Seismic performance of eccentrically braced frames

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Abstract. The main objective of this research is to assess the seismic performance of Eccentrically Braced Frames of different configurations. Modeled Eccentrically Braced Frames subjected to both linear and nonlinear analysis in SAP 2000. The linear analysis gives an insight to mode shapes and mass participation ratios. The nonlinear analysis includes the pushover analysis, which provides information about the collapse mechanisms and performance points. This study also extended to compare the performance of Eccentrically braced frames with the performance of Special Moment Resisting Frames that helps to understand the structural efficiency of both systems. This study concludes that all selected configurations of Eccentrically braced frames undergo small roof displacement that is well below the target drift whereas SMRF frames will show large displacement. 2D braced EBFs shows a better post yielding behaviour and is more ductile compared to other frame systems. Shear links in the EBF frames increase the stiffness of the frame which in turn results in high base shear demand. Post yielding behaviour of EBF frames can be improved by proper detailing of beam-column joints and link connections which confirms that performance of EBF frames is superior to SMRF frames in seismic areas.

1. Introduction
In the present era, the occurrence of earthquakes is quite often compared to ancient times. This increases the challenge in the construction industry to design the seismic-resistant structures which should reduce the human casualty, without much compromising the performance of the building. This resulted in the development of different types of lateral force resisting frames. In high seismic hazard areas, steel structures face the problem of Lateral stability. Eccentrically Braced Frame (EBF) is a combination of concentrically braced frame and moment-resisting frame which brings high elastic stiffness and high inelastic responses to the system [1, 2]. EBFs are one of the systems which resist the lateral load usually adopted in high seismic areas. Eccentrically braced systems have a ‘structural fuse’ which is the link element that dissipates energy through yielding. AISC Seismic Provisions for Structural Steel Buildings (1997) changed the link-to-column connections design requirements of EBF after the Northridge Earthquake. Currently, the AISC Seismic Provisions suggest the designer to either use a connection after the qualification testing or use a short shear link with reinforcement which exclude the inelastic action in the link-to-column connection. However, data available for the cyclic loading performance of EBF link-to-column connections, including connections with reinforcement are limited.

The present research work mainly focusing to understand the more realistic the behaviour of the EBF structures by the nonlinear static pushover analysis. This study also compares the performance of different Eccentrically Braced Frames with Special Moment Resisting Frames and thereby assess the efficacy of the EBF as a lateral force resisting structure. This study is limited to single bay nine-storey bare frame having horizontal shear yielding links.
2. Special moment resisting frames (SMRF) and Eccentrically braced frames (EBF)

In high seismic areas, Steel Moment Resisting Frames is preferred because of its high capacity of ductility, speed and ease of their construction. Lateral loads acting on this structural system is resisted by structural frame members and rigid joints. The lateral stiffness and strength of the whole structure depends on the strength of the frame and bending rigidity. Comparing to braced systems, moment frames requires members of larger size to keep the deflection in the lateral direction within the code limits, which may increase the cost of construction. The inherent flexibility of moment resisting frames may also induce non-structural damages under the earthquake excitation. After the occurrence of 1994 Northridge earthquake, it is seen that the performance of the special moment resisting frames is not up to the satisfactory level, because the high ductility property of the system is challenged by the brittle failure of beam-column connections [3]. Minimum inelastic rotation capacity of 0.03 radian is required for Beam-to-column connections in SMRFs as per AISC. Proper placing of semi-rigid connections along with the rigid connection could improve the performance of moment frames.

The unique characteristic of an EBF is that one end of the brace is joined to another brace or a column through a beam section called a link. Brace force is transmitted either through the shear and bending action of the link of an eccentric brace. Lateral stiffness of an EBF frame varies by the ratio of the length of the link to that of the beam. As the length of the link decreases, it makes the frame stiffer, thereby approaching the stiffness equivalent to a Concentrically Braced Frame. Long links make the frame more flexible thereby reduces the stiffness equivalent to that of a moment frame. The foremost function of the link is to provide a weak section in the frame so that its plastic deformation capacity can be utilised and thereby dissipates more energy due to earthquakes. Ductility and energy dissipation capacity of EBFs is explained, by evaluating the behaviour of EBF frames in cyclic load [4]. Well-designed EBF prevents buckling of braces and also link can withstand large deformations without loss in strength, and make stable hysteretic loops analogous to SMRF.

