NUMERICAL INVESTIGATION INTO I-SHAPE BRACE CONNECTIONS OF CONVENTIONAL CONCENTRICALLY BRACED FRAMES

C. Wang(1), R. Tremblay(2), C.A. Rogers(3)

(1) Ph.D. candidate, Department of Civil Engineering and Applied Mechanics, McGill University, chen.wang5@mcgill.ca
(2) Professor, Department of Civil, Geological and Mining Engineering, Polytechnique Montréal, robert.tremblay@polymtl.ca
(1) Professor, Department of Civil Engineering and Applied Mechanics, McGill University, colin.rogers@mcgill.ca

Abstract

Steel concentrically braced frames (CBFs) are effective seismic force resisting systems which are widely used in North America. In high seismic regions, capacity design principles and numerous additional seismic detailing provisions are explicitly required in design to confine the inelastic behaviour to the bracing members, i.e. yielding in tension and buckling in compression. However, in areas of moderate or low seismicity, the design of CBFs may be based on a conventional approach, in which the principal requirement is for the factored resistance to be equal to or greater than the factored load effect obtained from linear structural analysis; capacity protection is not required. The conventional design approach is predominantly used because of its simplicity in terms of design and fabrication, and therefore economy. Such CBFs are referred to as Conventional CBFs (CCBFs) in this paper. The type 'Conventional Construction' (CC) CBFs in the National Building Code of Canada (NBCC) and CBFs 'not specifically detailed for seismic resistance' in accordance with the American Society of Civil Engineers (ASCE) 7 fall into the CCBF category.

Along the lateral load path in CCBFs, the brace connection is usually the weakest link and may be prone to fracture when subjected to tension. In Canada, due to the lack of data to characterize the brace connection's inelastic seismic performance, the seismic design load for the connections of type CC CBFs is required to be amplified by 1.5 unless ductile behaviour can be guaranteed. However, no explicit demand, nor any design provisions regarding how to obtain ductile behaviour, are readily available since little research has been done on this issue. Hence, an extensive experimental and numerical research project regarding the I-shape brace connection design in CCBFs has been launched at Polytechnique Montréal and McGill University.

In this paper, a numerical simulation procedure was used to investigate the behaviour of a typical I-shape brace connection configuration, i.e. the flange plate connection. The accuracy of the numerical models was validated through comparison between the simulation results and the laboratory test results. Two force transfer branches (the flange branch and the web branch) within the brace-to-gusset connection were identified. The numerical analysis reveals that the flange branch will develop its capacity prior to the web branch since it needs less bolt slippage to achieve the bearing condition. Regarding the force partition between the two branches, a parametric study was conducted with variation of the flange lap plate thickness and the web lap plate thickness. The results indicate that the force sharing at the ultimate limit state is determined by the ultimate strength of each branch. To avoid the low-ductility bolt shear rupture and weld fracture, the bolts and welds are recommended to be designed based on the ultimate strength of each branch. Another welded web lap plate attachment was also studied, but it did not have an impact on the ultimate strength and force partition within the connection as long as it does not change the failure mode in the web branch.

Keywords: conventional CBFs, I-shape, brace connections, FE simulation, force transfer mechanism

1. Introduction

Concentrically braced frames (CBFs) are commonly used as seismic force resisting systems (SFRSs) in North America due to their efficiency in providing lateral stiffness and strength. In high seismic regions, capacity design principles and numerous additional seismic detailing provisions are explicitly required for CBF design, as found in AISC 341 [1] and CSA S16 [2]. However, in areas of moderate or low seismicity, CBFs based on the conventional design principle (the factored resistance equal to or greater than the factored load effect obtained from linear structural analysis) are exempted from seismic detailing requirements. These CBFs are predominantly used because of their simplicity in terms of design and fabrication, and thereof economy. Such CBFs are referred to as Conventional CBFs (CCBFs) in this paper. The type 'Conventional Construction' (CC) CBFs in the National Building Code of Canada (NBCC) [3] and CBFs 'not specifically detailed for seismic resistance' in accordance with the American Society of Civil Engineers (ASCE) 7 [4] belong to the CCBF category.

Along the lateral load resisting path in CCBFs, the brace-to-gusset connection is usually the weakest link when the brace is working in tension, because both the braces and gusset plates are generally chosen based on the compressive resistance in line with the conventional design principle. Because CBFs may exhibit low redundancy, this weakest link could be overstressed under strong ground shakings, and hence undergo substantial inelastic deformation. Premature brittle failure may occur if the brace-to-gusset connection is not properly detailed, e.g. premature fracture of HSS-to-gusset welds, which has been witnessed in both laboratory tests [5] and post-earthquake reconnaissance [6]. Any failure along the lateral load path would severely degrade the stiffness and strength of the seismic force resisting system, likely cause a soft-storey mechanism, and lead to large lateral deformation and eventually collapse of the structure. This has raised concern in the research and engineering community [7-10]. Due to the lack of data characterizing the brace connection's seismic performance, in Canada CSA S16 [2] requires the design load for brace connections of type CC CBFs to be amplified by 1.5, unless ductile behaviour is demonstrated to occur during an earthquake. However, no explicit inelastic demand requirements, nor any codified design provisions regarding how to obtain ductile behaviour are readily available. With the design seismic force multiplied by 1.5 (equal to the ductility related seismic force reduction factor R_d prescribed in the NBCC [3]), the brace connections are intended to remain essentially elastic during design level earthquakes.

I-shape sections are very common as bracing members in heavy construction. The flange plate connection is commonly used for I-shape braces, as shown in Fig.1. The connection mechanism between the I-shape brace and the gusset plates at its ends results in a complex transfer of load. However, little research has been done on the I-shape brace connection; its behaviour and performance under seismic loading are far from being well-understood, which poses a high risk in the seismic performance of such buildings.

To gain insight into the behaviour of typical I-shape brace connections, a comprehensive experimental and numerical research project was launched at McGill University and Polytechnique Montréal [11,12]. Four full-scale laboratory tests were initially conducted on the assemblies of I-shape braces with the flange plate connection. In this paper, a numerical simulation procedure was developed to investigate further the behaviour of these flange plate brace connections. Non-linear finite element models replicating the brace test specimens, including their connections, were first built. The accuracy of the numerical models was validated through comparison between the simulation results and laboratory test results. As a complementary tool, the FE models were relied on to detail the stress and strain distribution throughout the loading process, upon which the force transfer mechanism of the flange plate connection was analyzed. More importantly, the validated numerical simulation approach provides a powerful and economical tool to extend the investigated parameter range, especially given that the full-scale laboratory tests are extremely labour intensive and expensive [13].

