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Seismic Evaluation of Existing Steel Braced Frames Designed in accordance with the 1980 Canadian Code Requirements Using Nonlinear Time History Analysis

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Abstract: A 10-storey building located in Vancouver, BC, was designed following the provisions of the 1980 National Building Code of Canada and the CSA-S16.1-M78 steel design standard and its seismic behaviour was assessed using nonlinear time-history analysis. The building was framed by tension-only X-bracing and tension-compression chevron bracing built with back-to-back double angle braces as commonly used for steel structures in that era. An analytical model of the braces in the X-bracing configuration was developed in OpenSees and a parametric study was carried out to identify the modeling parameters that had significant impact on the representation of the brace inelastic response. Elastic response was represented for the other frame elements. Nonlinear time history analysis of the tension-only X-bracing was carried out for three historical ground motion records compatible with the NBCC spectrum. The structural elements were assessed based on the acceptance criteria proposed in the ASCE 41-06 standard. The deformation capacity of the braces is found to be insufficient at two levels and brace connections at all levels have inadequate strength. All columns except those at the top level also have insufficient strength. The strength of all beams is satisfactory.

1 Introduction

The seismic design provisions were first introduced in the National Building Code of Canada (NBCC) in 1941. Before that time, seismic loads were not considered in the design of buildings. Gradually, special studies have been conducted on the seismic hazard and the seismic response of structures and minimum requirements for seismic loading were defined (Mitchell et al. 2010). The seismic provisions of the current Canadian building design code, NBCC 2010 (NRCC 2010), reflect the most recent findings in the area of seismic engineering and are comparable to other seismic codes in developed countries. Special seismic design and detailing provisions for steel structures were first introduced in the CSA S16 design standard in 1989. Since then, they had significantly evolved as new research results and observation of the seismic response of structures in recent earthquakes have become available (Tremblay 2011). In earlier editions of CSA S16, ductile behaviour was not explicitly taken into account in seismic design. Therefore, the steel structures built before the 1990's are likely to exhibit seismic deficiencies, and their performance may endanger the safety of occupants during severe earthquakes. In view of the significant number of such buildings in the Canadian infrastructure, it is essential to reduce this seismic risk by properly assessing the seismic behaviour of steel structures designed according to pre-1989's codes and identifying their potential deficiencies.

In the 1980's, the tension-only X-bracing and tension-compression chevron bracing systems were the most common types of lateral load resisting system used in steel buildings in Canada. Braces were usually designed as back-to-back double angle sections while beams and columns were selected from

W-shape sections. At each end of the brace, back-to-back legs were typically connected to a single vertical gusset plate using high strength bolts.

In a previous study (Jiang et al. 2012a), the seismic assessment of a 10-storey building located in Vancouver, British Columbia, was carried out using the response spectra analysis as specified in the NBCC 2010. Resistance of structural elements and connections was calculated following CSA-S16-09 (CSA 2009) provisions. This study showed that none of the structural elements, including braces, beams and columns, had adequate capacity. For the majority of the brace connections, the resistance associated with block shear failure was insufficient, the only exception being the connections at 1st, 4th and 5th level where the tension failure on net section was critical.

In this study, the seismic response of the tension-only X-bracing of the same 10-storey building is assessed using the results of nonlinear time history analysis. The analysis is carried out for a group of three historical ground motion records scaled to match the NBCC design spectrum. Two options for the modeling of the braces were considered to achieve the most realistic representation of brace inelastic response: (i) brace is modeled as single member with the properties of double angle sections, and (ii) two angles constituting a brace are modeled as individual members connected back-to-back to each other by means of contact and gap elements. Other members of the frame were modeled for elastic response. The procedures specified in ASCE 41-06 standard (ASCE 2006) were used to evaluate seismic performance of braces, beams, columns and brace connections. This standard provides the requirements for the analysis and assessment of existing buildings, as well as the acceptance criteria for steel components. Based on these acceptance criteria, the potential deficiencies of the X-bracing of the 10-storey building were identified.

