IN-ELASTIC PERFORMANCE OF 2D-TWO BAY ORDINARY CONCENTRICALLY BRACED STEEL FRAME

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ABSTRACT

In this study, in-elastic performance of 2D-two bay ordinary concentrically braced steel framed is observed. The steel structure has equally 6m bays, and five stories with uniform 4m floor to floor height. It is assumed as a part of common office building resting on soft soil of Zone 6 in Indonesian Seismic Map (SNI 03-1726-2002). The seismic reduction factor, R is taken as much as 3.0. All braces are assumed to reach their capacity during severe earthquake event. The steel grade used for whole structure is kept the same in order to stimulate the commonly available steel used in Indonesian construction industry (f_y=240 MPa, f_u=370 MPa). Steel design is carried out by using Indonesian Steel Structure Code (SNI 03-1729-2002), while the in-elastic performance is evaluated by non-linear static pushover analysis. The non-linear hinge properties are adopted from Table 5 of FEMA 356 which and generated automatically inside SAP2000 software package.

Design result of the particular chosen geometry size, shows that the braces are controlled dominantly by slenderness ratio limitation. To satisfy the limitation criteria, the maximum axial stress ratio of the braces is still as low as 0.389. Beams and columns, which have to remain elastic while subjected to axial resistance capacity of the braces, have higher reserve capacity. Observed in-elastic behavior by non-linear static pushover shows that the structure still remains in elastic range due to design earthquake (500 years return period). However, with increasing demand spectra, it is observed that plastic hinges start to develop in the lower story of compression braces and propagate to higher story. It is concluded that the code provision succeeds to isolate the damage in braces. However, slenderness ratio limitation of braces specified by the code, if not accompanied by appropriate selection of steel grade (i.e. using lower steel grade), can result in over-conservative design.

Keywords: Seismic Performance, Ordinary Concentrically Braced Steel Frame, Non-linear Static Pushover.

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1. INTRODUCTION

To design seismic resistant structures with ductility concept is widely accepted practice. For any systems, the design should be conducted carefully that the damage will occur in selected parts of the structure which leads to a ductile mechanism. This concept is commonly adopted in the building codes. In Indonesia, such code for steel structure is the SNI 03-1729-2002. One system which is covered by the code is the ordinary concentrically braced frame (OCBF). OCBF has the most significant advantage of being very stiff, while maintains a reasonable ductility level. This system is not rare be used as substitution for moment resisting frame system. Conceptually, the OCBF stable ductile behavior is controlled by buckling mechanism of the braces, while maintain the beams and columns to remain elastic (Bruneau et.al. 1998).

2. STRUCTURE CONSIDERED

In this study, in-elastic performance of 2D-two bay ordinary concentrically braced steel framed is observed. The steel structure has equally 6m bays, and five stories with uniform 4m floor to floor height. It is assumed as a part of common office building resting on soft soil of Zone 6 in Indonesian Seismic Map (SNI 03-1726-2002). The seismic reduction factor, R is taken as much as 3.0. All braces are assumed to reach their capacity during severe earthquake event. The steel grade used for whole structure is kept the same in order to stimulate the commonly available steel used in Indonesian construction industry (BJ37 with f_y=240 MPa, f_u=370 MPa). All frame connections are considered rigid, while braces are pin connected to the frames. The structure illustration can be seen in Figure 1.

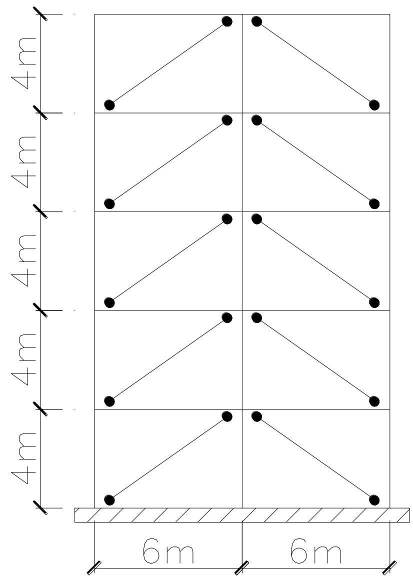


Figure 1: The Structure Considered.

3. STEEL DESIGN

Steel design is carried out by using Indonesian Steel Structure Code (SNI 03-1729-2002). Braces are designed as axial compression and tension members. In addition to conventional axial member design, a slenderness ratio limitation (Equation 1) should be satisfied in ordinary concentrically braced frame.

$$\frac{k \cdot L}{r} < \frac{1900}{\sqrt{f_y}} \qquad (1)$$

In Equation 1, the effective length factor, k is taken as 1.0 (braced simple connected member), while L, r, and f_y indicate un-braced length, radius of gyration, and steel yield strength, respectively. Beams and columns are designed as beam column members, as they suffer from both axial and bending actions. The frame should be designed stronger than the braces to ensure the plastic mechanism is controlled by the buckling of braces. This is accommodated by designing the beams and columns to withstand the axial resistance capacity of the braces in addition to the factored stress resultants (Bruneau et.al. 1998). The complete steel design can be seen in Figure 2.