There are two force transfer branches within the brace-gusset connection: the flange branch and the web branch. In practice, design engineers usually assume the force partition between the two branches to be in proportion to the brace flange area and the brace web area, respectively. A parametric study was conducted with varying flange lap plate thicknesses and web lap plate thicknesses; the force sharing between the flange lap plates and the web lap plates was studied. The results show that the brace flange-to-web area ratio is not an accurate prediction of the force distribution, and that the forces in the two branches are in proportion to the ultimate strength of each branch. In a supplementary study, it was illustrated that a change in the web lap plate attachment does not have an influence on the connection strength and force partition, as long as there is no change in the governing failure mode of the web branch.

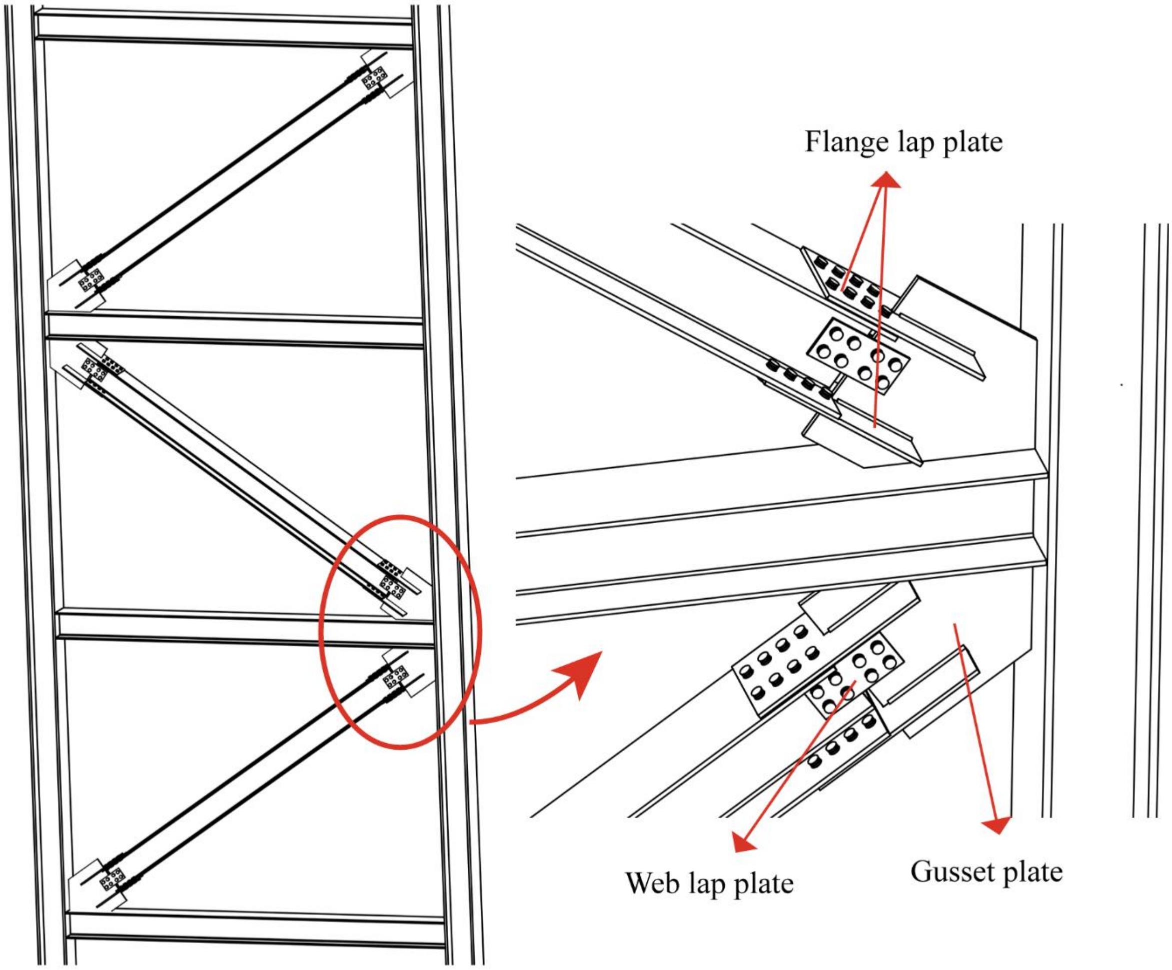

Fig. 1 Schematic drawing of CCBF with I-shape braces and flange plate brace connections

2. Finite element modelling and validation

A numerical modeling procedure based on 3D continuum elements was developed to study the behaviour of the bolted flange plate connection in ABAQUS 6.14 [14]. The 8-node brick element with reduction (C3D8R) was used for the majority of components. Refined meshes were implemented in areas where high stresses and strains were expected, e.g. in the vicinity of bolt holes. 'Hard contact' was defined for the normal behaviour between adjacent components, and a friction coefficient of 0.33 was adopted to depict the frictional tangential features [15]. Since extensive plastic deformation and even rupture of the bolts were witnessed in the laboratory tests [11,12], the 3D deformable part was built to model the bolt realistically, as shown in Fig.2. The contact between the bolt shank and bolt holes, and the contact between the nuts and the connected steel plates were defined using the 'surface-to-surface' contact method provided by ABAQUS [14]. All bolts were pretensioned using the 'turn-of-nut' method from snug-tight condition in accordance with the CSA S16 in the tests [11,12]. Since the bolt installation method is intended to achieve the minimum bolt pretension (70% of the specified minimum tensile strength), the pretension force with a magnitude of 70% of the factored tensile resistance of each bolt, was applied to simulate the clamping effect. The bilinear kinematic hardening model was implemented to describe the steel plasticity.