2 Design of the building studied

The plan view and frame elevation of the studied structure is illustrated in Figure 1. The 10-storey commercial building was designed in accordance with the design provisions of the 1980 NBCC (NRCC 1980) and the CSA-S16.1-M78 (CSA 1978) steel design standard. The NBCC 1980 was selected for design because of the large differences between the design seismic loads compared to those specified in the 2010 NBCC. The building is situated in Vancouver, British Colombia, on a Class C site. It is considered to be of normal importance. Tension-only braced frames with diagonals in an X configuration provide lateral resistance in the north-south direction whereas chevron braced frames are used in east-west direction.

Seismic loads were determined using the fundamental period of the structure obtained by Rayleigh method, as would have been the case in practice. An iterative design procedure was applied until the member selection converged. The fundamental periods of the X-braced and chevron braced frames are 2.77 s and 2.00 s, respectively. An equivalent static force procedure was applied, as permitted by NBCC 1980 for regular structures. The seismic base shear was determined from:

[1] $V=ASKIFW$

where A is the design acceleration ratio, S is the seismic response factor (= $0.5/T^{0.5}$), K is a coefficient calculated based on the type of construction, I is the importance factor, F is the foundation factor and W is the seismic weight. In this study A = 0.08, I = 1.0, F = 1.0, W = 83756 kN for the entire building, and K = 1.0 and 1.3 for the tension-compression and tension-only bracing, respectively. This resulted in seismic force coefficients, V/W, equal to = 0.032 and 0.029 for the tension-only and tension-compression bracing systems, respectively.

In-plane torsion and P-delta effects were considered, and a concentrated lateral force, F_t = 0.84% V, was applied at the roof level. The remaining seismic load, $V – F_t$, was distributed along the height of structure as a function of the relative product of the seismic weight and the elevation from the ground at the level under consideration. The overturning moment reduction factor, J, was equal to 1.0 for this structure.

The frames were designed following the requirements of CSA-S16.1-M78. The structural members were made of CSA-G40.21-300W steel. The braces consisted of double angle sections with equal legs in back-to-back position. Class 3 sections were selected to control width-to-thickness ratios prescribed in CSA-S16.1-M78. The overall slenderness ratio of the bracing members was limited to 300 for the tension-only X-bracing and to 200 for the tension-compression chevron bracing frames. An unsupported length of the brace in tension-only X-braced frame was taken equal the half of the total brace length. Back-to-back angles' legs were connected to a single vertical gusset plate using high strength A325 bolts, 19.1 mm in diameter. The design of the bracing members was governed by axial strength requirements, except at the roof level where the braces were selected to meet the maximum brace overall slenderness limits. The factored resistance of the bolted connections subjected to shear was determined as the lesser of the factored bearing resistance, B_r and the factored shear resistance of the bolts, V_r. CSA S16.1-M78 did not consider either shear lag effects or block shear failure mode to evaluate the resistance of such bolted connections.

The beams and columns were selected from W sections. Columns were tiered in two-storey segments and selected from Class 2 and 3 sections to provide sufficient axial strength. Beams were selected from Class 3 sections to resist the combined axial load and bending moment. The beams were assumed to be laterally supported.