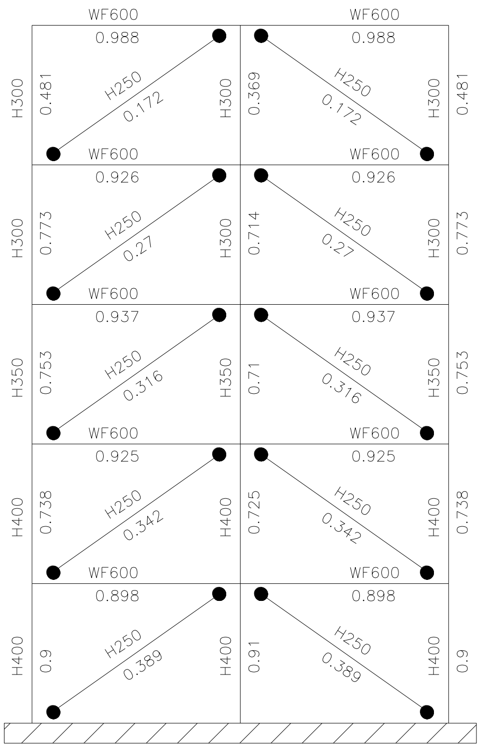


Figure 2: Steel Design Result.

The numbers shown in Figure 2 are the axial interaction ratio, and combined axial-bending interaction ratio in braces, and the frames, respectively. It should be noted that in this particular study, the braces design is governed by the slenderness ratio limitation (Equation 1). Thus, the braces still have relatively low axial interaction ratio (0.389). With the fact that the frames design is dominated by axial resistance capacity of the braces, they are more conservatively designed than the braces.

4. IN-ELASTIC PERFORMANCE OF THE STRUCTURE

In-elastic performance is evaluated by non-linear static pushover analysis. The non-linear hinge properties are adopted from Table 5 of FEMA 356 which and generated automatically inside SAP2000 software package. Performance point at design earthquake demand spectra (500 years return period) can be seen in Figure 3. Figure 3 shows that the structure still in elastic range due to the conservative design. If one increases the demand spectra beyond the design earthquake, it is observed that plastic hinges develops. The hinges start to form in the lower story of compression braces and develop to higher story (Figure 4a). Finally, Figure 4b shows the hinge formations with base shear as much as 2.24 times the design earthquake.

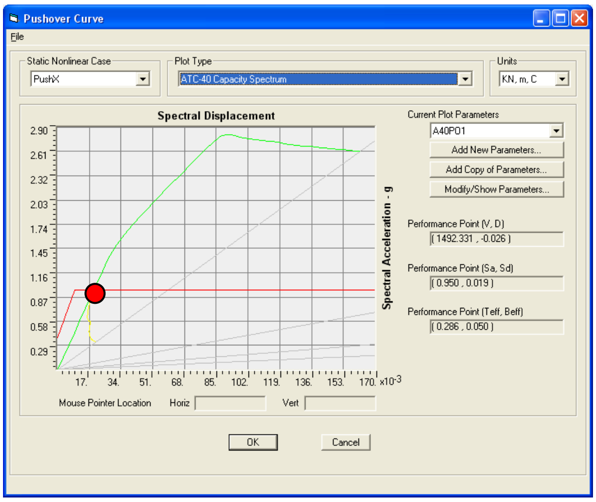
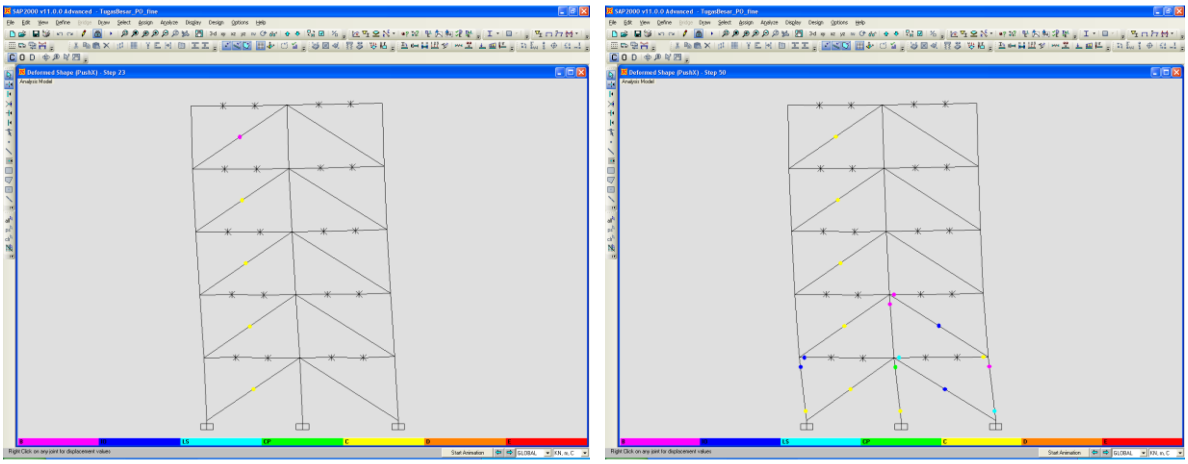


Figure 3: Performance Point



(a) (b)

Figure 4: Subsequent plastic hinge development.

5. CONCLUSION

It can be concluded that slenderness ratio limitation of braces specified by the code, if not accompanied by appropriate selection of steel grade (i.e. using lower steel grade), can lead to over conservative design. However, the expected mechanism is achieved in the study that the frame is damaged after the braces yields at their axial capacity.

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