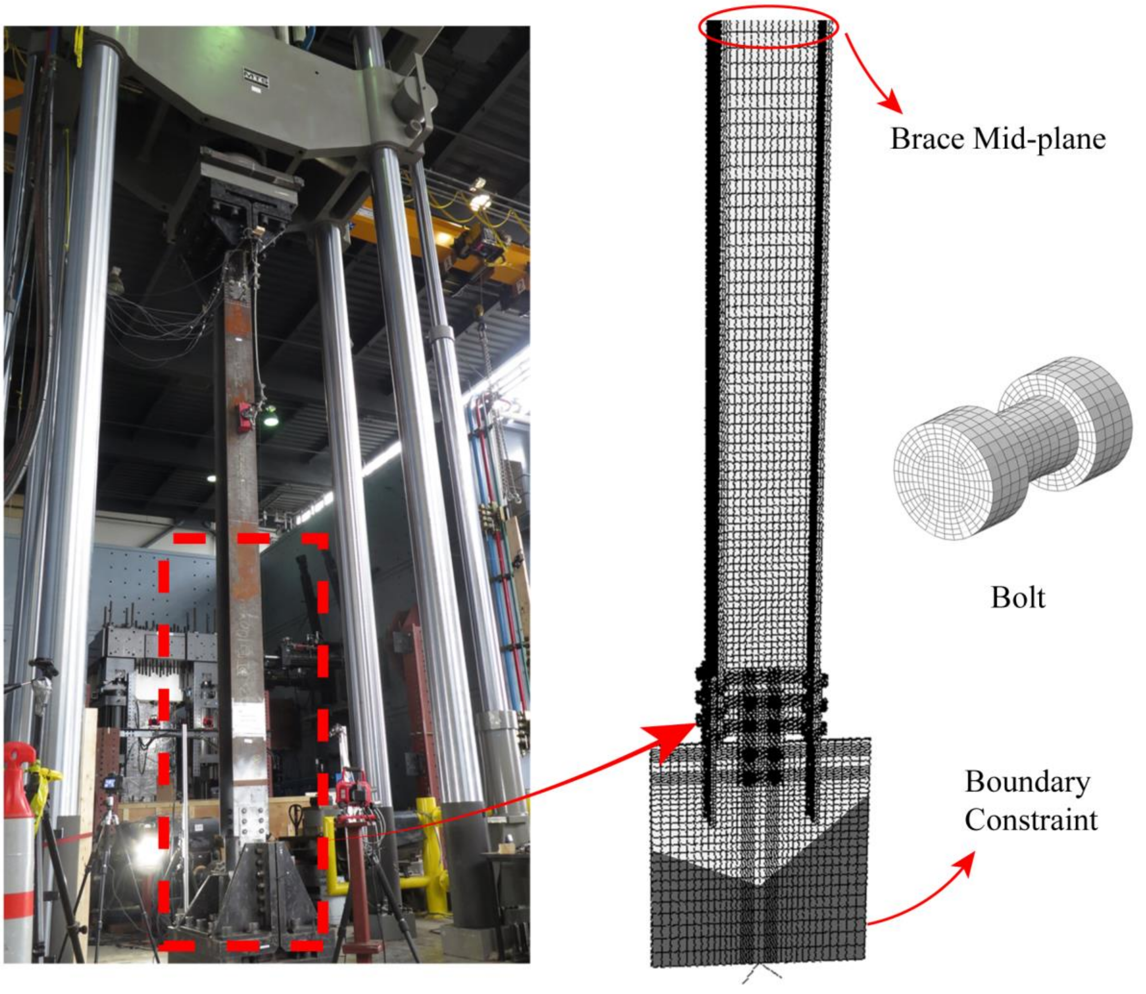

Fig. 2 I-shape brace-connection test set-up and FE brace-connection model

Models replicating the tests conducted in this research project were built first, as shown in Fig.2. All tested specimens had a length of 6667mm, which allowing them to fit a frame of 5500mm width and 3750mm height. Since the specimens of laboratory tests J-310-T and J-310-C were nominally identical [11,12], only one model was hence created (referred to as J-310). Similarly, the model labeled as J-360 was built for tests J-360-T and J-360-C. Half the brace-connection assembly was modeled, given the symmetry of the brace connection assembly. Due to the formidable number of elements in the model and the convergence difficulty arising from many bolted contacts, only monotonic loading simulation was conducted, both in tension and compression, rather than the reversed cyclic loading. Prior to loading, a linear perturbation analysis was carried out for each model to determine the buckling shape, which was then introduced with a magnitude of 2mm to the model to account for the initial imperfections. The reader is referred to Rudman [11] for more detailed information about the laboratory tests.

The load-deformation curves obtained from the monotonic loading simulation are plotted against the experimental hysteretic load-deformation curves in Fig.3. The monotonic loading curves match well with the outer envelope of the reversed-cyclic loading tests. The numerical simulation accurately predicted the buckling resistances and buckling modes (brace overall buckling in J310T and J310C, and gusset sway buckling in J360T and J360C). In terms of tensile behaviour, the FE models predicted the stiffnesses and the development of global nonlinearity with good agreement with the test results. As the deformation in tension increased, the monotonic simulations overestimated the tensile strengths. This is attributed to the plastic strains introduced by buckling and accumulated through repeated cyclic loading, which was not accounted for in the monotonic simulations. Nonetheless, the FE modeling procedure was proven to be accurate in the evaluation of the flange plate brace connection behaviour.

