Canadian Conference - Pacific Conference on Earthquake Engineering 2023

Vancouver, British Columbia

June 25th – June 30th, 2023

Influence of Connection Hysteretic Parameters on The Seismic Performance of Conventional Construction Steel Concentrically Braced Frames

Anirban Kundu[1*], Lydell Wiebe[2] and Georgios Balomenos[3]

[1]PhD Student, Department of Civil Engineering, McMaster University, Canada.
[2]Endowed Chair in Effective Design of Structures, Department of Civil Engineering, McMaster University, Canada.
[3]Assistant Professor, Department of Civil Engineering, McMaster University, Canada.
[*]kundua6@mcmaster.ca (Corresponding Author)

ABSTRACT

Steel concentrically braced frames (CBFs) are a widely used seismic force resisting system (SFRS) in Canada and can be designed with a variety of ductility-related reduction factors and associated detailing requirements. One of these options in CSA S16:19 is conventional construction (CC), which is less prescriptive than Type-MD (moderately ductile) CBFs. Type-CC CBFs are designed for a higher base shear and do not require strict adherence to capacity design principles, which fundamentally leads to a lack of enforced hierarchy in inelastic behaviour or failure of the elements such as braces, beams, columns and connections. To discourage premature failure in the connection, the provisions for Type-CC require connections to be designed either as ductile or for an amplified force demand, intending to promote elastic behaviour. A wide range of brace connections are possible consistent with these requirements, creating a challenge for studying the seismic behaviour of Type-CC CBFs as a single class of SFRS. Due to this large variability, it is necessary to study the effect of the features of a connection hysteresis on the global seismic response of Type-CC CBFs, and particularly on the possible range of deformation demand at the connections. In this study, a single-storey CBF is designed according to the provisions for Type-CC systems in CSA S16:19 and is modelled in OpenSees. The connections are parameterised considering a few factors, including initial stiffness and strength. Nonlinear response history analysis (NLRHA) is performed for a set of ground motions at the design ground motion (DGM) level. The peak displacement at the floor level as well as the deformation demand at the braces and connections are studied to answer the aforementioned questions.

Keywords: Seismic performance, steel structures, concentrically braced frames, conventional construction, ductility demand, connection hysteresis.

INTRODUCTION

Steel concentrically braced frames (CBFs) are a widely used seismic force resisting systems (SFRS), in which the lateral load is resisted by truss action of the braces, beams and columns. High-ductility CBFs (e.g., Type-MD in CSA S16:19 [1]) are designed for relatively lower strength but with a rigorous design process to ensure a hierarchy of inelastic behaviour. Type-MD CBF design follows the principle of capacity design where during severe and infrequent earthquakes, braces act as a fuse by yielding in tension and buckling in compression, thereby safeguarding the other capacity protected members. Alternatively, a CBF can also be designed with a lower ductility related reduction factor e.g., Type-CC CBFs in CSA S16:19, which have less restrictive design rules and thus are designed for higher strength in exchange. In this case, all the members are designed for equal force demand, so there is no hierarchy of inelastic behaviour. Due to this difference in design philosophy, the interaction between the brace and its connection to the framing element is affected significantly.

In Type-MD CBF design, the concept of 'weak brace strong connection' is imposed through capacity design provisions, where the brace is designed to dissipate all the seismic energy by yielding and buckling while the connection is meant to stay elastic as a capacity protected element. Conversely, in Type-CC design, the connections are not rigorously capacity protected, and which could lead to a premature failure in a brittle connection, or it could lead to a ductile connection dissipating seismic energy through inelastic behaviour [2]. The latter may even be beneficial, contributing significantly towards the total deformation capacity of the brace module [3,4]. In this regard, experimental tests of six full-scale I-shaped brace specimens with bolted end connections designed according to Type-CC [3] achieved storey drift ratios of 1%–2%, even though capacity design provisions were not incorporated in the design. Significant contributions to ductility capacity were noticed from yielding and buckling of the gusset plate and lap plate, together with bolt bearing, and friction associated with bolt slip.

In Type-CC CBFs, to avoid failure in connection, CSA S16:19 [1] requires that the connections be proportioned in such a way that the failure mode is ductile or designed with an amplified brace force demand. However, the connection ductility demand is not quantified nor is the way to achieve it. Due to the lack of restrictions in design of the members and connections joining the brace to the framing members in Type-CC CBF, a wide variety of members and connection designs is possible [2,3,5–7]. This underlying variability in the design of connection leads to various possible force-deformation behaviours of the connection in its axial direction (e.g., initial and post yield stiffness, ductility) as well as different failure modes (e.g., block shear, end-tear-out, bearing failure etc.) even for similar ultimate strength of the connection [5,8,9]. This makes it necessary to quantify the deformation demand that is expected in a seismic event from a connection in a Type-CC CBF, as well as the effect of the connection axial hysteresis parameters on the global response of the Type-CC CBF.

