prevented [1]. In response to the most practical and economic issues, most of the engineers have a tendency towards using steel structures with concentric braces as a lateral load resisting system. Thus, frequent damage to the steel structures with concentric braces in the last earthquakes such as the earthquake in 1985 in Mexico [2], 1989 Loma Pryta [3], 1994 Northridge [4], [5], 1995 Hyogo - Knabv [6]-[8] have increased concerns about the ultimate deformation capacity of this class of structures.

Several reasons were presented for the poor performance of bracing structures. For example, braces often have energy dissipation capacity or limited ductility under cyclic load [9], and most connections are subjected to vulnerable behavior. Hysteresis behavior of the braces is quite complicated and shows asymmetric characteristics of stretch and strain and also 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 [10], [11]. The consequences of such behavioral differences are twofold:

Selected braces for some floors are often much stronger than the required range, 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 have been proposed [12], [13]. Considerable studies have started to increase the performance of bracing structures by providing a new structure or the use of special braces including [14] the braces using the flow of metal [15], [16], high-performance materials [17], friction, and viscous damper [18]. 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 Fig. 1.

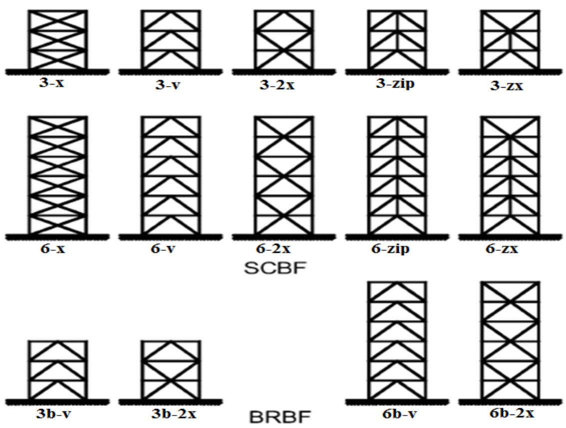

Fig. 1 Some structural formations under study [12]

Some studies were conducted on BRB by Yashino in 1971 [18]. He performed some alternative experiments on two

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samples called "shear wall or bracer", which consisted of a flat metal plate covered with reinforced concrete. Xie, Q. [19] made a bracing system made of a flat metal plate with a layer of reinforced concrete by reducing the friction between them. Kimura et al. [20] tested on braces encased by mortar-infilled steel tubes. The 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 a good behavior of the hysteresis loop. Iwata et al. [21] 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. [22] performed a different analysis on strong 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 a greater flexibility. In [23], the advantage of using BRB in dual system for reducing permanent deformation was investigated.

Min and Tsai studied the effect of friction reducers on the periodic response of braces [24]. Sabelli et al. [25] increased the seismic absorption of frames by coating the bracing system.

Kim et al. [26], [27] presented a process for BRBF seismic design based on energy dissipation and a direct displacement design process. His study aimed to investigate the design of steel structures with buckling-restrained braces and also investigated the behavior of the braces. Then, 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. The results of dynamic analysis are shown as the relocation charts of floors and the switching between floors, and the results are compared with each other. To ensure modeling and determining of error rate, the modeling was repeated with SAP2000 software, which is explained below.

A. Modeling and Analysis

To check the accuracy of the modeling and compare the results with the goals of the design, a dynamic analysis was performed using finite element software of ABAQUS. 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 the 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, 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 with each other is the size of braces and other structural elements, and the characteristics are the same in all cases.

II. 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 Fig. 2; it has one span and 3 floors. Each floor has the weight of 100 KN. The sections of the beams and columns can be seen in Table I, and the selection of beam and column members are according to the Korean Standard (KS) [28].

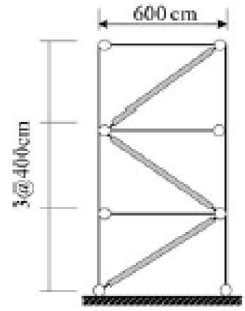

Fig. 2 3-Floor frames and spans

TABLE I
SECTIONS OF BEAMS AND COLUMNS (MM)

Story	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 II.