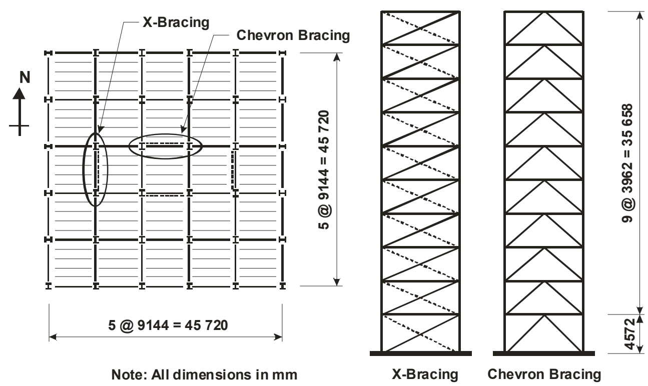

Figure 1: Plan view and braced frame elevations of the 10-storey building

3 Numerical modelling

3.1 Numerical modelling of the bracing members

Realistic assessment of the performance of concentrically braced steel frames under seismic loads strongly relies on an accurate prediction of the brace inelastic response. In this study, a numerical model of back-to-back double angle sections was developed using the OpenSees platform (McKenna and Fenves 2004, Jiang et al. 2012b). The model was first studied for the first-storey bracing members (2L-200x200x16mm). Two strategies were examined: (i) the braces were represented by a single element with the properties of double angle sections; and (ii) the two angles constituting a brace were modeled as individual members connected back-to-back to each other by means of contact and gap elements to reproduce the action of a built-up member.

For the second model, the contact elements were placed between the two angles and modeled by elastic beam column elements while zerolength elements were used to represent gap elements. When the two angle sections come in contact, the gap elements activate, otherwise each component can freely move away from each other. In the connection zone, the double angle sections in the second model were linked to each other by means of elastic beam column elements with high axial and flexural stiffness.

In both models, the connections of the braces to the gusset plates were represented by zerolength elements. These in turn were attached to the beams or columns by means of elastic beam column elements with high flexural and axial stiffness simulating the rigidity of the connection zones. The length of the brace members was 10223 mm while the net length of the brace, excluding the length of the brace connections, was 8307 mm. In the building frame models, out-of-plane flexural response of the gusset plates was modeled by using zerolength elements with uniaxial Giuffré-Menegotto-Pinto material (Steel 02) for flexure. Elastic material was considered to simulate the torsional response in the connection regions. For simplicity in the preliminary investigation of the brace models, pinned connections were used at both brace ends.

Each brace member was divided into 16 displacement-based beam-column elements with 4 integration points placed along each element (Bertero 2004). The cross section of each brace element was discretized by fibers. To reproduce the nonlinear behavior of braces, the Giuffré-Menegotto-Pinto steel material with isotropic hardening (Steel 02) was used. A yield strength of 300 MPa corresponding to the minimum specified yield value was assigned to the material. Residual stresses were assigned to the cross-section fibers. They varied linearly along the width of the angle legs following the pattern observed experimentally by Adluri and Madugula (1995). A sinusoidal deformation with maximum amplitude of 1/1000 of the unsupported brace length was assumed for the initial out-of-straightness of the braces, as specified in CSA-S16.1-M78. The initial out-of- straightness was applied both for in-plane and out-of-plane directions. At every level in the building frame model, the two braces of the X-bracing were connected at the intersection point. For both modelling strategy, in-plane and out-of-plane lateral supports were assigned at the brace mid-length to simulate the restraint offered by the intersecting brace.
In Figure 2, the hysteresis of the single element brace model is compared to that obtained with the model with two individual angle elements. For the second model, zero, one and two gap elements were considered to investigate the possible impact of the number of gap elements on the inelastic brace response. As shown, the compressive resistance is higher for the brace modelled with a single element because buckling of the individual angle members could not be represented. The study showed that in-plane buckling of the brace was the governing mode. Hence, the number of gap elements between the individual angle members had no impact on the buckling strength of the bracing member. By inspection it was determined that in-plane buckling mode was critical for all braces in the studied 10-storey frame.

According to CSA-S16.1-M78, the maximum slenderness ratio permitted for a tension member is 300. To satisfy this requirement, one stitch connector was required at mid-length of all braces at the intersection of the two braces. To model the stitch connector in the second modelling strategy, an elastic beam element was used to attach the individual angle members to each other. In Figure 2b, the hysteretic response of a double angle member modelled with two elements with and without a stitch connector is illustrated. No gap element was used as they do not affect the brace response. As expected, the addition of a stitch at the brace mid-length had nearly no influence on the brace axial response because buckling occurs in-plane and does not induce shear in the stitch connector.