To study the effect of the connection axial force-deformation hysteresis, two connection modelling techniques are adopted from literature: the component-based model (CBM) [4] and the *Pinching4* uniaxial material-based model [5]. The CBM provides a framework to model the response of a bolted connection by modelling each component e.g., plate bearing and yielding, bearing and shear deformation of bolt, rather than capturing the full range of behaviour with a single general model. This idea is suitable for modelling a designed connection but not for parameterising the hysteretic behaviour of a generalised the connection (e.g., stiffness and strength) which can be done using a simplified version of the CBM or using the *Pinching4* model. This paper investigates the merit and demerits of the two modelling approaches and the influence of the connection hysteretic parameters on the global response of the structure. In the current study, a one storey building is designed according to the provision of Type-CC in CSA S16:19 [1] and modelled in OpenSees [10]. The brace end connection is modelled using a single or system of multiple nonlinear springs and are validated. The parametric study is conducted by varying the design strengths to account the effect of CSA S16:19 specified force amplification factor and the stiffness. Seismic performance is evaluated by conducting a series of NLRHA for an ensemble of ground motion at the DGM level and the statistical tendency is explored.

CONNECTION MODELLING APPROACHES

Validation of simplified component-based model & Pinching4 model

To facilitate the parameterisation of the brace-to-frame gusset connection, as shown in the Figure 4, two modelling techniques (simplified-CBM and *Pinching4* model) are adopted, which are then validated against a brace test result (J310-T) by Rudman et al. [3] (schematic is shown in Figure 1 (a)). Figure 1 (b) shows the hysteretic behaviour of the connection of one side of the brace module modelled using a CBM following the recommendation Wang et al. [4]. Figure 1 (e) shows the good agreement between the overall force-deformation hysteresis of the brace module obtained from the experiment and the numerical model.

Based on the CBM, a simplified model is developed which also captures the same experimental result well (Figure 1 (c) and (f)). In a bolted gusset plate connection, typically observed events are bolt bearing at the two connected plates (gusset and the splice plate), bearing and shearing of the bolt shaft, yielding of the plate, etc. Rudman et al. [3] reported a frictional force between the two bearing plates that averaged 17 % of the connection design strength and persisted through the seismic event. So, in the simplified model the bolt slip and frictional behaviour between the plates are modelled using the same approach as in the CBM. Other behaviours (i.e., bearing of plates, bolts, etc.) which were modelled using multiple springs in CBM are simplified by a single “elastic perfectly plastic gap” (*ElasticPPGap*) spring in OpenSees. The initial stiffness is calculated such that the yield displacement occurs at 4 mm, and the yield strength is 80 % of the ultimate strength. The post yield stiffness is calculated such that the ultimate strength is achieved at a displacement of 15 mm, where the ultimate strength is obtained from the connection designed by Rudman et al. [3]. The shape of the backbone is based on the behaviour in the previous experiments on bolted connection [5,8]. A good agreement in the test and numerical model force-deformation hysteresis confirms the usability of simplified-CBM as a benchmark for the future analysis in the current study.

Next, another simplified approach using the *Pinching4* uniaxial material is adopted, as implemented previously by Castonguay [5] when defining the hysteretic backbone, the deformation at the yield and ultimate load is kept the same as in the previous case. Here the bolt slip and friction phenomenon cannot be captured explicitly but the first point on the backbone hysteresis is at a displacement of 0.4 mm and at the frictional force reported (i.e., 17% of the ultimate strength), after which the stiffness is reduced so that it reaches the yield strength at the yield deformation mentioned earlier. A typical case of such model is Pinch-502 in Figure 2. Like the two previous modelling techniques, Figure 1 (d) and (g) shows a good agreement between the force-deformation hysteresis obtained from experimental test result and numerical model.

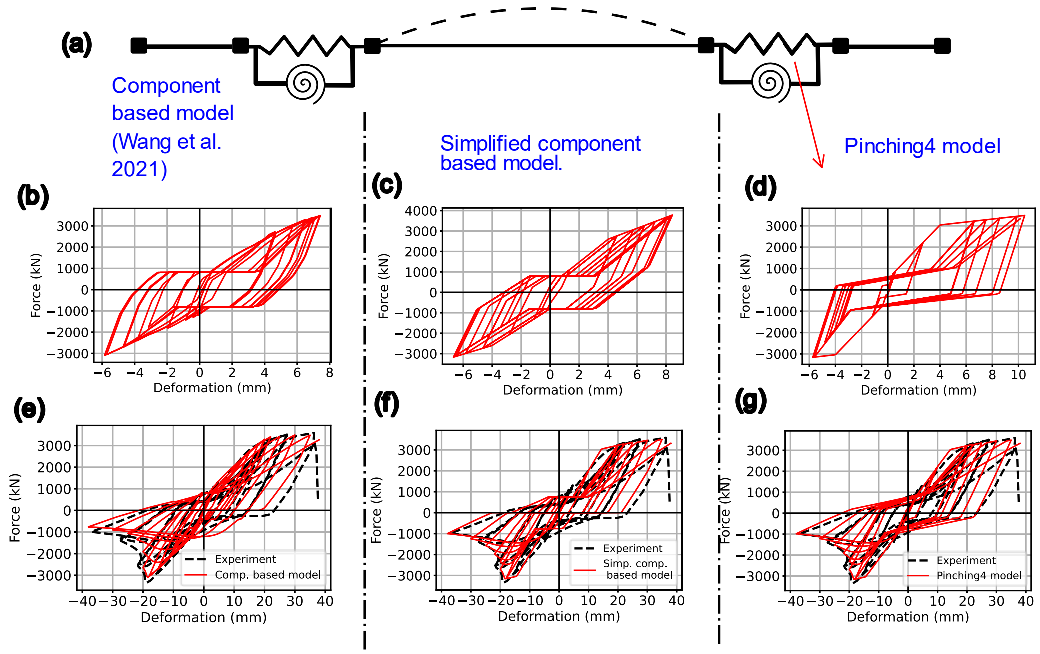

Figure 1. (a) Schematic of the brace test. (b-g) Validation of the CBM, simplified-CBM, and the Pinching4 material based model against the brace-test results by Rudman et al. [3]*. (b-d) shows the connection behaviour. (e-g) shows the overall brace behaviour.*