TABLE II
CROSS-SECTION OF BRACES (CM2)

Story	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

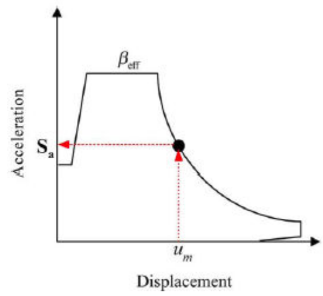

Fig. 3 Response spectrum of displacement- acceleration (ADRS)

As indicated in Table II, with increasing displacement of the target, the cross section of the braces is reduced compared to the braces' cross section. According to Fig. 3, it can be seen that with increasing target displacement (u_m), S_a value is reduced. Thus, reducing S_a, according to (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).

International Journal of Architectural, Civil and Construction Sciences
ISSN: 2415-1734
Vol:9, No:4, 2015

$$F_y = M \times S_a \tag{1}$$

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

$$A = \frac{F_y}{\cos\theta\sigma_y} \tag{2}$$

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

As it will be noted later, the reduced cross-section of brace in higher floors will have some advantages including the following:

- Preventing the formation of poor floor
- Smoothing the displacement between floors
- Uniform energy dissipation along the structural height

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

III. Material Characteristics of the Elements

Since the units of N and mm are used for modeling, the modulus of elasticity of steel used at beams and columns was considered as E=210000 N/mm2. Yield stress and failure stress were considered as 240 and 420 N/mm2, respectively. Poisson's ratio of 0.3 was used in the modeling. As previously mentioned, the steel used in buckling-restrained braces is of low-strength steel, In other words, the steel with yield stress of 100 MPa was used.

A. Creating Elements

Column:

The elements are modeled using Wire in ABAQUS, Figs. 4, 5. As it can be seen, according to Table I, the columns of the first and second floor have identical sections, and both of 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. While the left and right columns are the same and identical, they have been copied to the other side.

Bracing System:

According to the properties given in Table II, Bracing systems have been created and named as BRB1. While the three bracing systems consisted of three different sections and directions, two other bracing systems, called as BRB2 and BRB3, have been created.

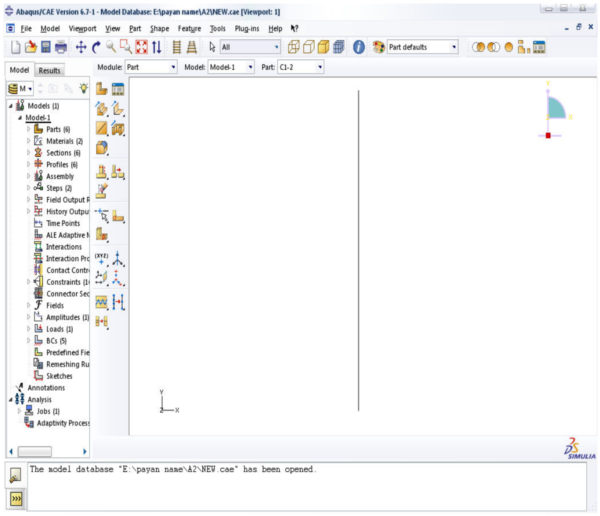

Fig. 4 First and second floor columns

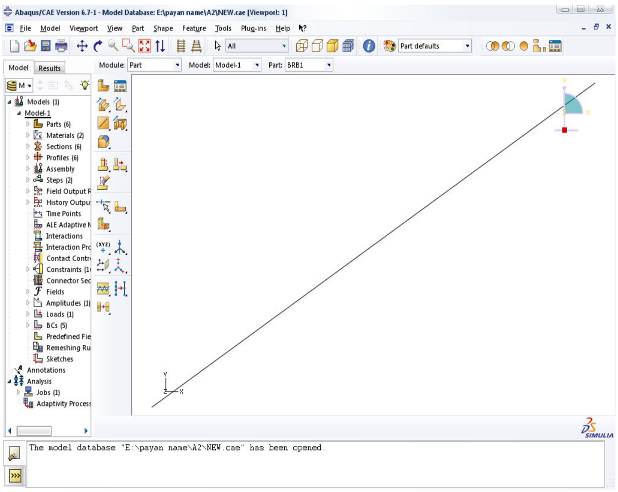

Fig. 5 First floor bracing system

Loading:

The following steps have been implemented as a step in the modeling:

1. Gravity loading
2. Lateral loading (dynamical load applied)
3. Creating amplitude for dynamic loading
4. Applying boundary conditions to the supports
5. Applying boundary conditions to braces

While the weight of each floor is 100,000 N, in order to determine columns' stress, linear gravity load has been applied to the beams (16.67 N/mm). Moreover, lateral load has been applied to the structure as dynamic load. Then, Earthquake loading has been applied according to the accelerograms (time vs. acceleration), Figs. 6, 7.

