Structural performance of concentrically and eccentrically braced frame

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**Abstract.** Indonesia is prone to earthquakes. Therefore, the building needs to be designed to withstand earthquake lateral force. One of the lateral-resistant elements is a steel frame system with bracing. There are two types of bracing based on its configuration, that is Concentrically Braced Frame (CBF) and Eccentrically Braced Frame (EBF). The objective of this research is to examine the capability (lateral resistance, plastic hinge mechanism, and ductility factor) of CBF and EBF. Structural analysis is performed using SAP2000 v.18. Pushover analysis is performed on Moment Resisting Frame (MRF), CBF, and EBF with various link lengths $e = 0.4$ m, $0.6$ m, $0.8$ m, and $1.0$ m. The increase in lateral resistance resulting from the use of CBF is $74\%$ and for EBF is $52\%$ compared to MRF. The plastic hinge mechanism for CBF is marked by the formation of plastic hinges on bracing, while EBF is marked by the formation of plastic hinges on the links then beams and columns. The ductility factor for CBF is $45.75\%$ smaller than MRF. While for EBF, the ductility factor is $18.20\%$ smaller than MRF but $33.68\%$ larger than CBF. Therefore, EBF is more ductile than CBF.

1. **Introduction**

The Indonesia archipelago is an area with three tectonic plates, namely the Indo-Australian, Eurasian and Pacific plates. Indonesia's geographical position in such a way caused it to frequently suffer either major or minor earthquakes. In the last few years, there has been a major earthquake in Indonesia, such as the Aceh earthquake in 2004 (9.3 on the Richter Scale), the Sumatra earthquake in 2005 (8.2 Richter Scale), and many other earthquake events [1]. Major damage was suffered. Therefore, earthquake resistance building is now taking a major role in the Indonesian building code [2]. The earthquake lateral force is essential in building design. Nowadays, the use of steel structures is quite popular in Indonesia, such as in high-rise buildings with the effect of high lateral force [3]. The lateral resisting structural system is often used to provide resistance for both wind and seismic loading [4]. The use of steel bracing as a lateral system is commonly used because it does not increase the structural weight significantly [5].

In AISC 2010 [6] and SNI 1729:2015 [7], there are two types of bracing based on its configuration, which are Concentrically Braced Frame (CBF) and Eccentrically Braced Frame (EBF). The objective of this research is to examine the capability (lateral resistance, plastic hinge mechanism, and ductility factor) of CBF and EBF.

2. **Literature review**

Pushover analysis Dewobroto and Wiryanto [8] is a computer-based nonlinear analysis by providing a static lateral load pattern on the structure, which is then gradually increased by a multiplier until it yields (plastic hinges) at one or more locations on the structure. Plastic hinge is developed due to the inability of structural elements (beams and columns) to resist the internal forces. Commonly, building with steel bracing tends to develop plastic hinge on bracing rather than column. A plastic hinge is expected to
occur only on the beam and the lowest column (base). Therefore, it is necessary to apply the concept of "strong column weak beam". A pushover analysis produces a capacity curve that describes the relationship between total lateral shear force \((V)\) and top lateral displacement \((D)\). The capacity curve is then compared with the spectral response of various intensities of the quake to observe the structural behavior under the given seismic force. The capacity curve can be also used to determine ductility, stiffness, and structural strength. By estimating the maximum shear forces and displacements, it can be clearly shown which structural members are collapsing. Therefore, weak structural members could be anticipated.

The performance targets consist of designated earthquake events and the level of damage or performance level of the building against the occurrence of the earthquake. Based on FEMA-440 [9], as the reference of performance-based design, there are several levels of structural performance, known as acceptance criteria, such as (Figure 1).

- **Operational (O)** is a level where buildings are expected to sustain minimal or no damage to their structural and non-structural components. There may be certain facilities in the building not functioning properly, such as power, water, and other required utilities, and possibly some nonessential systems, which makes it unsuitable for its normal use and occupancy. Extremely low risk of injury exists in buildings meeting this performance level.

- **Immediate Occupancy (IO)** is a level where buildings are expected to sustain minimal or no damage to their structural elements and only minor damage to their non-structural components. Buildings that meet this level of seismic performance would be safe to reoccupy immediately following a major earthquake, but non-structural systems may be unavailable due to either a lack of electrical power or equipment damage inside. Even though re-occupation of the building could occur immediately, some clean-up and repairs may still need to be done, and utility service may not be restored until the building can function normally again. The risk to life safety at this performance level is very low.

- **Life-Safety (LS)** is a level where buildings may experience extensive damage to structural and non-structural components. In some cases, reoccupation of a building will require repairs, and such repair may not be economically feasible. The risk to life in buildings meeting this performance level is low.

- **Collapse Prevention (CP)** is a level where buildings may pose a significant hazard to life safety resulting from the failure of non-structural components. The building itself does not collapse, so there should be no mass casualties. These levels of damage will result in complete economic losses for many buildings.