Parameterizing the connection

Connection stiffness and modelling approaches

Experiments done on different types of bolted connections show that both monotonic and cyclic force-deformation characteristics vary greatly based on the type of detailing. The strength, initial stiffness, ductility and failure mode could be affected by design parameters such as gusset and splice plate thickness, bolt strength, diameter, clearance to the bolt hole, and orientation [2,3,5–8]. To reflect this, 5 parametric connection cases are defined (Figure 2) based on a design strength of the connection, where the first case is modelled using simplified-CBM approach and Pinch-500 to Pinch-503 are modelled using the *Pinching4* material. As shown in Figure 2, Pinch-500 has a very high initial stiffness and the yield strength is equal to the ultimate strength, which is considered as the design strength of the connection. Pinch-501 is defined with a reduced post slip stiffness so that the yield strength (set equal to ultimate strength) reaches at 4 mm. Similarly, the Pinch-502 and Pinch-503 are defined but the yield strength is set to 80% and 60% of the ultimate strength, respectively.

Connection design strength

According to CSA S16:19 [1], a ductile connection then it can be designed with the brace force demand, which in the current study is 1EQ+1DL+0.5LL+0.25SL, where EQ is the Earthquake Load, DL is the Dead Load, LL is the Live Load, and SL is the Snow Load effect. Otherwise, the connection should be designed by amplifying the EQ load effect by 50 %, together with the same gravity load effect (1.5EQ+1DL+0.5LL+0.25SL). So, to study the effect of this, the amplification of the connection strength demand is varied from 0% to 50% with a 10% interval. Also, a capacity protected connection design is simulated by introducing a rigid elastic spring. From this point onward, the amplified connection force demand with which the connection is designed is referred to 'strength' followed by the amplification factor e.g., if the connection design force demand is 110 % of the brace force demand, then it is referred as 'strength 110'.

CASE STUDY STRUCTURES

Archetype building design

Figure 3 shows the plan and elevation of the 1-storey office building considered in this study, located in Victoria, BC, and having a total height of 6 m, with a plan dimension of 60 m by 36 m and bay lengths of 6 m in each direction. The CBF in E-W direction is designed following the equivalent lateral force procedure of the National Building Code (NBC) of Canada [11]. The gravity loads are shown in Table 1. Site Class C is considered with an importance factor of 1. The design spectrum is calculated for a 2% probability of exceedance in 50 years, noted as the design ground motion (DGM) [11]. The calculated base shear is 2929 kN with a fundamental period of 0.2 sec, considering R_d and R_o of 1.5 and 1.3 respectively, and the design complies with the provisions for conventional construction in CSA S16:19 [1]. For better comparison between the design and numerical simulation, the resistance factors are considered as 1 and the notional load and accidental torsion are neglected in the design phase. Unlike Type-MD design, in Type-CC design, there is no restriction on global slenderness ratio and width-to-thickness ratio of the HSS brace design. However, the brace members selected in the case study building followed the restriction imposed by the Type-MD design. The base of the column is assumed to be pinned and are designed for the axial force demand only.

