DETERMINATION OF LATERAL CAPACITY OF TWO-STORY X-BRACED FRAMES CONSIDERING HYSTERESIS BEHAVIOU OF CONNECTIONS

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ABSTRACT

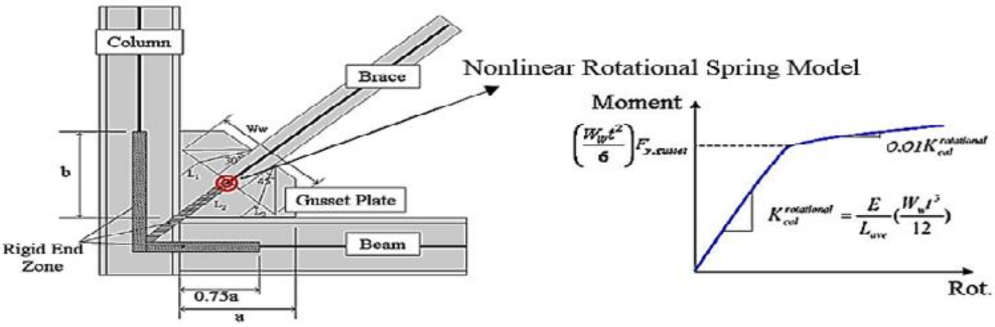
Special concentrically braced frames (SCBF) are stiff, strong and economical lateral load resisting systems which can sustain large inelastic deformation if properly detailed. The failures observed during recent earthquakes, such as bracing failures or deterioration of different types of connections, make it necessary to investigate these structural resisting systems more accurately. The main objective of this paper is to determine seismic lateral capacity of special concentrically two-story x-braced frame systems for responses in roof drift, interstory drift, base shear, frame fracture-index and failure contribution of each story in overall deterioration using nonlinear static cyclic analysis in OpenSEES software. Therefore, in this study, in order to have a thorough investigation of frame regularity, eight SCBFs were designed based on AISC seismic provision to represent different values for response reduction factor, R with identical plans and elevations. For performing analysis, recommended FEMA461 loading history was applied. In addition to modeling hysteresis behavior of gusset plate connections, shear tab connections were also considered. Therefore, for validating of this, a three-story one-bay tested in NCREE in Taiwan was particularly used. To identify the bracing fracture developements, the maximum strain range has been monitored. Due to the lack of the fracture expression implementation in the OpenSEES framework based on maximum strain range limit, fracture material model was imported in the program with the ability of being open-source in OpenSEES. Moreover, for validation of hysteresis behaviour of brace failure beyond brace fracture, a one-story one-bay tested in University of Washington was used.

INTRODUCTION

Special concentrically braced frames (SCBFs) are one of the economical systems providing high strength and stiffness against lateral earthquake loads for low-rise buildings in high seismicity regions. SCBFs are a special class of CBFs which complies with specific details of gusset plate connections and maximizes inelastic lateral capacity and ductility of buildings. Given the importance of seismic behavior of gusset plate connection to improve the seismic performance of SCBF (Lehman and Roeder, 2008), new restrictions are currently proposed on the design of gusset plate connections, resulting in creating a new balanced design procedure (BDP), maintaining elliptical clearance for corner gusset plate connections (NEHRP, 2013). Nowadays, an issue that has been the new concern of building engineers is to obtain the ultimate strength of a structure designed against earthquake lateral forces. The most precise seismic analyses are nonlinear time history analyses which are not only complex but also highly costly and require highly complex calculations and suffer from the lack of ground motion data and researchers have been trying to find an alternative way to be secure, accurate and yet simple. In this regard, the main activities are in line with nonlinear static analyses techniques. The first study in this regard belongs to Han and Chopra (2006) and there is also some discussion on displacement-based design criteria in the investigation of researchers such as Priestley (1997) for relocation of capacity-based design toward displacement-based design. In this article, in order to estimate lateral capacity of roof drift, interstory drift and base shear for special concentrically two-story x-braced frames which are regular in height and plan, displacement-based cyclic pushover analysis has been performed. For modeling hysteresis behavior of gusset plate and shear-tab connections of SCBFs, improved analytical modeling recommended by NEHRP (2013) has been used. Also for predicting bracing deterioration beyond the brace fracture, maximum strain range limit has been utilized (Hsiao et al., 2012). For performing cyclic pushover analysis, OpenSEES software is used with the ability of being open source. To conduct this study, the layouts of a set of three-story special concentrically two-story x-braced frame buildings designed according to the latest seismic design methods of gusset plate connections for SCBF and BDP have been selected. In addition to estimate capacity response of considered braced frames in incipient collapse, frame fracture-index has been defined and failure contribution measurement of each story in value of frame fracture-index has been reported.