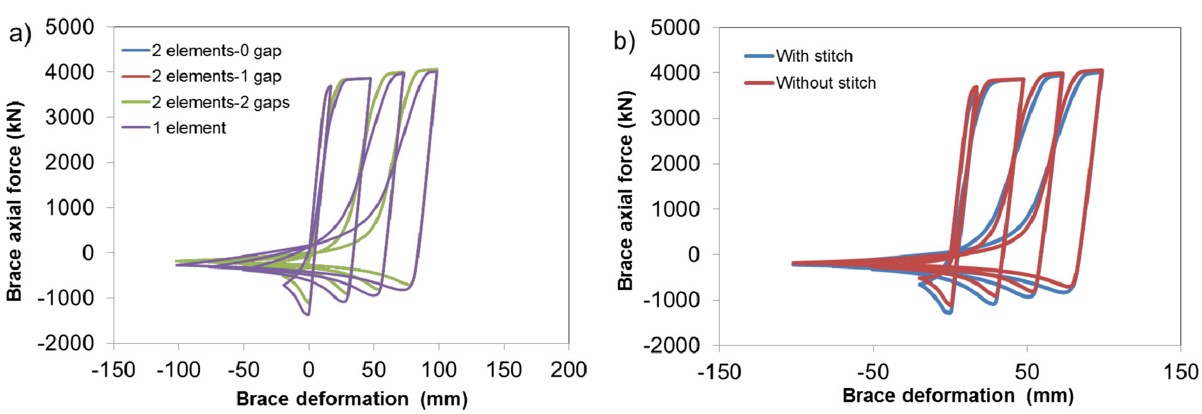

Figure 2: Hysteretic responses of double angle bracing members: a) comparison between models with single and two individual elements; b) Influence of the stitch connector on brace axial response

The brace buckling mode was further investigated by examining the brace response under monotonically increasing axial displacement. The section used for the braces at the 8th storey of the X-bracing (2L-125x125x8mm) was considered in this study. The brace was modelled with two individual elements. One stitch connector was assigned at mid-length of the brace. Gap elements were not introduced between the two angles. The influence of the intersecting bracing member was included in the analysis by preventing in-plane and out of plane movements at the brace mid-length. Monotonic displacement was applied in ten equal steps and in-plane and out-of-plane deformations of the two angles were recorded at each loading increment. In-plane deformations of the brace under the progressively increasing axial displacement at one end of the brace are shown in Figure 3a. In Figure 3b, the out-of-plane position of each angle is shown with respect to its longitudinal axis. In-plane deformations are much larger than out-of-plane deformations, confirming that in-plane brace buckling took place, with limited contribution of individual brace buckling.

This preliminary study showed that a single-member model is inadequate to represent the buckling response of a double angle built-up bracing member. Buckling of the braces in the frame studied is governed by in-plane deformations. In such situation, there is no contact between the two individual angles and thus the number of gap elements does not have impact on the calculated brace compressive resistance. Therefore, gap elements between the two angles were omitted in the final brace model used to analyze seismic response of the 10-storey frame. However, in the frame model, one stitch was located at mid-length of the brace members, where two braces of the X-bracing intersect.