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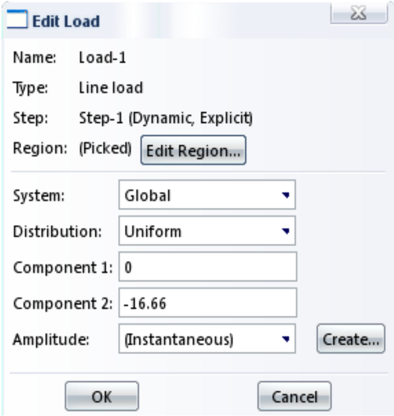
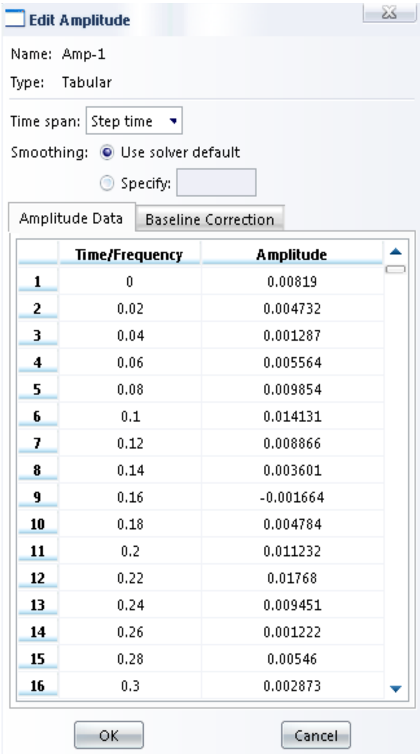


Fig. 6 Gravity loading



	Time/Frequency	Amplitude
1	0	0.00819
2	0.02	0.004732
3	0.04	0.001287
4	0.06	0.005564
5	0.08	0.009854
6	0.1	0.014131
7	0.12	0.008866
8	0.14	0.003601
9	0.16	-0.001664
10	0.18	0.004784
11	0.2	0.011232
12	0.22	0.01768
13	0.24	0.009451
14	0.26	0.001222
15	0.28	0.00546
16	0.3	0.002873

Fig. 7 Earthquake records

Regards to the steel structures, the movements are constrained in U2 direction, but rotations and movements are free in UR3 and U1 directions, respectively, Fig. 8.

In order to model the buckling resistant braces system, they were not allowed to be buckled under the load applied. As it is clear in Fig. 9, the movements are free in U1 and U2 directions, but rotation has been constrained in UR3 direction.

Meshing:

The approximate size of the meshes has been chosen according to the size of the elements (Figs. 10, 11). The overall shape of the structure after meshing is as given Fig. 12.