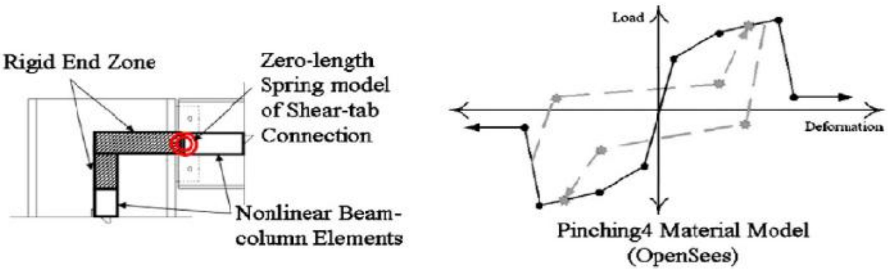
HYSTERESIS BEHAVIOR OF MEMBERS, CONNECTIONS AND BRACE FRACTURE

For modeling hysteresis behavior of beam, column, brace members, shear-tab and gusset plate connections in OpenSEES framework, STEEL02 material with ability of kinematic hardening behavior from the OpenSEES library was selected. Improved analytical modeling of gusset plate and shear-tab connections are illustrated in Fig. 1 and 2, respectively.



(a) Geometric details of gusset plate connection model (b) Material behaviour

Figure 1. Schematic structural model of gusset plate connection (NEHRP, 2013)



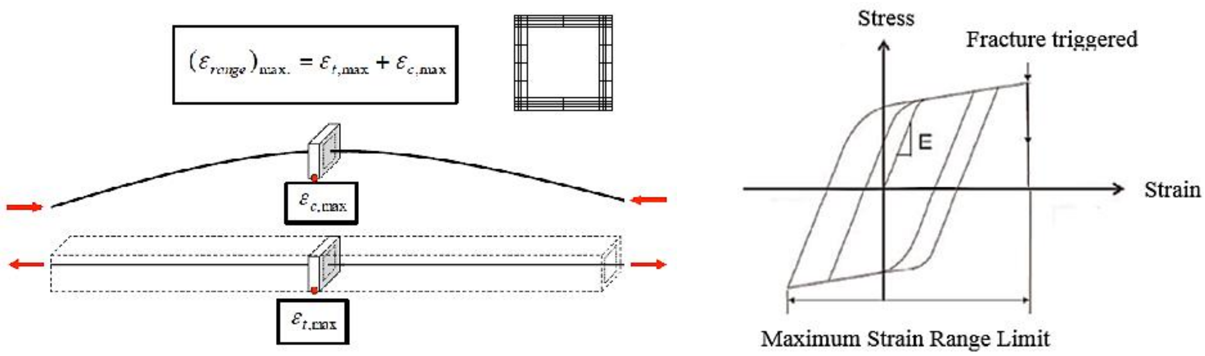
(a) Geometric details of shear-tab connection model (b) Material behavior

Figure 2. Schematic structural model of shear-tab connection (NEHRP, 2013)

For considering deterioration criterion of bracing members and noting that the most likely location of the brace fracture with HSS section is in the middle of bracing members, maximum strain range limit capacity has been used as shown in Eq. (1) (Hsiao et al., 2012).

$$\text{Max.}\,\varepsilon_{range,pred.} = 0.1435\left(\frac{w}{t}\right)^{-0.4}\left(\frac{KL}{r}\right)^{-0.3}\left(\frac{E}{F_y}\right)^{0.2} \tag{1}$$