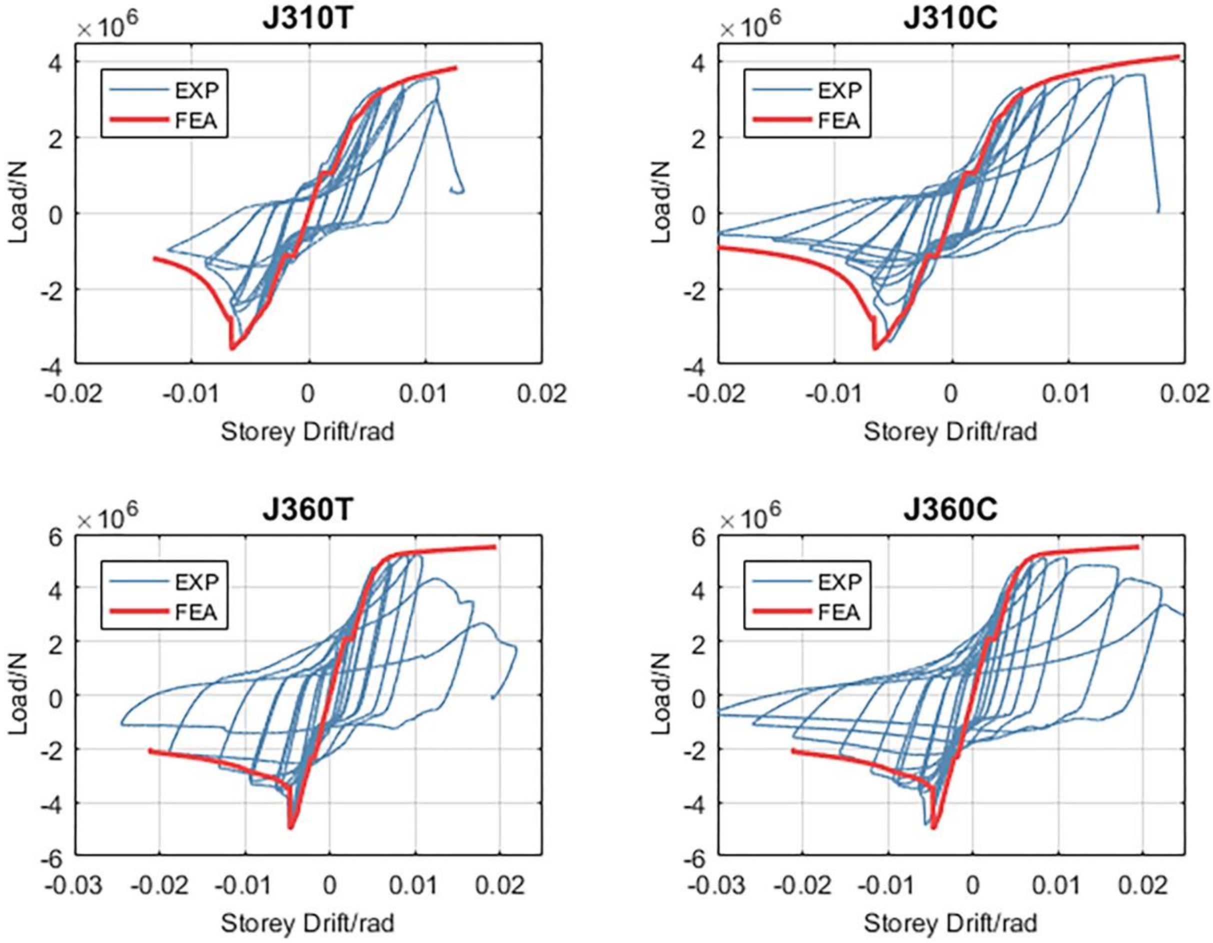

Fig. 3 Comparison of measured and predicted load-deformation curves of braces J310 and J360

3. Force transfer mechanism

In conventional design, the factored resistance of all components is required to be equal to or greater than the factored load effect obtained from a linear structural analysis. Along the brace-connection assembly, three zones can be further identified: I-shape brace, connecting plate zone, and gusset plate (Fig.4). They work in series when transmitting forces, and therefore, are designed based on the same force demand. Each brace branch will undergo both tension and compression loading due to the cyclic nature of the earthquake induced ground shaking. The tensile and compressive force demands obtained from a linear structural analysis are the same ($T_f = C_f$) since braces are often paired symmetrically. However, the braces and gusset plates are usually chosen based on their compressive resistances (C_r) to meet the requirement of the conventional design approach, because buckling normally leads to a lower compressive resistance than the capacity associated with tension loading ($C_r < T_r$). This will result in a large overstrength in braces and gusset plates when they are loaded in tension. In contrast, since buckling behaviour is often not considered in the design of the connecting plate zone, the tensile overstrength is typically less. As such, the connecting plate zone is usually the weakest link under tension loading, and is prone to fracture under strong earthquakes. Unlike structural members, e.g. braces, which can exhibit limited ductility before fracture, the connection may have very little deformation capacity if brittle failure modes occur, e.g. premature weld fracture and bolt shear rupture. Past research has demonstrated that the brace connections are very likely to experience higher seismic load than the design load level [16]. Therefore, the tensile behaviour of the connecting plate zone is critical for the survival of CCBFs under severe earthquakes, and as such, is the focus of this paper.

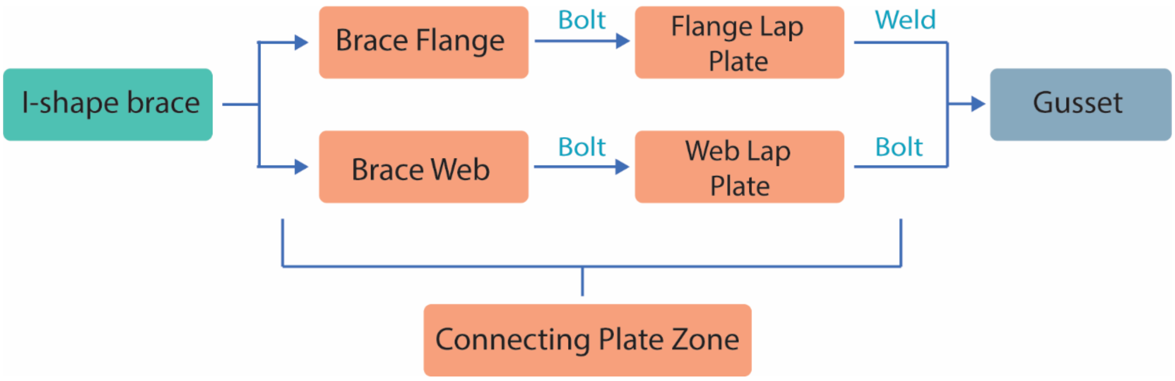

Fig. 4 Force flowchart along the I-shape brace and the flange plate connection

Within the connecting plate zone, there exist two branches working in parallel: the flange branch and the web branch. In practice, engineers usually assume that the forces in the flange branch and the web branch are in proportion to the brace flange area and brace web area, respectively. The bolts and welds are also designed in accordance with the brace flange-to-web area ratio.

The forces developed in the flange lap plates and the web lap plates, and the total global forces in the brace-connection assembly are plotted in Fig.5. To illustrate the force distribution within the connecting plate zone, the development of force proportions (in percentage) in the flange branch and web branch are also exhibited in Fig.5.

When the force level is low, the force is transferred by means of frictional resistance between all parts. As the force increases to a critical level, the slippage of the bolted connection occurs simultaneously in both the flange branch and the web branch, resulting a plateau in the load-deformation curve. Soon thereafter, the bolts in the flange plates come into contact with the bolt holes and start transferring force by bearing. With the force continuing to increase, extensive plastic deformation develops in the flange lap plates, during which another major slippage occurs in the web lap plates, given that bolts are installed on both the brace and gusset plate sides of these web lap plates. Not until after the second major slippage has taken place, will all the bolts in the web lap plates participate in the force transfer through bearing. The sequential occurrence of bearing action in the two branches is determined by the way they are connected: the flange lap plates are bolted to the brace and welded to the gusset plate, while bolted connections are used on both sides of the web lap plates for constructability purposes.