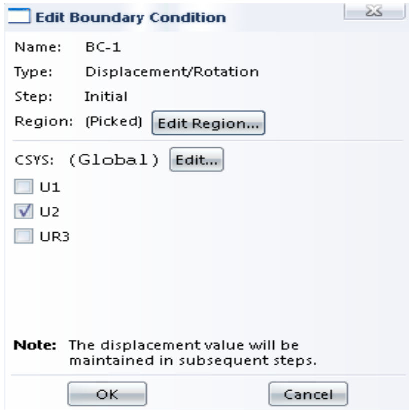


Fig. 8 Boundary conditions of supports

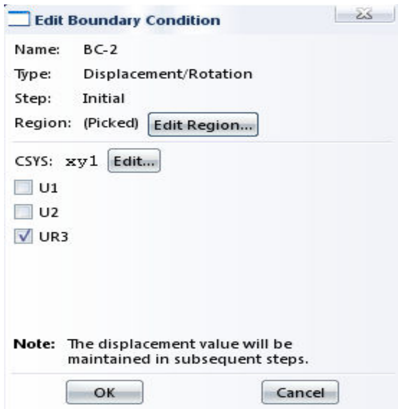


Fig. 9 Boundary conditions of bracing system

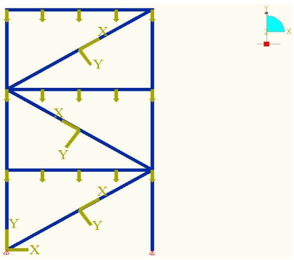


Fig. 10 Schematic view of the frame

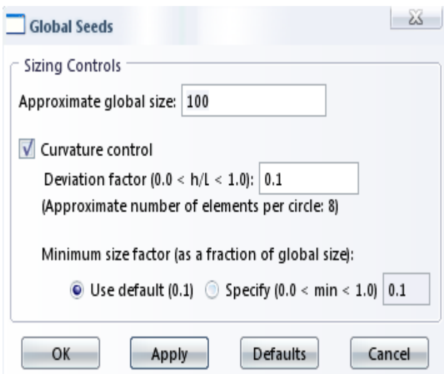


Fig. 11 Meshing size

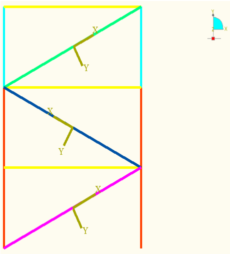


Fig. 12 Overall shape of structure after meshing

IV. The Results of Dynamic Analysis and Comparing It with Design Goals

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

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

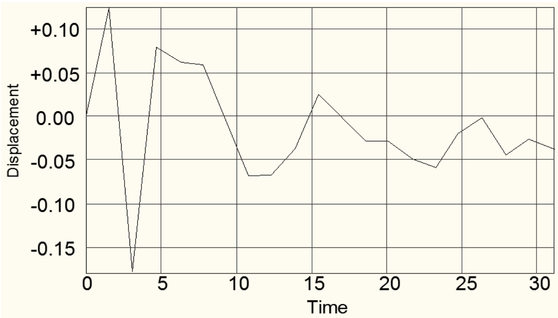


Fig. 13 Displacement of the third floor level (m)

Figs. 16, 17 and Tables III, IV show the maximum displacement and the displacement between the floors. It can be seen that the maximum displacement of floors and the displacement between floors correspond to the design goals. Fig. 7 shows that the graph of the maximum displacement between floors is close to the line. On the other hand, Fig. 17 shows that the displacement between floors is uniform in all floors, showing the lack of damage concentration in floors.

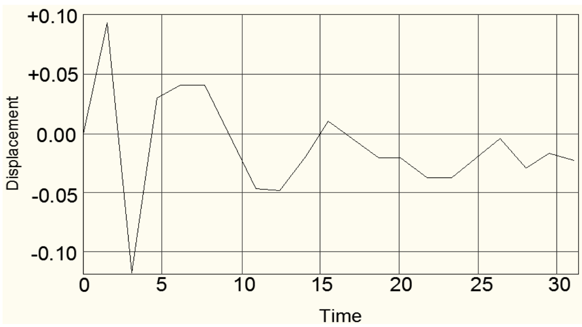


Fig. 14 Displacement of the second floor level (m)

TABLE III
THE MAXIMUM DISPLACEMENT OF FLOORS LEVEL (MM)

CASE A TD = 1%H	CASE B TD= 1.5%H	CASE C TD = 2%H	CASE D 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

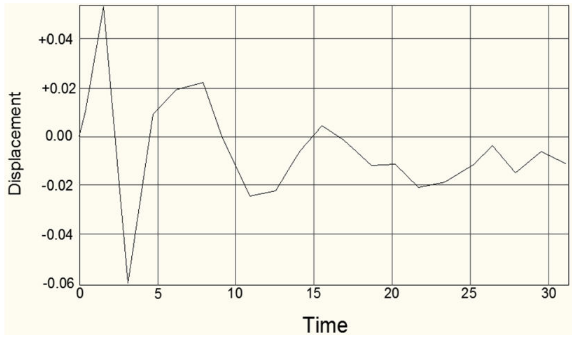


Fig. 15 Displacement of the first floor level (m)