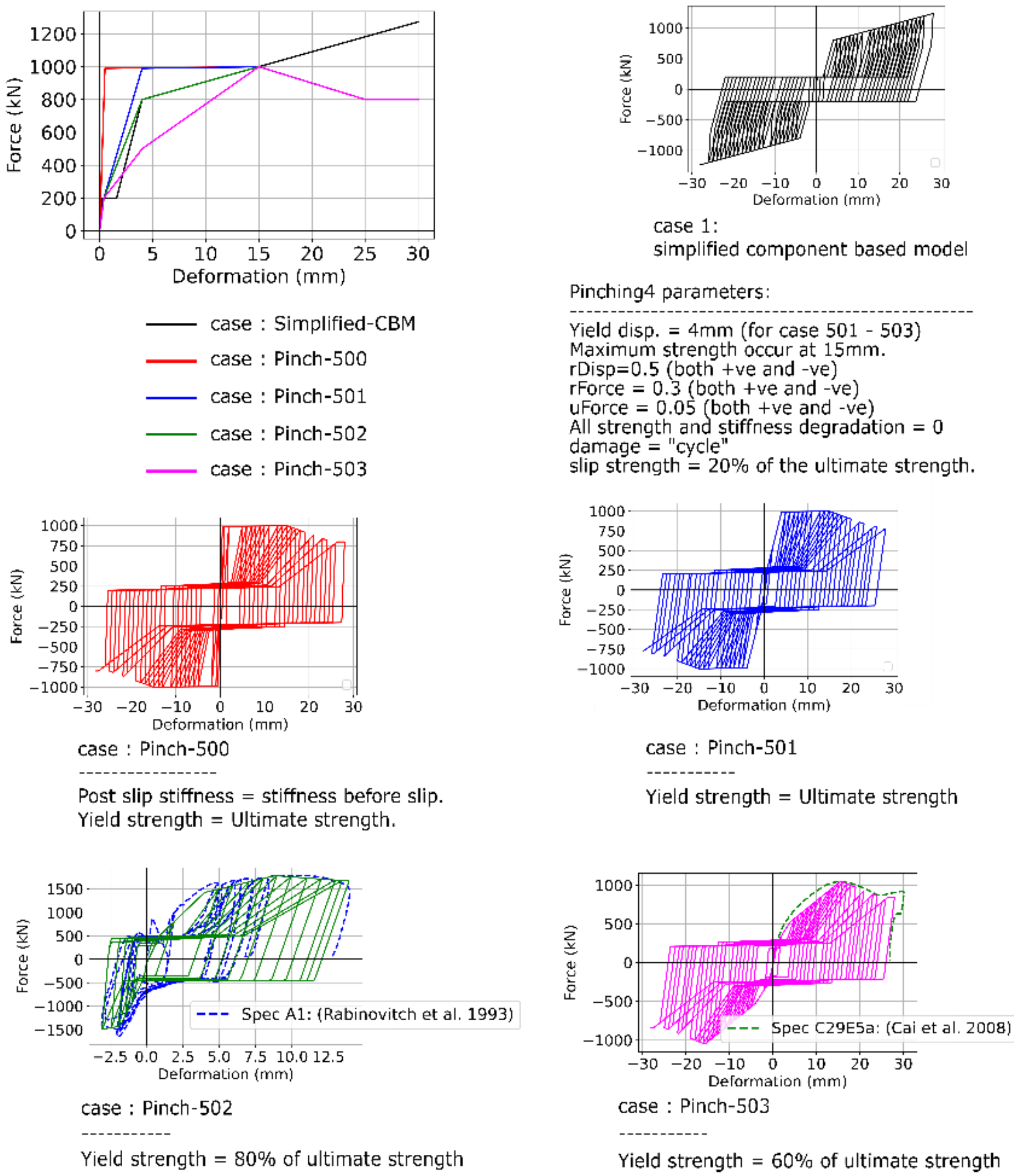

Figure 2. Different connection cases for the parametric study.

Numerical modelling

A 3D model of the CBF is developed in OpenSees [10], schematic of which is shown in Figure 4 where distributed-plasticity based beam–column elements are used for defining the braces, beams, and columns and concentrated-plasticity based zero-length elements are used to define the gusset plate, beam to column shear tab connections. To incorporate the out of plane buckling in the braces, 16 nonlinear displacement-based beam–column elements with 4 integration points are used along the length of the brace with an imperfection of 1/500 of the effective length, in a sinusoidal shape. The cross section of the HSS brace is discretized according to Hsiao et al. [12]. Also, post-buckling fracture at the middle of the brace is simulated by the maximum-strain-range model [13]. Force-based nonlinear beam-column elements with 4 integration points are used for beams and columns. A Giuffre–Menegotto–Pinto model (*Steel02* in OpenSees) is used to model each constitutive elements for all members [12,13].

A parameterised axial spring is modelled at the brace end as discussed earlier. An approximate evaluation of the gusset plate thickness and sizing defined the stiffness and strength of the rotational spring as reported by Hsiao et al. [12]. The beam-to-column shear tab connection is modelled using a *Pinching4* material according to Elkady et al. [15]. Seismic masses are applied at the top of each column. Columns are modelled with pinned bases, and an elastic leaning column is incorporated to account for the P-Delta effect. Rayleigh damping of 2 % is assigned to the first two modes.

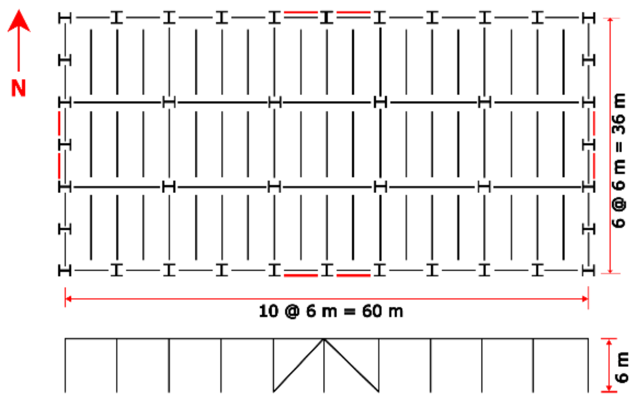

Figure 3. Archetype building layout.

Table 1. Design parameters

Load (kPa)				
	Dead	*Live*	*Snow*	*wall*
Roof	1.86	1	1.64	1.5
Designed section				
Beam	*Column*	*Brace*		
W360×33	W130×28	HSS178×178×13		

Ground motion selection and scaling

A set of 41 ground motions are selected from PEER ground motion database, based on a target response spectrum corresponding to a probability of exceedance of 2% in 50 years for Victoria, BC. Ground motions are scaled to the target spectrum at the fundamental period of the structure, 0.2 sec, using only records with a scale factor between 0.5 and 10. The selected records are equally distributed between strike-slip and reverse type faults and have a magnitude range of 5.1 to 7. Figure 5 shows the group of selected spectra and their mean, along with the target design spectrum to which they are scaled. The spectrum for the San Fernando record is shown as a representative ground motion for the examined location.