As shown in Fig.5, after all slippage occurs, the force partition within the connecting plate zone approaches a stable value; the ratios of the force in the flange lap plates to the force in the web lap plates are 3.5 and 5.25 for the two specimens with brace sections of W310X97 and W360X134, respectively. The flange-to-web area ratios for the two I-shape sections are 4.4 and 5, respectively. As such, it is shown that the flange-to-web area ratio assumption is not an accurate assumption for the force sharing between the two branches. The concerning ramification lies in the bolt and weld safety in the connecting plate zone. Since the force demands for bolts and welds are also assumed to comply with the flange-to-web area ratio in practice, such inaccuracy may lead to inappropriate arrangement of the bolt and weld resistances, and may possibly result in early bolt rupture or weld fracture.

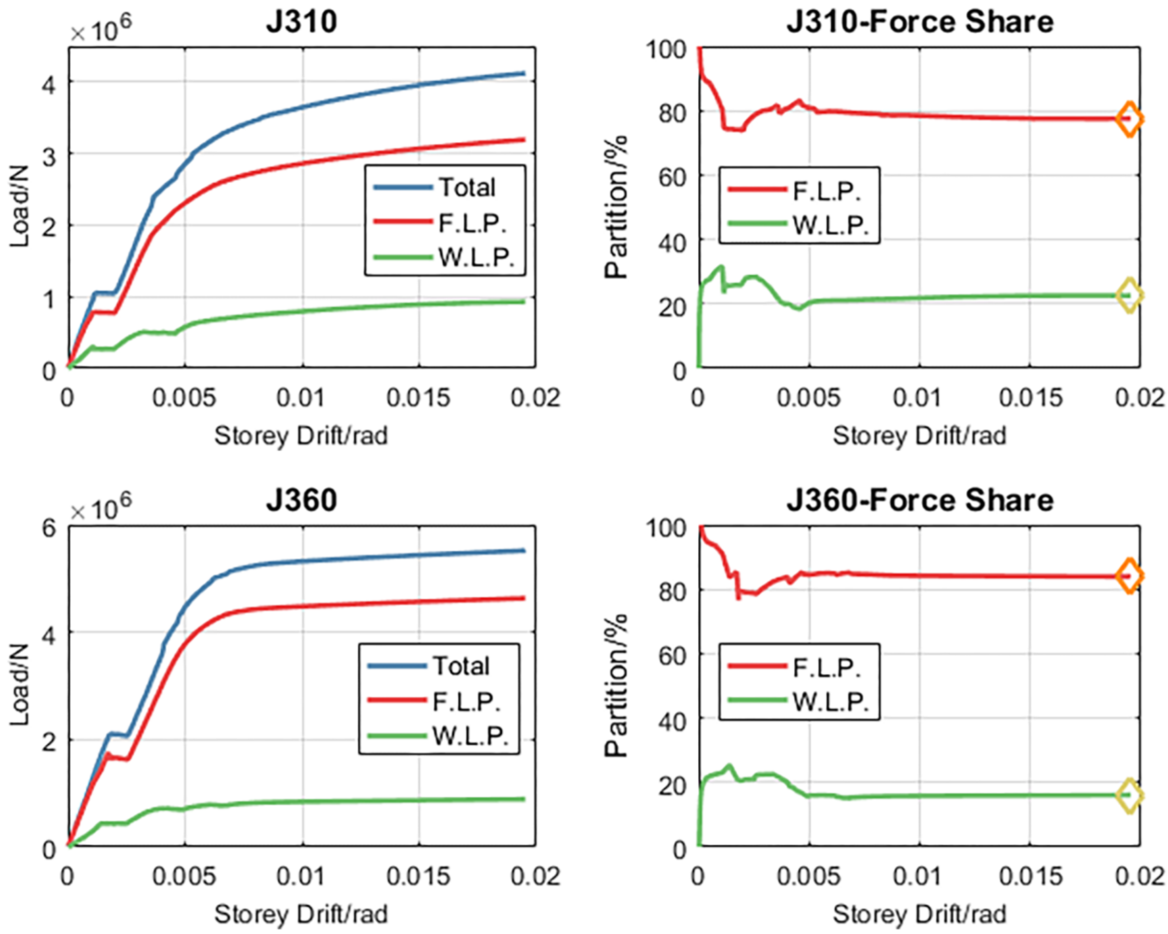

Fig. 5 Force development and partition within the connecting plate zone: F.L.P.=flange lap plate, W.L.P.=web lap plate

4. Parametric study on force partition

To further investigate the force partition within the connecting plate zone, a parametric study was conducted using the FE models. The two models replicating the test specimens [11,12] served as the control cases; J-360 and J-310. Two parameters were studied, the flange lap plate thickness and the web lap plate thickness. A full list of models with detailed information about the studied parameters is found in Table 1. For the model ID labeling, the first two parts indicate the reference model upon which the model is built. The third part shows the parameter varied: 'F' means the flange lap plate thickness and 'W' means the web lap plate thickness. The last part represents the ratio of the corresponding parameter to that of the reference model. For instance, J-360-F-125 refers to the model based on the control model J-360 but with the flange lap plate thickness 1.25 times that of the model J-360.

All the models were loaded in tension to 60mm (corresponding to 2% storey drift). The ultimate ratios of force developed within the flange lap plates to the force in the web lap plates (F_{FLP} / F_{WLP}), referred to as force ratios hereafter, were calculated in the same manner as in the previous section (Fig.6). For comparison, the ratios of the brace flange area to the brace web area, simplified as area ratios, were also plotted.

Generally, the force partition in the connecting plate zone is not in proportion with the brace flange area and the brace web area. The J-310 series and the J-360 series exhibited the same trend: the flange-to-web force ratios increase with thicker flange lap plates, but decrease as the web lap plates increase in thickness.