Brace fracture depends on the width–thickness ratio of the cross-section (w/t), the slenderness ratio of the bracing member (KL/r) and the ratio of the elastic modulus to the yield strength of the steel (E/Fy). In this article, due to the lack of fracture expression implementation in the OpenSEES framework based on maximum strain range limit, fracture material integrated with steel02 constitutive material model was imported with a .dll file in the program with the ability of being open-source in OpenSEES. To better understand the calculation of maximum strain range limit in the brace member and to model the brace fracture, Fig. 3 (a) and (b) has been presented, respectively.

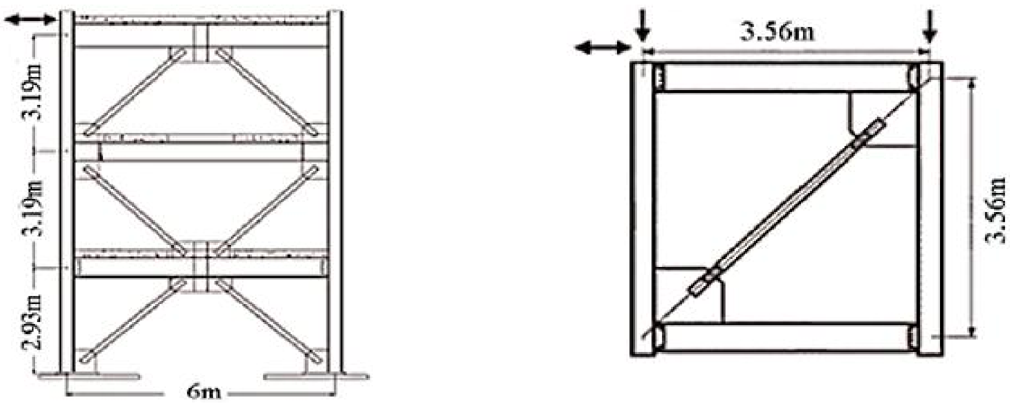


(a) Schematic of the midspan fiber section for brace model (b) Schematic of the fracture material model
Figure 3. Basis of fracture material (Hsiao et al., 2012)

Strain range at the outermost compression fiber of brace section is calculated and after the initiation of fracture at the time of reaching the maximum strain range, deterioration moves toward other fibers in the cross section progressively.

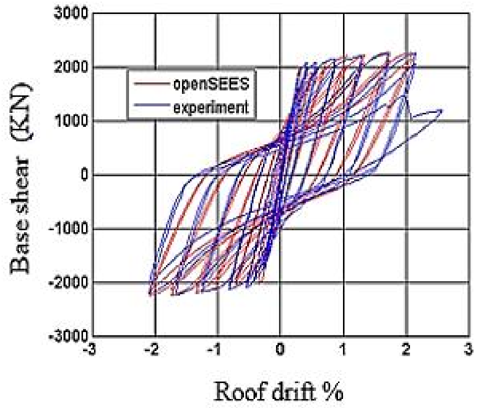
VALİDATİON OF HYSTERESİS AND FRACTURE RESPONSE OF SCBF SYSTEM

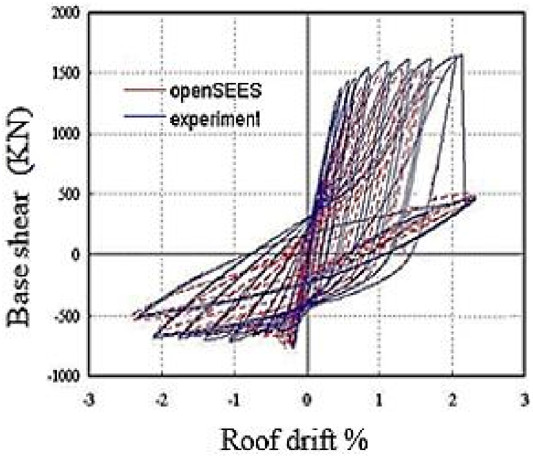
For validation of hysteresis behaviour of gusset plate and shear tab connections, a three-story one-bay tested in NCREE in Taiwan (TCBF2-1(HSS)) was particularly used. In addition, for validation of hysteresis behaviour of braced frame beyond brace fracture, a one-story one-bay tested in University of Washington (HSS13) was used. Configuration and geometry of both the tested frames are shown in Fig. 4.