![Figure 1 Capacity curve (shear force vs displacement)](image-url)
Figure 1 shows shear force versus displacement from point A-B-C-D-E. This point presents the characteristics of plastic hinges on structural elements. Point A is the starting point, point B indicates the first yielding state on a structural element, point C is the ultimate condition, point D indicates the residual strength, and point E is the collapse state. The important aspect of high-rise building design is the structural stability and ability to withstand lateral forces, whether caused by wind or earthquake. For steel buildings, it is common to use bracing as the lateral resisting element. It is also known as a braced frame system. Steel bracing can minimize the stiffness problem found in Moment Resisting Frame (MRF). Commonly, the braced frame structural system can be classified into two types: the CBF and EBF. CBF is a structural system for resisting lateral loads with high structural stiffness, by introducing a diagonal stiffening element (bracing) which serves to resist lateral loads on the structure. Bracing is expected to be deformed in large inelastic form without significant loss of strength and stiffness of the structure. The disadvantage of this system is the absence of architectural flexibility due to bracing location. There are several types of braced frames, such as (a) type Z, (b) type X, (c) inverted type V, and (d) type V (Figure 2).

EBF is a structural system using a diagonally stiffening element and short beam element called a “link”. The link is designed to withstand shear and bending moments. It is also designed to have inelastic behavior so that the beam element does not suffer significant damage to prevent structural collapse. The architectural advantage on EBF can be expected to be better than CBF because there would be more space to be explored. Some types of confinement portals are: (a) type Z, (b) type V, (c) inverted type V, and (d) type Z (Figure 3).

3. Methodology
Structural analysis is performed using SAP2000v18. The study was conducted by analyzing and comparing the structural performance between CBF and EBF. Pushover analysis is performed on MRF, CBF, and EBF with various lengths of link (e = 0.4 m, 0.6 m, 0.8 m, and 1.0 m). Commonly, two different types of load patterns are used. First is the load pattern due to gravity load and the second is the load pattern based on the lateral load distribution. The pushover analysis steps can be described as:

- Model the structure (include all elements of the building).
- Define the gravity loads.
- Define the lateral force.
- Assign a plastic hinge on each structural element.
- The lateral loading is increased step by step until the weakest component of the structure deforms.
- The 5th step is repeated until several components reaching the limit condition of their strength.
- Plot the shear force for each load stage to illustrate the nonlinear behavior response of the structure (capacity curve).

A multi-story steel building (Figure 4 and 5) was analyzed with the following data. The building functions as an office and is located in Jakarta. Load value is based on SNI 1727:2013 [10]. Soil classification is medium type. The response spectrum is based on SNI 1726:2012 and PUSKIM earthquake maps [11]. For EBF, there are four link lengths \( e = 0.4 \text{ m}, 0.6 \text{ m}, 0.8 \text{ m} \) and \( 1.0 \text{ m} \).
4. Result and discussion

Inter-story drift should meet the limit requirements based on the used lateral resisting element. Table 1 shows the results of inter-story drift for CBF and EBF. The largest inter-story drift for CBF is 1.67 cm while for EBF is 1.43 cm for \( e = 0.80 \) m. Both values are still less than the limit value of 5.38 cm. Figure 6 shows the shear force versus displacement. From Figure 6 it can be seen that CBF has greater strength to resist seismic forces than other models (EBF and MRF). The ultimate shear force for CBF is 234210.48 kg. The percentage of shear force increase by about 74.0% for CBF compared to MRF. While for EBF with link length \( e = 1.0 \) m can resist the shear force 126438.58 kg greater than the other link length. It is higher about 52.0% for EBF with \( e = 1.0 \) m compared to MRF. It can be concluded that the shear force is increasing with the increase of link length.

| Story | \( h \) (m) | \( \Delta_j \) (m) | limit \( \Delta \) (m) |
|-------|-------------|-------------------|------------------|
| Roof  | 3.5         | 0.0123            | 0.0538           |
| 8     | 3.5         | 0.0144            | 0.0538           |
| 7     | 3.5         | 0.0161            | 0.0538           |
| 6     | 3.5         | 0.0167            | 0.0538           |
| 5     | 3.5         | 0.0167            | 0.0538           |
| 4     | 3.5         | 0.0156            | 0.0538           |
| 3     | 3.5         | 0.0136            | 0.0538           |
| 2     | 3.5         | 0.0110            | 0.0615           |
| 1     | 4           | 0.0085            | 0.0615           |

The plastic hinge mechanism for CBF can be seen in Table 2 and Figure 7. The pushover analysis results for CBF indicate that the plastic hinges begin to occur on step 1 with performance level B (bracing begins to yield). In step 3, the structural elements begin to develop the plastic hinges with the performance level of Immediate Occupancy (IO) where there has been no significant damage to the structure followed by the appearance of the plastic hinges with the performance level C on step 4. At this point, the structural elements have reached their ultimate state. The ultimate shear force is 234210.48 kg with a maximum displacement of 0.30 m.
Figure 6 Shear force vs displacement