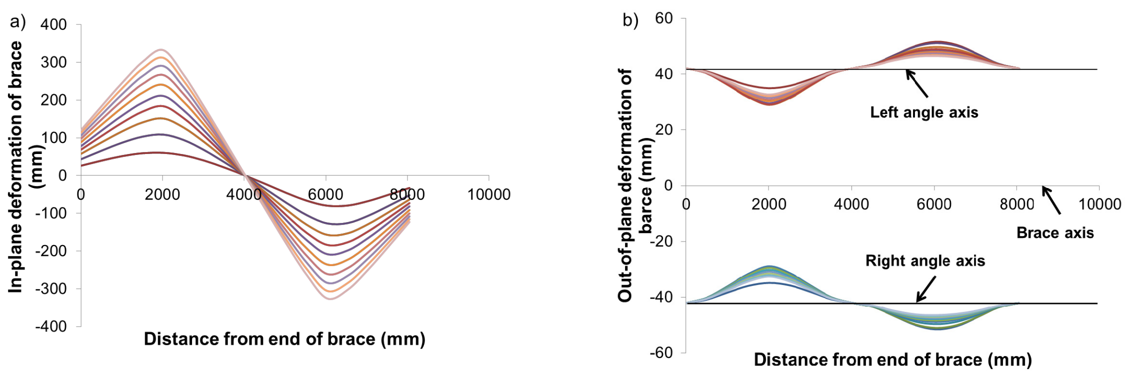

Figure 3: a) In-plane deformation; and b) out-of-plane deformation of the brace with one stitch at the mid-length of member

3.2 Numerical modelling of the other structural members

The main objective of this study was to investigate the nonlinear behaviour of the braces in the 10-storey braced steel frame; therefore, the other structural members were modelled using elastic beam column elements to evaluate the elastic force demand on these members and assess their seismic behaviour. Actual flexural and axial stiffness properties of the beams and columns were assigned to the beam-column elements and the zerolength element with high axial and negligible flexural stiffness was considered to model the beam-column connection. Column bases were assumed to be pinned.

4 Structural assessment

4.1 Ground motions

Twenty ground motions were first selected that respected the magnitude-distance scenarios proposed by Atkinson (2009). Following the procedure described in this reference, three ground motion records having the lowest standard deviation of SA_{targ}/SA_{sim} and a mean value of SA_{targ}/SA_{sim} in the 0.5 to 2.0 range were identified in the original set and kept for analyses. The ground motions were scaled such that the average value of the response spectra did not fall below the NBCC 2010 spectrum for periods ranging from 0.2T to 1.5T, where T is the fundamental period of the structure (ASCE 2005). Table 1 gives the information related to the selected three ground motion records.

Table 1: Summary of the selected three ground motion records

Earthquake Name	Year	Recording Station	Mean SA_{targ}/SA_{sim}	Standard Deviation SA_{targ}/SA_{sim}	Scale Factor
Imperial Valley	1979	El Centro	0.74	0.17	0.95
Superstition Hills	1987	El Centro Imp. Co	1.32	0.38	1.69
Superstition Hills	1987	Poe Road	1.06	0.22	1.36

4.2 Assessment procedures

The seismic performance of the tension-only X-braced frame was assessed using nonlinear time history analysis under the selected three ground motions. Canadian design provisions do not provide specific acceptance criteria for structural steel components of concentrically braced steel frames. For this reason, the assessment was done following the procedure prescribed in the ASCE 41-06 standard. This standard includes requirements for the analysis and assessment of existing buildings and provides acceptance criteria for steel components. The provisions used in this study are presented below.

ASCE 41-06 specifies that a two dimensional model can be used to analyse a structure if the structure has rigid diaphragms and the displacement multiplier due to total torsional moments does not exceed 1.5 at any floor. The displacement multiplier is determined as the ratio of the maximum displacement at any point on the floor diaphragm to the average displacement ($\delta_{max}/\delta_{avg}$). The accidental torsion should be considered in the analysis unless it can be shown that the accidental torsional moment is less than 25% of the inherent torsional moment in the building or that the displacement multiplier calculated for the applied loads and accidental torsion is less than 1.1 at every floor.

For the building studied, the maximum value of displacement multiplier was 1.09. Consequently, the effects of accidental torsional were not considered in the analysis and a two-dimensional model was used. The building had no plan irregularities and no primary columns were part of two or more intersecting seismic force resisting systems; therefore, multidirectional seismic effects did not need to be considered. The analysis was thus carried out independently along each principal direction of the structure. The maximum values of the response parameters were used for the evaluation, as required in ASCE 41-06 when the results for fewer than seven time history are available.