(a) Three-story one-bay frame (b) One-story one-bay frame
Figure 4. Tested geometric configuration in Washington and Taiwan (Lumpkin, 2009), (Katulka, 2007)

In these experimental case studies, all the bracing members were box HSSsection and American wide-flange sections were used for all the beams and columns. Experimental and numerical data obtained from the validation have been shown in Fig. 5.





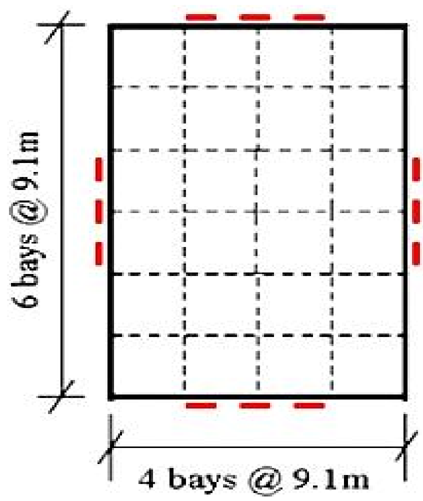
a) Response of three-story one-bay frame b) Response of one-story one-bay frame

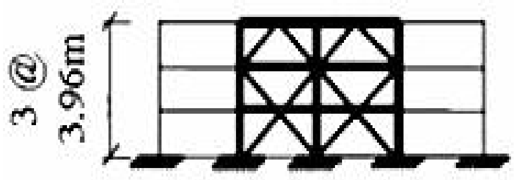
Figure 5. Roof drift-base shear response of the tested frames

By comparing the experimental and numerical results illustrated in Fig. 5, (a) and (b), it can be observed that the differences in initial stiffness, base shear demand and capacity response in each step, roof drift capacity and dissipated energy under the curve are in acceptable level before overall deterioration after the brace fracture. As well as hysteresis response of displacement-force curve, the response of one-story one-bay tested frame, beyond the brace fracture is shown in Fig. 5, (b), which is reasonably consistent with the experimental results, progressively.

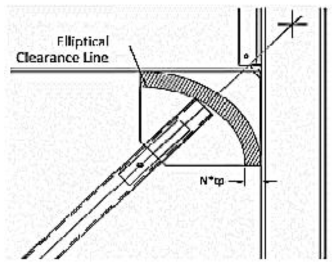
CYCLIC STATIC ANALYSIS AND MODEL BUILDINGS

To conduct this article, eight model building layout sets of three-story special concentrically two-story x-braced frames, according to AISC (2010) using the equivalent lateral force were designed for Seattle area, WA, zoning seismic design category D, and soil site class C. Plan, building height, and load classes were adopted from SAC steel research project, FEMA-355C (2000) shown in Fig. 6, (a) and (b). All the designed beams and columns rotate in the direction of strong axis and all story floors are considered to be rigid. Gusset plate connections are rectangular and all beam to column connections are shear-tab connections. The elliptical patterns for gussets are shown in Fig. 6, (c).





(b) Elevation of model buildings



(a) plan of model buildings (c) Elliptical model for gusset plate connections

Figure 6. Geometric characteristics of SCBF buildings

All SCBF systems were designed based on AISC seismic provision assuming different values of R equal to 3 to 10 to represent different values for high and low response reduction factor, *R*. In order to give the project symmetry, SAC project has been strengthened in two bays as two-story x-configuration. The effective seismic weight of all model buildings is 4711 and 4525 kilo Newton in the stories and roof, respectively. All connections have been designed according to balanced design procedure, elliptical and linear clearance to balance desired yielding mechanisms as much as possible. For modeling in OpenSEES framework, the braced frame has been analyzed in east-west direction. Table 1 gives the resulting design member sizes of SCBF designed based on R=6 and their maximum strain range limits which shows the cumulative strain range capacity for each brace.