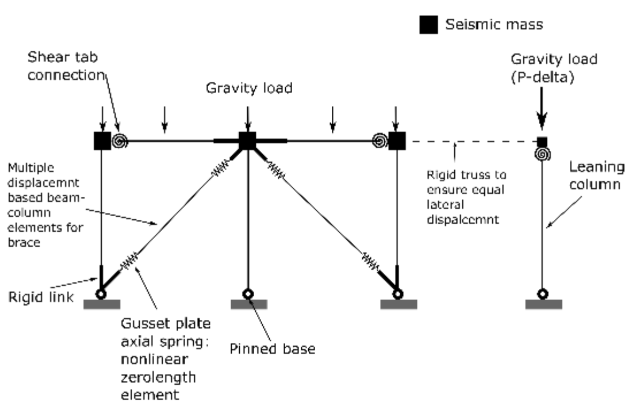

Figure 4. Schematic of the numerical model in OpenSees.

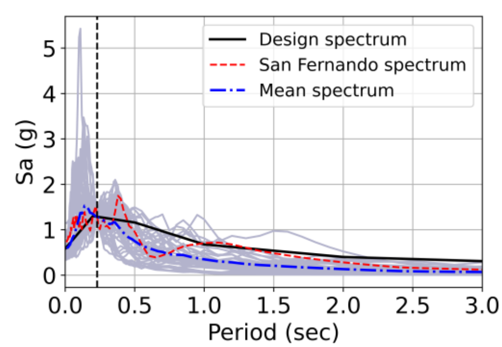

Figure 5. Ground motion scaling.

RESULTS AND DISCUSSION

Figure 6 shows the seismic behaviour of the CBF subjected to the San Fernando ground motion, for connection strength 100 and 150 for a typical connection model, Pinch-502. For both the examined connection strengths, the displacement time history of the first floor (Figure 6 (a) and (e)), the connection force-deformation hysteresis of the lower connection of the left brace (Figure 6 (b) and (f)) and the brace behaviour (Figure 6 (c) and (g)) are presented. Then the contribution of the connection and the brace itself towards the displacement of the total brace module is shown (Figure 6 (d) and (h)).