Table 1 FE models in the parametric study

Model ID	Brace Section	Bolt Grade and Size (in.)	Gusset Thickness (mm)	Flange Lap Plate Thickness (mm)	Web Lap Plate Thickness (mm)
J-310	W310X97	A325 (7/8)	15.9	15.9	9.53
J310-F-075	W310X97	A325 (7/8)	15.9	12.0	9.53
J310-F-125	W310X97	A325 (7/8)	15.9	20.0	9.53
J310-W-075	W310X97	A325 (7/8)	15.9	15.9	7.14
J310-W-125	W310X97	A325 (7/8)	15.9	15.9	12.0
J-360	W360X134	A490 (1)	19.1	15.9	9.53
J360-F-075	W360X134	A490 (1)	19.1	12.0	9.53
J360-F-125	W360X134	A490 (1)	19.1	20.0	9.53
J360-W-075	W360X134	A490 (1)	19.1	15.9	7.14
J360-W-125	W360X134	A490 (1)	19.1	15.9	12.0

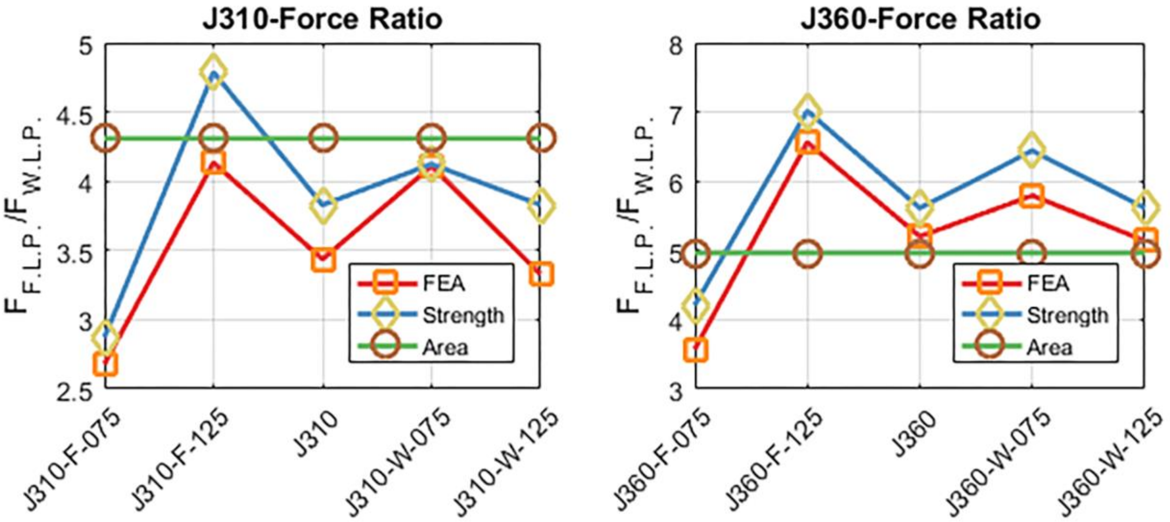

Fig. 6 Ultimate ratio of force in the flange lap plates to force in the web lap plates

The ultimate strengths of the flange branch and the web branch in the connecting plate zone were calculated using the measured material properties without resistance factors for all the models, based on respective formulas in CSA S16 [2]. For instance, in model J-310-F-075, the governing failure modes for the flange branch and the web branch are net section fracture of the flange lap plates and the block shear rupture of the brace web (shown in Fig.7), having the unfactored resistance of 2065kN and 719kN, respectively. Therefore, the flange branch strength to the web branch strength ratio is 2.87 for model J-310-F-075. Similarly, the strength ratios for all models were determined, and are plotted in Fig.6. The comparison shows that the strength ratios have the same trend with the force ratios. This is mainly attributed to the fact that the connecting plate zone is usually the weakest in terms of tensile resistance in CCBFs compared to the brace members and gusset plates, as explained before. At large tensile deformations, the ultimate tensile capacities of the two branches in the connecting plate zone will be fully developed. The strength ratios predicted the force partition with an average error of 10%, compared to 21% by the area ratios. The average error is defined as

$$\frac{\Sigma[(R_{strength/area}-R_{FEA})/R_{FEA}]}{10} \tag{1}$$

in which, $R_{strength}$, R_{area} and R_{FEA} represent the ultimate strength ratio of the flange branch to the web branch, the ratio of brace flange area to brace web area, and the flange-to-web force ratio obtained by FE analyses, respectively.

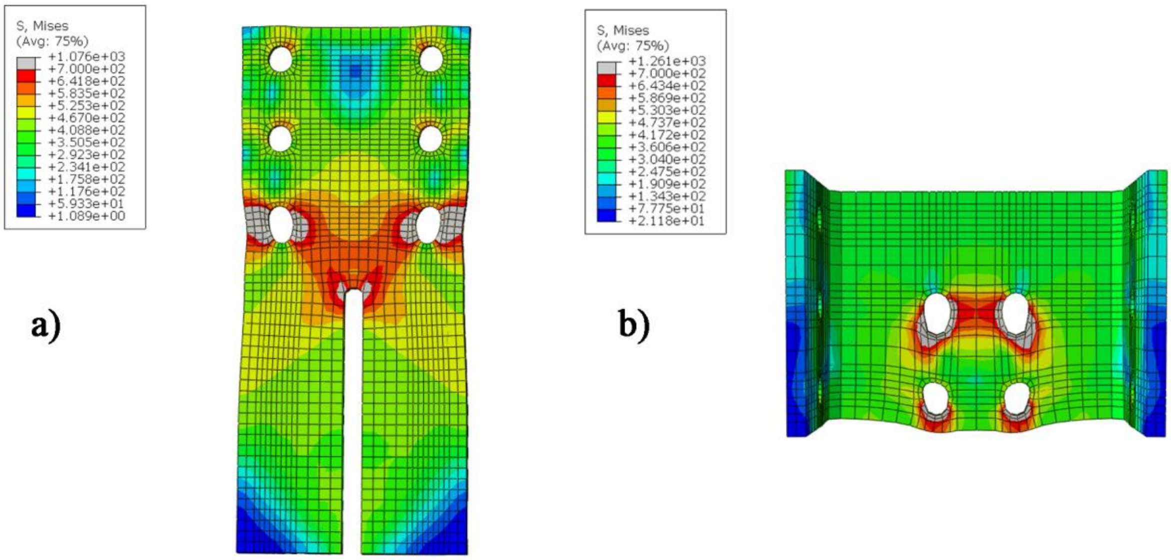

Fig. 7 Failure modes of the connecting plate zone in J-310-F-075: a) net section fracture of flange lap plate; b) block shear of brace web

For most cases in the parametric study (9 out 10 models) the strength ratios overestimated the force ratios. This overestimation comes from the prominent presence of frictional forces between the web lap plates and the brace web in the FE models. In addition to the force provided by brace web block shear, the frictional forces also contributed a significant portion to the force in the web lap plates. The error by the strength ratio is believed to be further reduced under real earthquake ground shakings, because the frictional coefficient will decrease substantially due to the cyclic rubbing between all components, and the bolts will lose most of their pretension when extensive plasticity takes place.