Table 1. Member sizes for SCBF designed for R=6

Story	Column	Beam	Brace	Max. $\varepsilon_{range,pred}$
1	W14x90	W21x93	HSS 6x6x5/8	0.0555
2	W14x90	W21x93	HSS 6x6x1/2	0.0508
3	W14x90	W24x104	HSS 5x5x1/2	0.0532

For performing Displacement-Based cyclic analyses, lateral load distribution of each frame is based on the first dominated mode and in all the analyses, it was assumed that the first mode shape remains constant after yielding of the braced frame and the analyses will continue until 15% degradation of maximum strength capacity (FEMA440A, 2009). It is plainly clear that after yielding of the braced frames, this assumption is an approximation. However, studies conducted by researchers in recent years have shown good estimations and displacements. The conceptual diagram of the recommended loading history represented by FEMA461 (2007) is shown in Fig. 7.

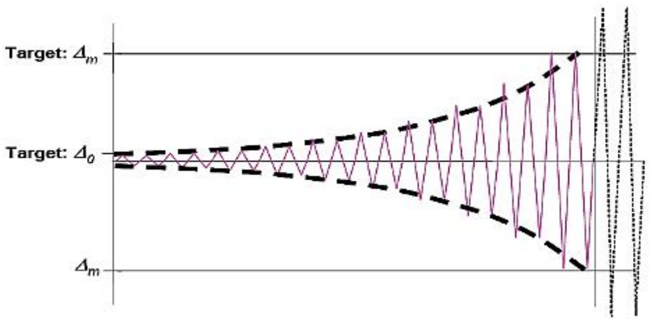
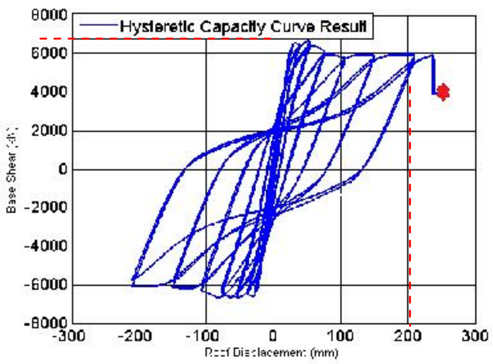


Figure 7. Sketch of displacement-controlled loading history [19]

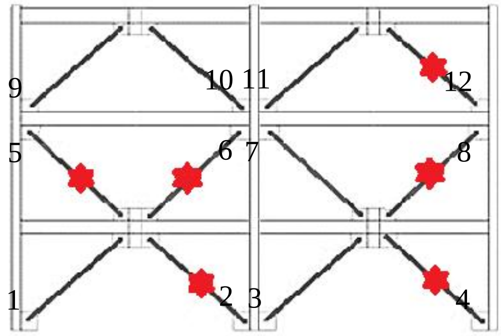
The loading history consists of repeated cycles of step-wise increasing displacement amplitudes. Two cycles shall be completed at each amplitude and the predefined load pattern was considered constant based on the first mode of structures in each cycle. In Fig. 7, Δ_0 and Δ_m are the targeted smallest and maximum deformation amplitude of the loading history. During analysis, the braced frame shows a self-hardening behavior after reaching the first yielding stage and will continue until its maximum strength capacity. Since there is considerable lateral capacity after the first failure of the brace, it is not conservative enough to consider the first failure as the general deterioration of frame. Afterward, as the analysis continues and bracing members deteriorate one after another, the overall maximum strength of the braced frames is reduced to 15% of maximum strength degradation and at that moment instability or incipient mechanism occurs.

EVALUATION AND INTERPRETATION OF RESULTS

As previously mentioned, the goal of this article is the estimation of maximum lateral capacity such as maximum roof drift, inter story drift, base shear, frame fracture-index and failure contribution of each story in overall deterioration of considered braced frames. For example, Fig. 8, (a) illustrates the capacity curve which shows the incipient capacity point for a SCBF designed according to reduction factor of R which is equal to 6. Given that each bracing member has a potential maximum strain range limit capacity, if maximum value of strain range exceeds the potential maximum strain range limit predicted by Eq. (1) during analysis runtime, it means that fracture has occurred. Typical strain history of the fibers at midspan of all the bracing members have been shown in Fig. 9 and brace fracture location upon reaching incipient collapse has been shown in Fig. 8, (b) schematically.