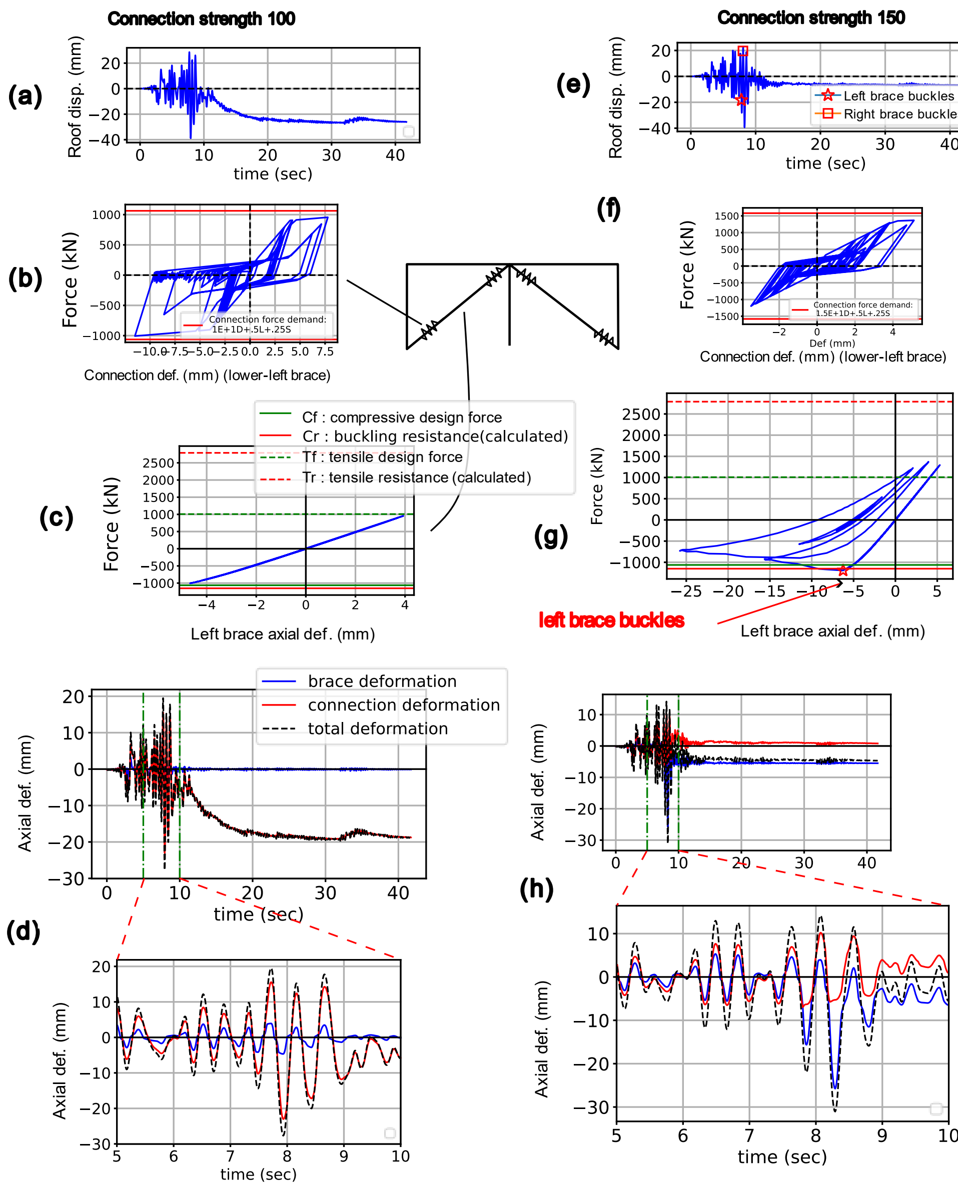

Figure 6. Behaviour for the CBF braces with connection strength-100 and case: Pinch-502. (a) displacement time history at the first floor, (b) connection force deformation hysteresis, (c) force deformation hysteresis of the left brace, (d) contribution of the brace and connection deformation towards the total deformation. (e-h) The same sequence follows for the strength 150.

The connection behaved inelastically in both the cases. In case of connection strength 100, the major contribution towards the total deformation of the brace module comes from the connection, while the braces remain in the elastic range. Conversely for the connection strength 150, braces buckled and the deformation in the brace is considerably higher than in the connection. The amplified connection design force demand succeeded in increasing the inelastic behaviour of the braces while decreasing the connection deformation demand.

Response to a single earthquake

Modelling approaches

Figure 7 compares the displacement time history resulting from the San Fernando ground motion record for the connection strengths 100 and 150. The simplified-CBM model always resulted in zero residual displacements whereas *Pinching4* material-based models resulted in a similar non-zero absolute value of residual displacement, irrespective of the initial stiffness and strength. However, the peak response from the two different modelling approaches (i.e., Simplified-CBM and *Pinching4*) is within 6.9 %. The peak response from all the connection models is comparable specially from simplified-CBM, Pinch-501 and

Pinch-502 are similar for the connection strength 100 because all three connections have similar initial stiffness. This suggests that the friction and bolt slip behaviour which is simulated explicitly in simplified-CBM is insignificant for the connection which is designed with minimum amplified force demand. Peak displacement is slightly higher using the simplified-CBM compared to the equivalent *Pinching4* model, which is Pinch-502 in this case (similar initial stiffness).

As shown in Figure 6(b), in the *Pinching4* material-based model, once the peak of the ground motion acceleration completed, due to small force demand cycles and the intrinsic law of unloading-reloading stiffness, it starts producing a permanent residual displacement at each small cycle which causes the residual displacement at the system level. So, it is appropriate to study the peak response of the building but not the residual response. This residual displacement produced by *Pinching4* model is relatively lower in case of higher connection strengths compared to the lower strengths (Figure 7). This is attributed to the simultaneous braces buckling and inelastic behaviour in the stronger connections at the peak ground acceleration. Once the brace buckled, the deformation demand is taken by the brace instead of the connection. Note, braces remain elastic where connections are designed with equal brace force demand (e.g., strength 100) in which case *Pinching4* model demonstrated aforementioned modelling deficiency, unlike the connection designed with largely amplified force demand (e.g., strength 150).

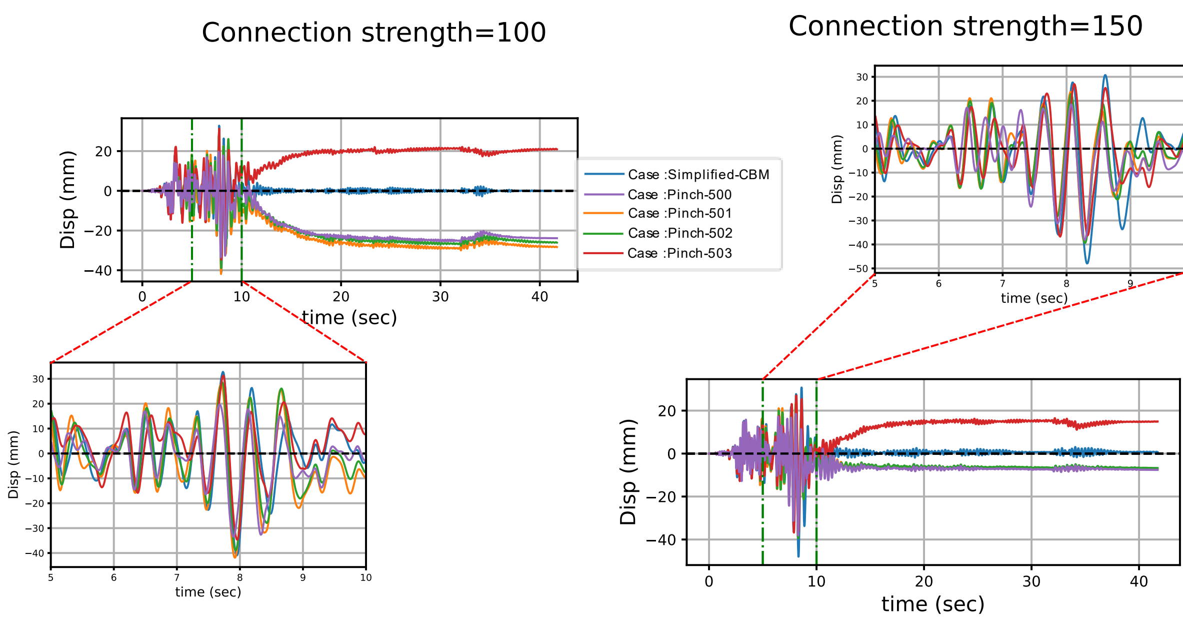

Figure 7. Comparison of the different models for connection strength 100 and 150.