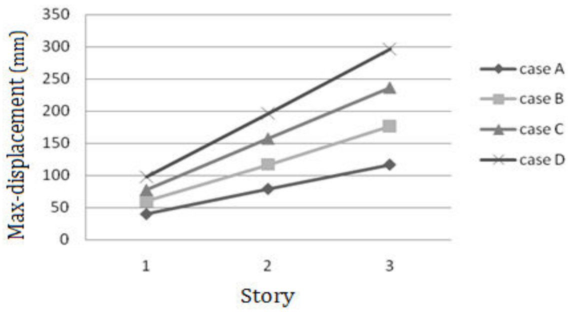


Fig. 16 The maximum displacement of floors level

As previously mentioned, one of the fundamental problems in the design of braces is that in some floors brace capacity is very close to the required capacity, and in some floors, the capacity is greater than the required capacity. Therefore, the earthquake condition is different based on power distribution in the floors with power distribution considered in the design; thus, due to the concentration of damage on the poor floor, the entire structure will be damaged. In the design method, the direct displacement of the brace's cross section is determined so that the problem will be prevented. As Fig. 16 shows, the uniform displacement between the floors is an indication of the decentralization of the damage in a particular floor. It means the braces are designed in such a way that the concentration of damage is not on any of the floors.

TABLE IV
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

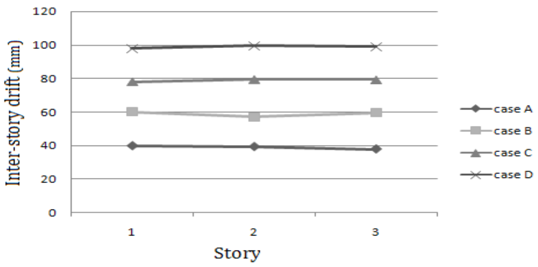


Fig. 17 Displacement between floors

V. 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 the analysis of the results are presented. Clearly, modeling the steel frame is easily done in SAP2000 software, but modeling of buckling resistant brace has some implications that should be considered. Buckling resistant brace modeling was done using the definition of the plastic joints. However, 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 buckling is not given to the brace, and it will not be like the common braces that the plastic joints are defined at the center of the element.

A. Braces' Plastic Joint

In this part, A modeling will be shortly discussed. The characteristics of the braces' plastic joint at the ends of the brace were inserted according to Figs. 18-20. 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 Displacement section, the inserted coefficients, according to the amount of cross section, were the same as the stretch and pressure modes.

The values of Scaling for Force and Displacement were entered according to the cross-section of the brace and yield stress of the brace. For example, for case A, the cross section of brace is 2.18 square centimeters on the first floor and the yield stress of brace is 1,000 kilograms per square centimeter.

Scaling for Force and Disp $= 1000 \times 2.18 = 2180\ kgf$

With the analysis, the stages of which are observed in Fig. 21, the maximum displacement made in floors and the performance of braces was obtained. According to Fig. 22, it can be seen that the displacement of the floors corresponds with the design goals. In this section, the modeling of metal frame with buckling-restrained braces was performed using SAP2000 software. The purpose of this modeling is to compare the maximum of the 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 the results of Abaqus Software that correspond with the design goals.