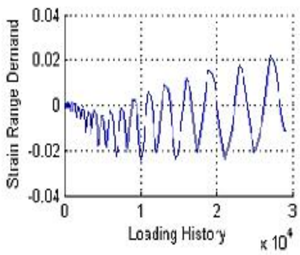


(a) Roof drift-base shear capacity curve

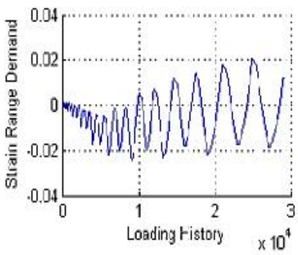


(b) brace fracture location

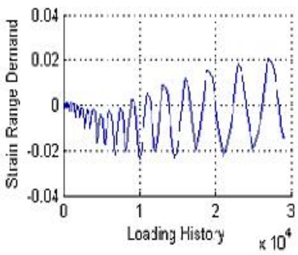
Figure 8. sketch of hysteresis behavior and fracture location of SCBF designed R=6



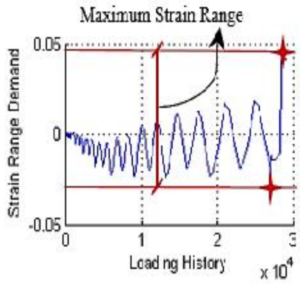
1. Typical strain history for brace No 9



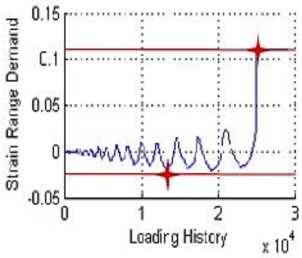
2. Typical strain history for brace No 10



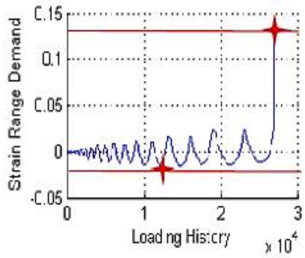
3. Typical strain history for brace No 11



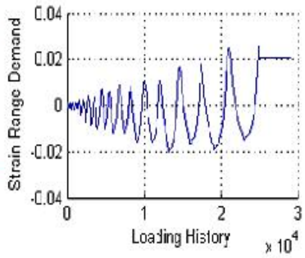
4. Typical strain history for brace No 12



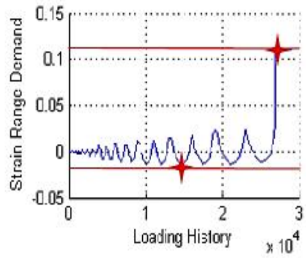
5. Typical strain history for brace No 5



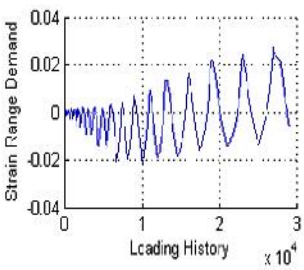
6. Typical strain history for brace No 6



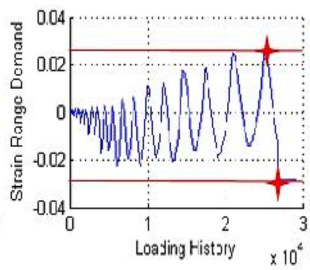
7. Typical strain history for brace No 7



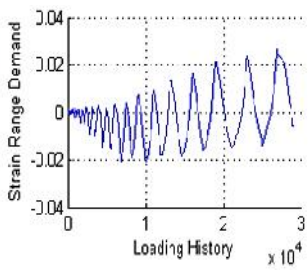
8. Typical strain history for brace No 8