Short links yields in shear whereas the long links will form plastic moment hinges at the ends. An intermediate link shows significant amounts of both shear and moment yielding occur. Several shear link specimens are failed by fracturing the link web before achieving the required inelastic rotation levels [5] and finds that AISC 2002 loading protocol is inadequate. The revised loading protocol AISC 2002 for shear links achieves the inelastic rotations and thereby avoids the premature failure. Increasing the link beam length leads to a reduction in the stiffness of the structure whereas the ductility increases [6]. Premature failure of specimens is observed in the experimental investigation of link-to-column connections in eccentrically braced frames [7]. The inelastic behaviour of six Eccentrically Braced Frames under Non-Linear Range is studied in 2014 [8]. The V-braced frame is more economical whereas Eccentric Inverted-V frame shows a better performance. Stiffness, ultimate capacity, and inelastic member deformation requirements of an active beam links in Eccentrically Braced Frames were compared [9]. The behaviour of long links in eccentrically braced frames is analysed in 1989 [10]. Long links attached to columns is not usually preferred in EBFs. The present study aims to assess the performance of different configurations of Eccentrically Braced Frames in seismic areas. This study also aims to compare the seismic performance of Eccentrically Braced Frames and Special Moment Resisting Frames.

3. Methodology and Numerical modelling of frames

3.1 General

From the past researches, a suitable methodology is adopted to analyse the performance of the frames in the seismic environment as shown in Figure 1. The numerical modelling is done with the help of SAP 2000. The effective time period, roof displacement, performance points, lateral stiffness, post yielding behaviour, the formation of the hinge mechanism is discussed from the results of the analysed model. Link damage is stated in terms of post-yield and inelastic deformation limits.
3.2 Pushover method

This nonlinear static procedure can be used to find the maximum displacement of the structure. By using the capacity spectrum method, the intersection point of the capacity (pushover) curve and a reduced response spectrum can be found out (ATC40 Vol. 1). In this method, the global force-displacement capacity of the structure can be plotted and is used to compare the response spectra representations of the earthquake demands. In this analysis, the structure is loaded in a specific predefined pattern and is boosted until the target displacement or before the collapse. SAP2000 have the ability to carry out nonlinear static pushover analysis for both two and three-dimensional structures as described in ATC-40 and FEMA-273 documents.

3.3 Hinge details

The inelastic behaviour of SMRF is obtained by the introduction of plastic hinges which estimates the additional capacity of the frames beyond the elastic limit. In this research, auto hinges are used for beam and column elements. From Tables in ASCE 41-13, Moment M3 hinges are assigned at the beam endings and for columns, P-M2 hinges are provided at both ends. Nonlinear behaviour of EBF is captured by modelling the plastic hinges in the link member using the nonlinear modelling parameters as recommended by ASCE41-13 (ASCE, 2014). A deformation-controlled shear (V2) hinge is provided at the middle of the link where shear is expected to be maximum. All other elements are designed as elastic elements as they are expected to behave linearly. A default kinematics hysteresis model is considered in the study.

3.4 Modelling and Designing of frames

SAP2000 version 20 is the platform used for numerical modelling and analyses of EBFs. Both material nonlinearity and geometric nonlinearity such as P-Delta effects are considered for this study. Fixed base restraints are provided in all cases.

![Figure 1. Methodology.](image1)

![Figure 2. Different configurations selected for the study.](image2)
The 9 storey 2D building SMRF and EBF designed as per ASCE provisions. Grade of steel used for the frame is A992 steel G50. The single bay 9 storey 2D frame has a bay width of 9.15m, with a storey height of 3.96m. The soft storey of the frame have a height of 5.49m. The 2D frame is designed and is checked in SAP 2000. The designed frame section details are shown in Table 1 and Table 2. In this study, four different configurations of EBF frames are selected namely, Split V-Braced EBF, V-Braced EBF, D Braced EBF, 2D Braced EBF represented as EBF 1, EBF2, EBF3, EBF4 respectively.

### Table 1. Section details of EBF frames.

| Storey | Beam size | Column size | Brace size | Link size |
|--------|-----------|-------------|------------|-----------|
| 1      | W 150X24  | W 250X131   | HSS 335.6X335.6X12.7 | W 150X24  |
| 2      | W 150X22.5| W 250X131   | HSS 406.4X406.4X12.7 | W 150X22.5|
| 3      | W 150X22.5| W 250X115   | HSS 406.4X406.4X12.7 | W 150X22.5|
| 4      | W 150X18  | W 250X101   | HSS 406.4X406.4X12.7 | W 150X18  |
| 5      | W 150X18  | W 250X89    | HSS 406.4X406.4X12.7 | W 150X18  |
| 6      | W 150X13.5| W 250X80    | HSS 406.4X406.4X12.7 | W 150X13.5|
| 7      | W 150X13.5| W 250X73    | HSS 406.4X406.4X12.7 | W 150X13.5|
| 8      | W 150X13  | W 250X67    | HSS 406.4X406.4X12.7 | W 150X13  |
| 9      | W 150X13  | W 250X58    | HSS 406.4X406.4X12.7 | W 150X13  |