Connection strengths considering modelling approaches

In Figure 7, for connection strength 100, the peak displacement is the same for Pinch-501 and Pinch-502 which indicates that for a connection designed with an unamplified brace force demand, the modelled yield strength has only a minor influence on the peak response provided that the initial stiffness is similar.

Pinch-500 and Pinch-501 have the same yield strength but different initial stiffness. At the strength 100, Pinch-501 produces higher floor displacement than Pinch-500, and this difference reduces as the strength increases. At strengths 140 and 150 the peak displacement from these two cases is similar. Thus, the influence of the initial stiffness is insignificant for the connections designed with higher amplified force demand, given the same yield strength.

Figure 8 shows the distribution of the brace and the total connection deformation demand of the left brace module for San Fernando record at the DGM level. For connections designed near the brace force demand (e.g., strength 100 or 110), the following is observed:

- The deformation demand in the connection from different models are similar.
- The brace stays elastic and the major deformation demand in the brace module comes from the connection.

With an increase in the connection design force, the deformation demand on the connection decreases while the deformation of the complete brace module and of the brace itself increases. In general, for every connection design strength, the overall deformation demand decreases as the connection stiffness decreases (from Pinch-500 to 503).

For connection strengths of 130 and higher:

- The connection deformation demand is less, and the brace contributes more towards the total deformation.

Connection with lowest stiffness and yield strength (i.e., Pinch-503) behaved inelastically irrespective of all the design force amplification factor and allowed the brace to remain elastic.

Response to the group of earthquakes

Figure 9 shows the average deformation demand of the lower connection at the left brace, for the ensemble of 41 ground motions at the DGM level. The deformation demand decreases with the increase of the amplification factor in the connection design demand. The deformation demand in the connection increases with a decrease of the initial stiffness (from Pinch-500 to 503). The connection having the highest initial stiffness and yield strength has the lowest deformation demand, whereas the connection having the lowest initial stiffness and yield strength has the highest deformation demand. Note, two different modelling approaches, Simplified-CBM and *Pinching4* material-based model Pinch-502 results into similar peak connection deformation demand (Figure 8 and Figure 9).

The difference in connection deformation demands due to varying initial stiffness and yield strength is more significant for the connections designed with a higher force demand. Considering the mean plus standard deviation, the deformation demand in the connection is around 10 mm to 13 mm if it is designed based on 100%-130% of the brace force demand and typically less than 11 mm if the connection is designed for 140%-150% of the brace force demand.

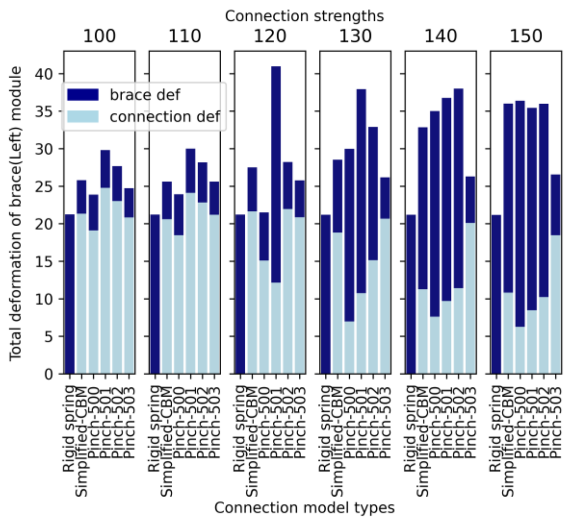

Figure 8. Contribution of the brace and connection towards the maximum deformation demand in the left brace module over different connection cases and strengths.

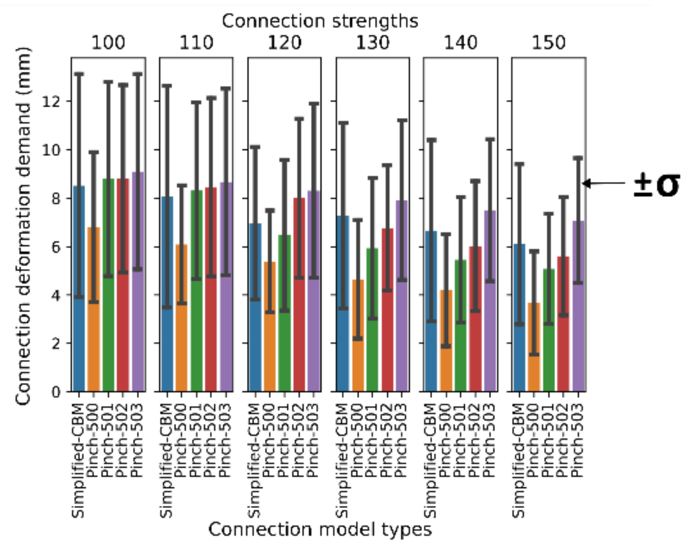

Figure 9. Deformation demand of the lower left brace connection.