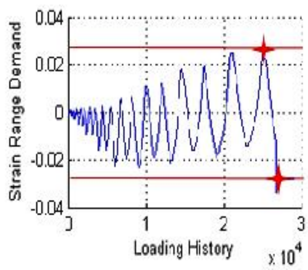
9. Typical strain history for brace No 1



10. Typical strain history for brace No 2



11. Typical strain history for brace No 3



12. Typical strain history for brace No 4

Figure 9. cumulative strain changes at the extreme fiber on the compression side of the hinge in the middle

Since failure in the brace fracture can be a desirable state, this parameter is defined as a ratio which is brace fracture normalized on the whole number of brace members. The purpose of this indicator is to assess the quantity of brace fracture occurrence in SCBFs. With designing all the gusset plate connections based on balanced design procedure, the assumption is made that none of the connections will fail before the bracing fracture. Post processing the results show that capacity results of roof drift, interstory drift, base shear and frame fracture-index are 2.11%, 2.47%, 0.61w (per unit building weight) and 50%, respectively for SCBF designed for R equal to 6 reduction factor. Table 2, shows the scope of the variety of the studied parameters and median of results including roof drift, interstory drift, base shear and frame fracture-index for all considered SCBFs. Due to the importance of brace fracture in incipient collapse, the scope of changes of contribution of deterioration of each story level for brace fracture measurement for considered braced frames including median of their results are shown in Table 3.

Table 2. Scope of the changes in lateral capacity for considered SCBFs

Studied parameters	Scope of the changes	Median results
Roof drift	0.64% ~ 2.47%	1.83%
Interstory drift	1.26% ~ 2.86%	2.23%
Base shear	0.35w ~ 0.88w	0.55w
Frame fracture-index	8% ~ 42%	25%

Table 3. Contribution rate of brace fracture in each story level

Story level	Scope of the changes in Percentage contribution of brace fracture	Median results
Story three	10% ~ 65%	20%
Story two	12% ~ 70%	29%
Story one	18% ~ 80%	33%

Fig. 9 shows that the strain accumulation of each brace member increases with the growth of lateral displacements in cyclic loading history. For example, compression and tension values of strain range in extreme fiber on the compression side of buckling area in the middle of brace member has increased during analysis runtime with large deformation. Bracing members (number 2, 4, 5, 6, 8 and 12) have deteriorated in incipient 15% maximum capacity degradation. Fig. 8 (b) indicates that half of all the brace fracture has occurred in the second story and also frame fracture-index is equal to 50%. The failure of the considered two-bay three-story of SCBF is somewhat asymmetric despite their geometry symmetry because of nonlinear inelastic behavior of members which mainly lies in the brace members due to the large deformations. As shown in Table 2, median lateral capacity results of eight considered SCBFs for roof drift, interstory drift, base shear and frame fracture-index are 1.83%, 2.23%, 0.55w and 25%, respectively. Based on the regulation permissible limit in American seismic provisions for SCBF systems, median results of roof drift has been less than 2% and median results of interstory drift has been more than 2%. Regardless of the design earthquake and response reduction factor, median value of base shear capacity results per weight unit is estimated to be 0.55 w.

Moreover, the results of the analysis show that the median value of the fracture of the braces has been estimated to be 25% of all the brace members. In addition to frame fracture-index calculated for all considered SCBFs, for a given braced frame, the median percentage of brace fracture contribution relative to the total brace fracture as shown in table 3, is 33%, 39% and 20%, for the first, second, and third stories, respectively. The results obtained from nonlinear static cyclic analyses conducted on the studied SCBF buildings can begin moving design toward ductility-based or performance-based design.