### Table 2. Section details of SMRF.

| Storey | Beam Size | Column Size |
|--------|-----------|-------------|
| 1      | W 150X24  | W 250X131   |
| 2      | W 150X22.5| W 250X131   |
| 3      | W 150X22.5| W 250X115   |
| 4      | W 150X18  | W 250X101   |
| 5      | W 150X18  | W 250X89    |
| 6      | W 150X13.5| W 250X80    |
| 7      | W 150X13.5| W 250X73    |
| 8      | W 150X13  | W 250X67    |
| 9      | W 150X13  | W 250X58    |

### Table 3. Link length of different configurations.

| Storey | EBF 1 | EBF 2 | EBF 3 | EBF 4 |
|--------|-------|-------|-------|-------|
| 1      | 1.22  | 0.61  | 1.08  | 0.61  |
| 2      | 0.61  | 0.60  | 1.08  | 0.61  |
| 3      | 0.61  | 0.60  | 1.08  | 0.61  |
| 4      | 0.91  | 0.50  | 1.08  | 0.46  |
| 5      | 0.91  | 0.50  | 1.08  | 0.46  |
| 6      | 1.22  | 0.61  | 1.08  | 0.61  |
| 7      | 1.22  | 0.61  | 1.08  | 0.61  |
| 8      | 1.83  | 0.91  | 1.08  | 0.99  |
| 9      | 1.83  | 0.91  | 1.08  | 1.83  |
4. Results and discussions

4.1 Linear analysis

During earthquake shaking, building oscillates which induces inertia force in the building. During earthquake shaking, inertia force is induced in the building due to its oscillation. The magnitude of induced inertia force, its intensity and duration of oscillation depends on the dynamic features of a building. Dynamic characteristics of building includes modes of oscillation and damping. For steel buildings, the damping ratio is commonly adopted as 2%. The modal analysis gives a mathematical description of structures about its dynamic behaviour. The time period and frequency of different structural systems are shown in Table 4.

| Structural Systems | SMRF | EBF 1 | EBF 2 | EBF 3 | EBF 4 |
|---------------------|------|-------|-------|-------|-------|
| Mode 1              |      |       |       |       |       |
| Time Period         | 1.995| 0.306 | 0.292 | 0.359 | 0.358 |
| Frequency           | 0.501| 3.268 | 3.425 | 2.788 | 2.795 |
| Mode 2              |      |       |       |       |       |
| Time Period         | 0.611| 0.133 | 0.102 | 0.133 | 0.158 |
| Frequency           | 1.636| 7.529 | 9.84  | 7.499 | 6.332 |
| Mode 3              |      |       |       |       |       |
| Time Period         | 0.276| 0.085 | 0.072 | 0.083 | 0.115 |
| Frequency           | 3.62 | 11.790| 13.825| 11.988| 8.719 |

From Table 4, it is clear that EBF has large stiffness as compared to a SMRF which owing to large time period to SMRF frames. SMRF undergoes large oscillations and takes more time to come back to its mean position. Introduction of braces in the structure improves the stiffness of the structure and reduced the time period. Among the four configurations, EBF 2 has shown the maximum stiffness and reduced the time period to 0.292 seconds. EBF 2 is 85% stiffer than SMRF so that it can attract large forces into it. Mass participation of EBF frames in higher modes is negligibly small as compared to SMRF. Thus, lower modes are dominating the overall response of the structure in EBF systems. Introducing braces in EBF improves the in-plane stiffness in the vertical plane and increases the torsional stiffness of the building.

4.2 Pushover analysis

Earthquake-resistant design controls the type of damage and sequence of its occurrence in various structural elements of a building. In this design, some structural damage is allowed for normal buildings, but it prevents the collapse condition during a strong earthquake shaking. The nonlinear analysis makes it possible to understand the whole failure mechanism of the building. In this study, pushover analysis is used to study the nonlinear behaviour of the structure. The whole deformability of the building is assessed from the Pushover Analysis. The ideal lateral load-deformation curve of a building in pushover analysis reveals the linear, nonlinear and plastic behaviour under the monotonic lateral displacement loading.

Five different types of bare frames with the same cross-sections are analysed for the study and the undergo pushover analysis. The graph showing the variation of base shear with displacement is shown in Figure 3. EBF 4 is showing the maximum lateral strength and more ductile compared to other systems whereas EBF 2 is the stiffest configuration. EBF 2 frame has two links at a floor level which is connected to fixed support using braces. Additional support increases the redundancy of the structure and makes the structure stiffer. SMRF frames show poor resistance to lateral load and are more flexible compared to other systems. When the demand curve goes through the capacity curve of the structure makes the performance point.
Figure 3. Graph showing variation of base shear with displacement for different structural systems.