The accurate prediction of the force partition is crucial for the safe design of the bolts and welds along the two branches, especially given that the bolt shear rupture and weld fracture are two well-known failure modes showing little ductility. Moreover, bolts and welds are usually designed based on their ultimate strength, and therefore, have less overstrength compared to components designed by the yielding limit state. When subjected to the design earthquake, the actual capacity of the structure is expected to be fully mobilized [17]. In order to avoid brittle failure of the bolts and welds in the flange plate connection, it is recommended that these elements be designed based on the ultimate strength of the corresponding flange branch or web branch for CCBF type structures.

5. Web lap plate attachment

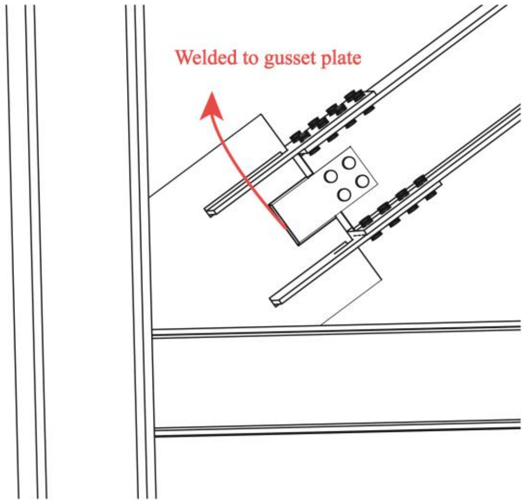

Fig. 8 Welded attachment of web lap plate to gusset plate

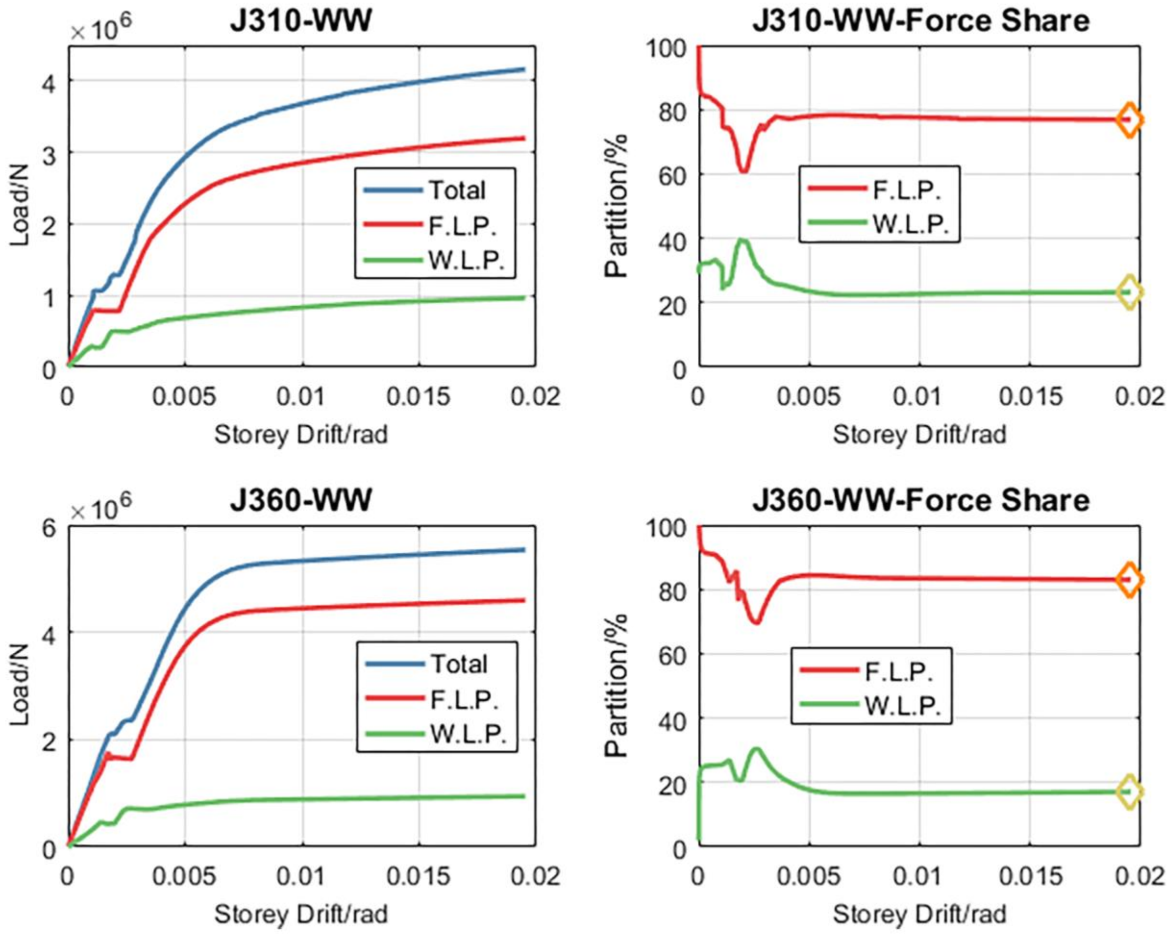

Fig. 9 Force development and partition within the connecting plate zone: F.L.P.=flange lap plate, W.L.P.=web lap plate

In the previous section, it was found that the web branch needed two times the bolt slippage to develop the bolt bearing action as compared to the flange branch. To investigate the effect of such delay in developing the full resistance of the connection, another web lap plate attachment shown in Fig.8 (welded to the gusset plate and bolted to the brace web) was studied. This connection detail for web lap plates was also recommended by Astaneh-Asl [18]. Two FE models were created with the new web lap plate attachment based on the two control models in the parametric study, labeled as J-310-WW and J-360-WW, respectively. The weld sizes (7mm for both models) were chosen to ensure the governing failure modes did not change, that is block shear failure of the web of the I-shape brace.