CONCLUSION

The goal of this article was to estimate seismic lateral capacity of SCBFs using displacement-based nonlinear static cyclic pushover analysis by considering hysteresis behavior of gusset plate connection, shear-tab connection and an appropriate deterioration criterion for brace fracture more accurately. For this purpose, the capacity of seismic parameters such as roof drift, interstory drift, base shear, frame fracture-index and the contribution of each story in frame fracture-index in incipient collapse at 15% overall degradation of capacity curve were determined. In this regard, at the beginning, for validation of hysteresis behaviour of gusset plate and shear-tab connections, a three-story one-bay tested in NCREE in Taiwan was particularly used. In addition, for validation of hysteresis behaviour of brace deterioration beyond brace fracture, a one-story one-bay tested in University of Washington was used. To identify the bracing fracture development, the maximum strain range limit has been monitored. In the second place, due to the lack of the fracture expression implementation in the OpenSEES framework based on maximum strain range limit, fracture material model was imported in the program with the ability of being open-source in OpenSEES. To perform the analysis, in order to have a thorough investigation of frame regularity, eight SCBFs were designed based on AISC seismic provision to represent different values for response reduction factor, R with identical plans and elevations. All the connections were designed according to a balanced design approach to balance desired yielding mechanisms as much as possible. Interpretations and conclusions can be inferred from the following results:

Since the buildings were designed based on seismic design provisions of steel structures despite the restriction of 2% with the same height, plan and seismic weight, median capacity results of roof drift for all considered SCBFs were equal to 1.83% which is less than permissible range of design codes while median capacity results of interstory drift were equal to 2.23% which is greater than permissible range of design codes which has a more potential to withstand lateral displacements to onset of incipient collapse. Moreover, median capacity results of base shear for all studied braced frames irrespective of response reduction factor, R and design earthquake, were equivalent to 55% per unit weight, in the sense that energy dissipation was also well done against the lateral displacements in these types of resistant systems for lateral loads. Furthermore, median results of damaged braces in terms of frame fracture-index were equal to 25% which means that 25% of the whole braces have been deteriorated. It was also observed that 33%, 29% and 20% of the whole brace fracture occurred in the first, second and third stories, respectively. Based on the results, other kinds of performance based seismic design problems can be solved in future studies. The results related to SCBF designed with R=6 showed that fifty percent of all the braces have failed at incipient collapse. This indicates that damage redistribution of brace fracture has been done well in this type of structural system. Due to the fact that nonlinear inelastic behavior of beams, columns and braces which mainly lie in the brace members due to the large deformations, the failure of this braced frame despite geometry symmetry has been somewhat asymmetric in both bays.

Finally, it is important to note that more and more extensive studies need to be conducted on a different number of stories and bays of SCBFs in order to universalize the results of this case study for all special concentrically two-story x-braced frames.

REFERENCES

AISC (2010) Seismic Provisions for Structural Steel Buildings, *American Institute of Steel Construction*, Chicago, IL

FEMA 355C (2000) State of the art report on systems performance of steel moment frames subjected to earthquake ground shaking, Federal Emergency Management Agency, Washington, DC

FEMA 461 (2007) Interim Testing Protocols for Determining the Seismic Performance Characteristics of Structural and Nonstructural Components, Federal Emergency Management Agency, Washington, DC

FEMA P440A (2009) Effects of Strength and Stiffness Degradation on Seismic Response, Federal Emergency Management Agency, Washington, DC

Han SW and Chopra AK (2006) Approximate incremental dynamic analysis using the modal pushover analysis procedure, *Earthquake Engineering and Structural Dynamics*, 65: 1853–1873

Hsiao PC, Lehman DE and Roeder CW (2012) A Model to Simulate Special Concentrically Braced Frames Beyond Brace Fracture, *Earthquake Engineering and Structural Dynamics*, 42: 183–200

Kotulka BA (2007) Analysis for a design guide on gusset plates used in special concentrically braced frames, M.S thesis, University of Washington, Seattle

Lehman DE and Roeder CW (2008) Improved seismic design of concentrically braced frames and gusset plate connections, *ASCE*

Lumpkin EJ (2009) Enhanced seismic performance of multi-Story special concentrically braced frames using a balanced design, M.S thesis, University of Washington, Seattle

NEHRP (2013) Seismic Design of Steel Special Concentrically Braced Frame Systems, NEHRP Seismic Design Technical Brief No 8

Priestley MJ (1997) Displacement-based seismic assessment of reinforced concrete buildings. *Journal of Earthquake Engineering*, 1(1): 157-192