CONCLUSION

The influence of the features of a connection hysteresis such as strength and stiffness on the global seismic response of Type-CC CBF is studied in this paper. A single-story Type-CC CBF system was designed according to the CSA S16:19 and modelled numerically in OpenSees. Connection at the brace end was modelled using nonlinear springs by varying force-deformation hysteretic features and strength. NLRHA was conducted at DGM level and the deformation demand in connection coupled with the brace behaviour was studied. The following conclusions can be drawn from this study:

- Two modelling approaches, *Pinching4* and Simplified-CBM are both appropriate if the peak response is considered. If the residual response of the system is considered, then the Simplified-CBM is a better modelling alternative over the *Pinching4* material-based model.
- For connections designed with equal brace force demand, the peak deformation from different connection models is similar.
- With the increasing amplification in connection force demand:
  - The brace buckling behaviour becomes more prevalent.
  - The deformation demand in the overall brace module increases
  - The connection deformation demand decreases.
- Connections designed with largely amplified (e.g., 50 %) brace force demand tend to remain elastic, leading to brace buckling.
- NLRHA result from a single record and the mean tendency of the ensemble of 41 records at DGM level shows:

- The deformation demand in the connection is within10 mm to 13 mm if it is designed with equal or slightly higher brace force demand. And typically, less than 11 mm for the stronger connections (e.g., strength-140 or150).
- The lowest connection deformation demand comes from the connection with the highest initial stiffness and yield strength.
- Connections with the lowest initial stiffness and yield strength tends to perform equally at all levels of amplified design force demand and give higher deformation demand than other connection cases, regardless of the connection's design strength. This enables braces to stay elastic and results the lowest brace deformation demand.
- The difference in connection deformation demands due to varying initial stiffness and yield strength is more noticeable in the connections designed with force higher than brace force demand.

Further study is needed on the multistorey CBF structures located in different seismic zones and soil classes. Also, the effect of connection deformation demand and hysteretic features considering various commonly used brace sections in Type-CC designs needs to be investigated.

ACKNOWLEDGEMENT

The authors gratefully acknowledge the support of this study from Discovery Grant funding provided by the Natural Sciences and Engineering Research Council (NSERC) of Canada. Any opinions, findings, and conclusions or recommendations expressed in this material are those of the authors and do not necessarily reflect the views of the sponsors.

REFERENCES

[1] CSA S16:19 12th Edition, Design of steel structures, Canadian Institute of Steel Construction, 2019.

[2] J.S. Rabinovitch, J.J.R. Cheng, Cyclic behavior of steel gusset plate connections, Structural Engineering Report No. 191. University of Alberta, 1993.

[3] A. Rudman, R. Tremblay, C.A. Rogers, Conventional I-shape brace member bolted connections under seismic loading: Laboratory study, J Constr Steel Res. 184 (2021).

[4] C. Wang, R. Tremblay, C.A. Rogers, Component-based model for bolted brace connections in conventional concentrically braced frames, Eng Struct. 247 (2021).

[5] P.X. Castonguay, Seismic performance of concentrically braced steel frame of the conventional construction category, Thesis, Master of Applied Sciences, Dept. of Civil Engineering, Ecole Polytechnique De Montreal, 2009.

[6] H. Bino, G.Y. Grondin, R.G. Driver, Block shear behaviour of bolted gusset plates., Structural Engineering Report No. 248. University of Alberta, 2002.

[7] M.C.H. Yam, J.J.R. Cheng, Experimental investigation of the compressive behavior of gusset plate connections, Structural Engineering Report No. 194. University of Alberta., 1993.

[8] Q. Cai, R.G. Driver, End tear-out failures of bolted tension members, Structural Engineering Report No. 278, University of Alberta, 2008.

[9] Y. Cai, B. Young, Effects of end distance on thin sheet steel bolted connections, Eng Struct. 196 (2019).

[10] F.T. McKenna, Object-oriented finite element programming framework for analysis, algorithm and parallel computing., PhD thesis, Dept. of Civil Engineering, University of California, Berkeley, 1997.

[11] NBC, National Building Code of Canada, National Research Council of Canada. Canadian Commision on Building and Fire Codes. (2015).

[12] P.C. Hsiao, D.E. Lehman, C.W. Roeder, Improved analytical model for special concentrically braced frames, J Constr Steel Res. 73 (2012) 80–94.

[13] P.Chien. Hsiao, D.E. Lehman, C.W. Roeder, A model to simulate special concentrically braced frames beyond brace fracture, Earthq Eng Struct Dyn. 42 (2013) 1–6.

[14] A.D. Sen, C.W. Roeder, D.E. Lehman, J.W. Berman, Nonlinear modeling of concentrically braced frames, J Constr Steel Res. 157 (2019) 103–120.

[15] A. Elkady, D.G. Lignos, Effect of gravity framing on the overstrength and collapse capacity of the steel frame building with perimeter special moment frames, Earthq Eng Struct Dyn. 44 (2015) 1289–1307.