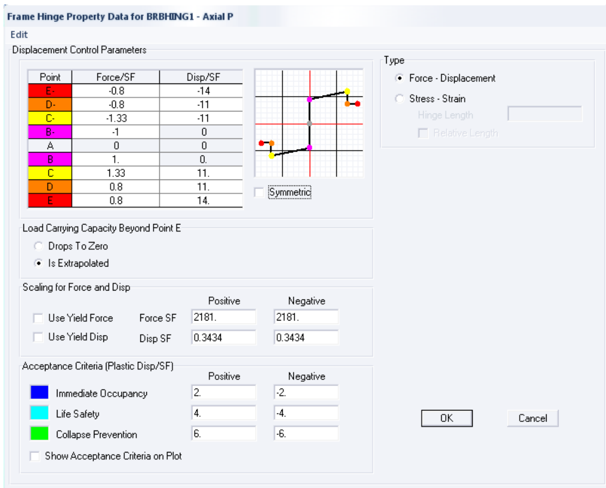


Fig. 18 The brace plastic joint of the first floor

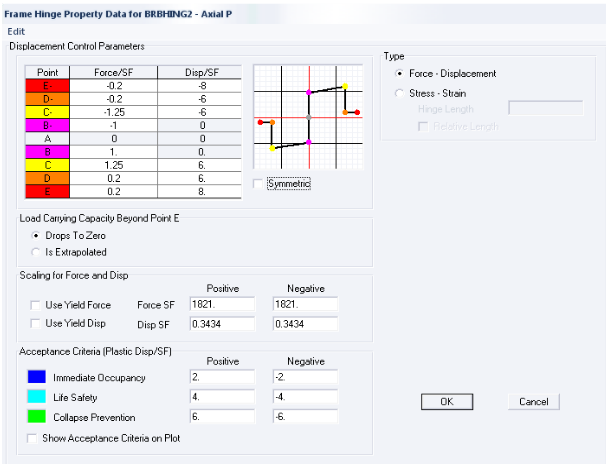


Fig. 19 The brace plastic joint of the second floor

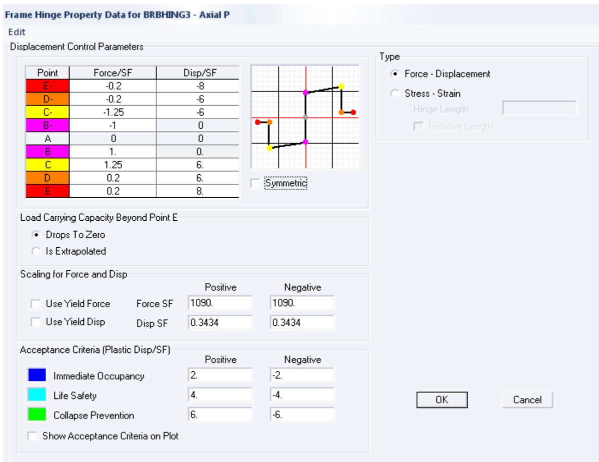


Fig. 20 The brace plastic joint of the third floor

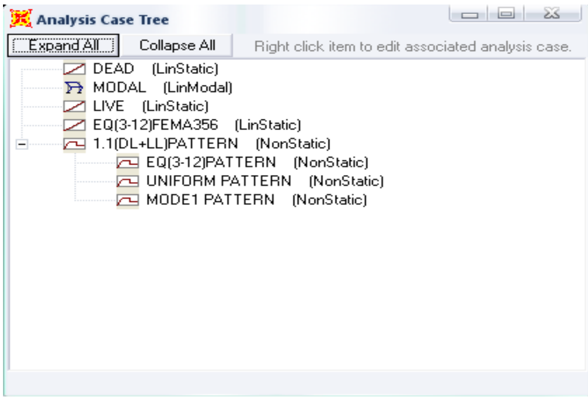
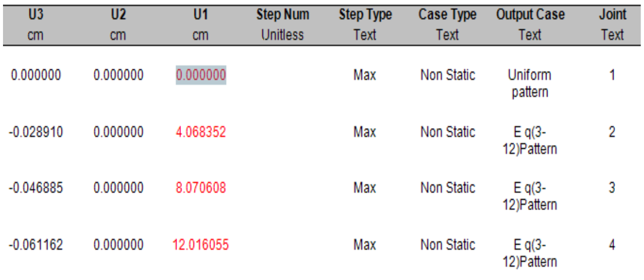


Fig. 21 The analysis steps

One important point according to Fig. 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 the plastic stage. It was mentioned earlier that one of the design goals has been achieved.



U3 cm	U2 cm	U1 cm	Step Num Unitless	Step Type Text	Case Type Text	Output Case Text	Joint 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

Fig. 22 Values of maximum displacement of floors (cm)

In Table V, the comparison of the results of the reference paper (modeling in Drain 2dx 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.

TABLE V
COMPARISON OF THE RESULTS FOR THE MAXIMUM DISPLACEMENT OF FLOORS FOR CASE B (CM)

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

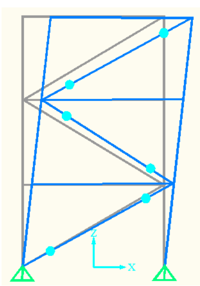


Fig. 23 Formation of Plastic Joints in the Structure

VI. Steel Frame Behavior with Buckling-Restrained Braces under Impact Load

Great force that works in a very short time is called the impact force [29]. An important class of dynamic forces is studied under the 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 dealt with in some studies from 1950 to 1960. In this study, the steel frame with buckling-restrained braces designed in Chapter 2 is placed under the impact load, and its behavior is investigated. The desired steel frame is given in Fig. 2 for easy access. The characteristics of the cross sections related to the beams and columns and braces are visible in Tables I and II. Steel frame under study is case B which 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 the roof of the third floor.