Table 5. Performance points of different configurations.

| Parameters          | SMRF | EBF 1 | EBF 2 | EBF 3 | EBF 4 |
|---------------------|------|-------|-------|-------|-------|
| Performance Point, V (K N) | 16.223 | 218.967 | 213.75 | 191.572 | 186.494 |
| Roof Storey Displacement (M) | 0.3 | 0.033 | 0.029 | 0.038 | 0.044 |
| Effective Time Period (S) | 1.599 | 0.257 | 0.245 | 0.305 | 0.309 |

Table 5 illustrates the performance point of all five frames and is compared. EBF1 attracts a large amount of lateral force compared to other frames. SMRF frames have larger roof displacement and base shear demand of magnitude 16.223k N. EBF2 frame reduces the roof storey displacement to 90% as compared to SMRF frames. SMRF resist lateral loads by using both beams and columns. The axial forces, bending moment and shear force generated in them will handle these forces. Braces and links in EBF frames diminish the overall lateral displacement, bending moment and shear force demands on beams and columns. Thus, EBF frames are able to resist more lateral loads as compared to SMRF and dissipate energy through shear deformations (shear link).

4.3 Post yield mechanisms

Due to reversed cyclic loading, inelastic material steel will enter into a stage beyond yield and eventually hysteresis will take place. EBF frames enter into the inelastic range after the yielding is much earlier than SMRF frames. EBF frames designed by the capacity design principles. Here links beam is the weak element which shows the inelastic response. In all the configurations of EBF, it is seen that nonlinear hinges are formed only in the link elements which are assigned with shear hinges and all other elements are in the elastic range. This behaviour of frames validates the capacity design principles. Figure 4 shows the collapse mechanisms of different frames. Uniformity in forming the hinges is not achieved in these frames. In EBF 3 formation of hinges is concentrated only on the first floor whereas in EBF 1 and 2 shows the formation of more hinges in two stories. Among all the four EBF frames, EBF 4 shows a better formation of hinges. Non-uniformity in the formation of hinges is due to the presence of the soft storey. The soft storey is subjected to large forces and is stressed more compared to other storeys resulting in the damage to be concentrated in the soft storey and it results in a collapse of link elements.
From the analysis, the EBF 4 frame shows that up to a roof displacement of 0.02, the frame is in the collapse prevention stage. Table 6 shows the maximum roof displacement permitted to each frame before the collapse. EBF 4 frame shows the maximum displacement of 0.129 m before collapse confines that it is more ductile compared to other frames.

![Figure 4](image)

**Figure 4.** Collapse mechanisms of EBF frames.

Table 6. Maximum displacement of different configurations.

| Frames | Maximum Displacement (m) |
|--------|--------------------------|
| EBF 1  | 0.101                    |
| EBF 3  | 0.094                    |
| EBF 4  | 0.129                    |

5. Conclusions

Seismicity of the world is increasing day by day. This creates a new challenge in the construction industry to create more earthquake-resistant infrastructures having desired performance and economically viable structures. In this study, seismic performance of different configurations of EBF is analysed and is compared with the performance of SMRF. The main criteria selected for the study are target drift and post yield mechanisms. The frames under study are subjected to both linear and nonlinear analysis. Major observations and conclusions arrived from the study are:

1. Mass participation ratios of EBF frames are much more in lower modes than SMRF. Therefore, the lower modes predominate the overall response of the structure.

2. Modal analysis suggest EBF 2 is stiffer than all other frames, having a natural time period of 0.292 seconds.

3. Pushover analysis gives the performance point of various frames. Introduction of braces and links in EBFs makes the frame stiffer and reduces the displacement. EBF 2 reduces displacement to 90% as compared to SMRF whereas it enhances the demand of the structure to a magnitude of 213.75 kN.

4. SMRF show a maximum displacement of 300mm whereas EBF frames shows a maximum displacement of 44mm which is below the target displacement of 93mm.

5. Link beams dissipate energy through shear deformations which reduce the demand on beams and columns.

6. Post yield mechanisms of EBF 4 shows a better formation of hinges as compared to other EBF frames.
7. Non uniformity in the formation of hinges is due to the presence of the soft storey. This can be improved by proper detailing of beam column joints and link connections.

8. EBF 4 allows a displacement of 0.129m before the collapse and is exhibiting more ductile behaviour.

9. Among all these frames, EBF 4 can be selected as the better configuration, whose displacement values are below the target drift, enough lateral strength, better post yielding mechanisms and is more ductile compared to other frames.

10. Stiffness of the structure can be improved by EBF frames results in minimum displacement as compared to SMRFs.

6. References
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