Figure 9 exhibits the force development in both the flange branch and the web branch, and their force share in percentage. As expected, it took almost the same amount of bolt slippage as occurred for the flange lap plates (even though the slippage occurred twice) for the web lap plates to start transferring force by means of bolt bearing. Nonetheless, in terms of global strength and ultimate force partition within the connecting plate zone, the change of web attachment seemed not to have an influence, since it did not change the governing failure mode of the web branch.

However, such change in web lap plate attachment will greatly increase the difficulty of constructing such a structure. Without the pre-welded web lap plates, the installation of the I-shape braces is easily accommodated. In contrast, if the pair of web lap plates are first welded to the gusset plate, there is only one way to put the I-shape braces in place. As such, for the convenience of field assembly, it is not recommended to weld the web lap plates to the gusset plate.

6. Conclusion

A numerical simulation procedure was developed to study the behaviour of a typical I-shape type CCBF brace connection, i.e. the flange plate connection. Material nonlinearity was considered in the FE models. The bolted connection was simulated through the creation of 3D deformable bolt parts. The accuracy of the FE models was validated by comparison between the numerical results and the laboratory test results of I-shape braces and connections provided by Rudman [11]. Three zones (I-shape brace, connecting plate zone, and gusset plate) were identified along the brace-connection assembly, and two branches (the flange branch and the web branch) exist within the connecting plate zone. When the force level is low, the force is transferred by means of friction due to the clamping effect of bolt pretension. As the force increases, the bolts in the flange branch will start to transfer forces through bearing first after slippage, followed by those in the web branch. As for the ultimate force share between the two branches at large deformations, the parametric study results show that both branches will develop their ultimate strengths since the connecting plate zone is usually the weakest link in CCBFs. Therefore, to avoid bolt shear rupture and weld fracture in CCBFs, it is recommended that both bolts and welds be designed based on the ultimate strength of the corresponding branch. Moreover, another detail of the web lap plate attachment (welded to gusset plate and bolted to brace web) was studied. The results show that such a change does not affect the ultimate strength and force partition within the connecting plate zone. This alternative attachment approach is not recommended since it will cause extra difficulty in brace assembly in the field.

7. Acknowledgements

The authors would like to thank the ADF Group Inc. and DPHV Structural Consultants for their generous technical and financial support, as well as the Natural Sciences and Engineering Research Council of Canada (NSERC), the Fonds de recherche du Québec - Nature et technologies (FRQ-NT) and the Centre d'études interuniversitaire des structures sous charges extremes (CEISCE). The first author is supported in part through the China Scholarship Council (CSC). The finite element computations were conducted on the supercomputer Graham cluster, which is managed by Calcul Québec and Compute Canada. The supercomputer operation is funded by the Canada Foundation for Innovation (CFI), NanoQuébec, RMGA and the Fonds de recherche du Québec - Nature et technologies (FRQ-NT).

8. References

[1] AISC (2016): AISC 360-16 Specification for structural steel buildings. American Institute of Steel Construction, Chicago, IL, USA.

[2] CSA (2014): CSA S16-14 Limit state design of steel structures. Canadian Standard Association, Mississauga, ON, Canada.

[3] NRCC (2015): National building code of Canada, 14th edition. National Research Council of Canada, Ottawa, ON, Canada.

[4] ASCE (2016): ASCE/SEI 7-16 Minimum design loads and associated criteria for buildings and other structures. American Society of Civil Engineers, Reston, Virginia, USA.

[5] Davaran A, Tremblay R, Beland T, Fahnestock LA, Hines EM (2014): Experimental behavior of low-ductility brace connection limit states. *Structures Congress*.

[6] Tremblay R, Filiatrault A, Timler P, Bruneau M (1995): Performance of steel structures during the 1994 Northridge earthquake. *Canadian Journal of Civil Engineering*. 22(2), 338-60.

[7] Sen AD, Sloat D, Ballard R, Johnson MM, Roeder CW, Lehman DE, Berman JW (2016): Experimental evaluation of the seismic vulnerability of braces and connections in older concentrically braced frames. *Journal of Structural Engineering*, 142(9):04016052.

[8] Sen AD, Swatosh MA, Ballard R, Sloat D, Johnson MM, Roeder CW, Lehman DE, Berman JW (2016): Development and evaluation of seismic retrofit alternatives for older concentrically braced frames. *Journal of Structural Engineering*, 143(5):04016232.

[9] Sizemore JG, Fahnestock LA, Hines EM, Bradley CR (2017): Parametric study of low-ductility concentrically braced frames under cyclic static loading. *Journal of Structural Engineering*, 143(6):04017032.

[10] Bradley CR, Fahnestock LA, Hines EM, Sizemore JG (2017): Full-scale cyclic testing of low-ductility concentrically braced frames. *Journal of Structural Engineering*, 143(6):04017029.

[11] Rudman, A (2018): Testing of conventional construction W-shape brace members and their bolted end connections undergoing reversed cyclic loading. Master's thesis, Department of Civil Engineering, McGill University, Montreal, QC, Canada.

[12] Rudman A, Tremblay R, Rogers CA (2019): Seismic performance of Conventional Construction I-shape brace members and their bolted connections undergoing reversed cyclic loading. *12th Canadian Conference on Earthquake Engineering*, Quebec, Canada.

[13] Maggi YI, Gonçalves RM, Leon RT, Ribeiro LF (2005): Parametric analysis of steel bolted end plate connections using finite element modeling. *Journal of Constructional Steel Research*, 61(5), 689-708.

[14] Systèmes, Dassault (2014): *Abaqus 6.14 Documentation*. Providence, RI: Dassault Systèmes.

[15] Kulak GL, Fisher JW, Struik JHA (1987): *Guide to design criteria for bolted and riveted joints*. 2nd edition. New York: Wiley.

[16] Tremblay R, Castonguay PX, Guilini-Charette K, Koboevic S (2010): Seismic force demand on brace connections of conventional construction braced steel frames designed according to Canadian seismic provisions. *Structures Congress*, 1361-1372.

[17] Mitchell D, Tremblay R, Karacabeyli E, Paultre P, Saatcioglu M, Anderson DL (2003): Seismic force modification factors for the proposed 2005 edition of the National Building Code of Canada. *Canadian Journal of Civil Engineering*, 30(2), 308-27.

[18] Astaneh-Asl A, Cochran M, Sabelli R (2006): Seismic detailing of gusset plates for special concentrically braced frames. *Structural steel educational council*.