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 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 on the structure behavior. The load values in both triangular and rectangular forms are observed in three different effects in Figs. 24, 25 [30].

As Tables VI and VII show, in both cases, the load value is fixed, but the time of load effect or the rectangular form is half 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 VI
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 VII
LOAD VALUES OF RECTANGULAR IMPACT

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

A. Study of the Design Goals under Impact Load

By applying the impact load according to Tables VI and VII, the maximum displacement and displacement between floors are visible in Figs. 24-27.

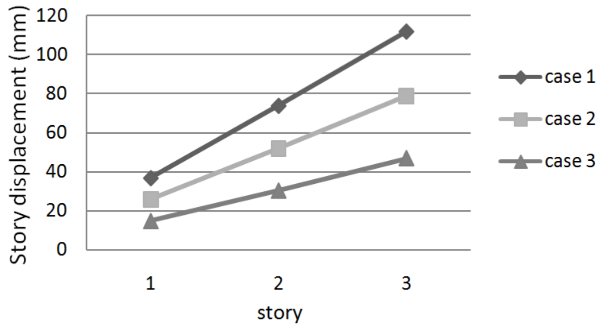


Fig. 24 The maximum displacement of floors under triangular impact load

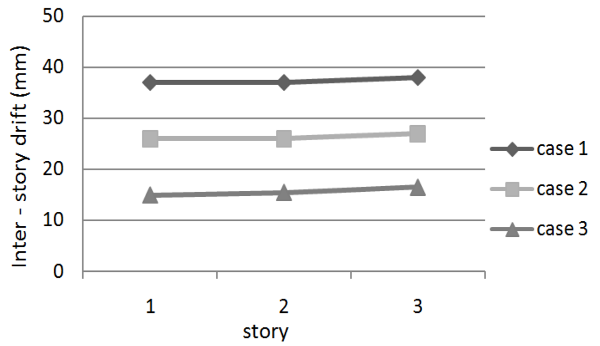


Fig. 25 The maximum displacement between floors under triangular impact load

As Fig. 24 and 26 shows, the maximum displacement of floors is close to the line, and in all cases, as the rectangular is applied with greater intensity to the structure, the maximum displacement of a rectangular load gets larger than the maximum displacement of the triangular load. In addition, according to Figs. 25 and 27, it can be concluded that the displacement between floors for all loads and both forms of the load is fairly the same. Thus, it can be concluded that under the impact load, as well as the earthquake load, all the design goals of the design method (direct motion) are estimated, and the structure under impact load also shows good performance.

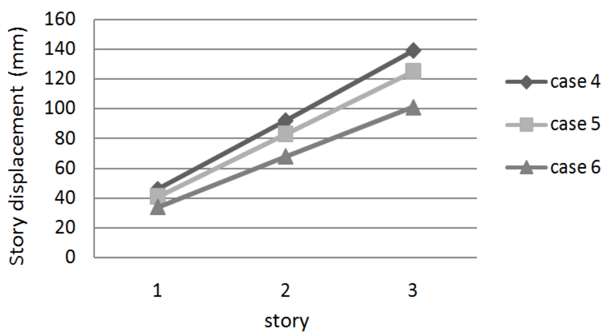


Fig. 26 The maximum displacement of floors under rectangular impact load

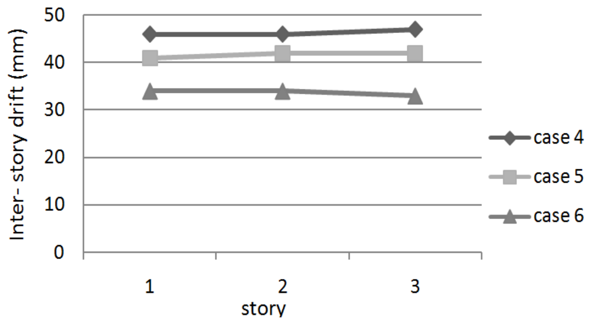


Fig. 27 The maximum displacement between floors under rectangular impact load

VII. Conclusion

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

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

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