Table 3 Plastic hinge mechanism for EBF with e = 1.0 m

| Step | Displacement Δi (m) | Shear force Vi (kg) | Plastic hinge mechanism |
|------|---------------------|---------------------|-------------------------|
| 0    | 0.000088            | 0                   | A-B 224                 |
| 1    | 0.071592            | 69103.94            | B-IO 0                  |
| 2    | 0.092675            | 80704.34            | IO-LS 223              |
| 3    | 0.160807            | 100288.9            | LS-CP 210              |
| 4    | 0.265838            | 116529.06           | CP-C 203               |
| 5    | 0.356535            | 125994.93           | C-D 196                |
| 6    | 0.3626              | 126438.58           | D-E 196                |

Figure 7 Plastic hinge mechanism for CBF
The plastic hinge mechanism for EBF with \( e = 1.0 \) m can be seen in Table 3 and Figure 8. The result of pushover analysis for EBF with \( e = 1.0 \) m indicates that the plastic hinge starts to occur on step 2 with performance level B. In step 4, the structure begins to develop the plastic joints with performance level Immediate Occupancy (IO), followed by the appearance of plastic joints with performance level Life Safety (LS) at step 5. The ultimate shear force is 126438.58 kg with a maximum displacement of 0.36 m. The ductility factor is obtained from the ultimate displacement divided by the first yield displacement (Table 4). The ductility factor for CBF is 45.75% smaller than MRF. While for EBF, the ductility factor is 18.20% smaller than MRF but 33.68% larger than CBF.

### Table 4 Ductility factor

| No | Type               | Yield point displacement (m) | Ultimate displacement (m) | Ductility factor (\( \mu \)) |
|----|--------------------|------------------------------|---------------------------|-------------------------------|
| 1  | MRF                | 0.1841                       | 1.1401                    | 6.1917                        |
| 2  | CBF                | 0.0916                       | 0.3076                    | 3.3588                        |
| 3  | EBF (e = 0.4m)     | 0.0500                       | 0.1697                    | 3.3938                        |
| 4  | EBF (e = 0.6m)     | 0.0554                       | 0.2302                    | 4.1594                        |
| 5  | EBF (e = 0.8m)     | 0.0625                       | 0.2991                    | 4.7863                        |
| 6  | EBF (e = 1.0m)     | 0.0716                       | 0.3626                    | 5.0648                        |

## 5. Conclusion

There are several conclusions obtained from these results:

- The increase in shear force resistance for the CBF is 74%, whereas EBF is 52%, compared to MRF. Therefore, CBF has a higher elastic stiffness than EBF.
- The plastic hinge mechanism for CBF is marked by the formation of plastic hinges on bracing, while for EBF is marked by the formation of plastic hinges on the links then beams and columns.
- The ductility factor for CBF is 45.75% smaller than MRF. While for EBF, the ductility factor is 18.20% smaller than MRF but 33.68% larger than CBF. Therefore, EBF is more ductile than CBF.

## References

[1] Budiono, Bambang. 2011. Concept of SNI 1726-201X. Jakarta: HAKI 2011
[2] National Standardization Agency. 2012. SNI 1726:2012 Earthquake Resistance Design Procedures for Structural and Non-Structural Building. Jakarta: National Standardization Agency
[3] Dewobroto, Wiryanto. 2016. Steel Structures, Behaviour, Analysis, and Design–AISC 2010. Yogyakarta: Pelita Harapan University
[4] Safariski, H., 2012. Evaluation of Use of Steel Bracing in Improving Structural Performance Concrete Earthquake, ISSN 977-19799705 Civil Engineering Journal Vol XIII No. 1 Mar 2012
[5] Senila, M. & Petran, I., 2016, Development of Plastic Hinges in Steel and Composite Beams of Eccentrically Braced Frames, page 75-84 Vol 13 no. 1 Intersections/Intersecți, ISSN 1582-3024

[6] American Institute of Steel Construction. 2010. Seismic Provisions for Structural Steel Buildings. America: American Society of Civil Engineering

[7] National Standardization Agency. 2015. SNI 1729:2015 Specifications for Structural Steel Buildings. Jakarta: National Standardization Agency

[8] Dewobroto, Wiryanto. 2006. Performance Evaluation of Earthquake Resistant Steel Structures with Pushover Analysis. Yogyakarta: Pelita Harapan University

[9] Federal Emergency Management Agency. 2005. FEMA 440 Improvement of Nonlinear Static Seismic Analysis Procedures. Washington, D.C

[10] National Standardization Agency. 2013. SNI 1727:2013 Minimum Loading for Building Design and Other Structures. Jakarta: National Standardization Agency

[11] Center for Settlement Research and Development. 2011. Design of Indonesia Spectrum. (Online), (http://puskim.pu.go.id/Aplikasi/desain_spektra_indonesia_2011/, accessed on 18 Mei 2017)