Several acceptance criteria are given in ASCE 41-06 for structural members in steel concentrically braced frames. Steel components must be classified as either deformation controlled (ductile) or force controlled (non-ductile) elements. Moreover, different actions of the same element can be classified in different categories. For instance, axial actions on braces are considered as deformation-controlled, actions on beams and columns with non-negligible axial loads can be considered force- or deformation-controlled actions depending on the amplitude of the axial loads, whereas shear and moment actions on brace connections must be treated as force-controlled actions.

According to ASCE 41-06, structural elements must also be categorized as primary and secondary components. A primary component resists seismic forces to provide the selected performance level for the structure while a structural component not designed to resist seismic forces and achieve the selected performance level is categorized as a secondary component.

The acceptability of deformation-controlled and force-controlled actions should be evaluated for each component based on the acceptance criteria proposed in the ASCE-41-06 standard. For deformation-controlled actions of structural steel components, these acceptance criteria are expressed in terms of specific deformation limits. The plastic deformation of bracing members is considered as the acceptance criteria for nonlinear procedure. For instance, the plastic deformation of the brace in tension, categorized as the primary component is $7\Delta_T$, where Δ_T is the axial deformation of the brace at the expected tensile yielding load. The expected yield strength is determined as the mean value of resistance of a component at the deformation level for a group of similar components that considers the variability in material strength including strain hardening and plastic section development. In this study the value of 330 MPa was used.

According to ASCE 41-06, the strength of beams with axial loads exceeding 10% of the members' axial strength should be evaluated following the procedure prescribed for columns. If a structural member, either a beam or a column, is subjected to combined axial compression and bending moment and the axial load in the member is less than 50% of the lower bound axial strength, P_{CL}, the compressive behaviour of the member is considered as force-controlled, and flexural behaviour of the element is considered as deformation-controlled. In that case, the member must be satisfied:

[2] $P_{UF}/2P_{CL} + M_x/m_xM_{CEx} + M_y/m_yM_{CEy} \leq 1.0$ when: $P_{UF}/P_{CL} < 0.2$

[3] $P_{UF}/P_{CL} + 8/9\,[M_x/m_xM_{CEx} + M_y/m_yM_{CEy}] \leq 1.0$ when: $0.2 \leq P_{UF}/P_{CL} \leq 0.5$

In equations [2] and [3], P_{UF} is the axial force in the member, P_{CL} is the lower-bound compression strength of the member, M_x and M_y are the bending moments in the member for the x-axis and the y-axis, M_{CEx} and M_{CEy} are respectively the expected bending strengths of the member for the x-axis and the y-axis, and m_x and m_y are values reflecting the ductility of the element bending about the same two axes. The axial and flexural strengths of the member are determined using the procedure specified in the AISC Load and Resistance Factor Design (LRFD) Specification for Structural Steel Buildings (AISC 1999), except that the strength reduction factor ϕ is taken equal to 1.0. For axial strength, the lower-bound strength is obtained with the lower bound yield strength which is equal to the mean minus one standard deviation of the yield strengths for a population of similar components. In this study, the nominal value of 300 MPa was used. For the bending moments, the expected flexural strengths are determined with the expected yield strength (330 MPa) and the factors m take a value of 1.25 for flexure of beams and columns.

If the axial compressive force of the structural member exceeds 50% of the lower-bound axial compressive strength, P_{CL}, the compressive and flexural responses of the member are considered force-controlled. In this case, the member must be satisfied:

[4] $P_{UF}/P_{CL} + M_{UFx}/M_{CLx} + M_{UFy}/M_{CLy} \leq 1.0$ when: $P_{UF}/P_{CL} > 0.5$

In equation [4] M_{UFx} and M_{UFy} are the bending moments in the member about the x-axis and the y-axis, respectively, and M_{CLx} and M_{CLy} are the corresponding lower-bound flexural strengths of the member. The moments M_{UFx} and M_{UFy} are obtained by multiplying the first order bending moments from analysis by the magnification factor B_1 specified in the AISC Specification. The flexural strengths M_{CLx} and M_{CLy} are determined using the AISC Specification with a strength reduction factor of 1.0 and the lower-bound yield strength (300 MPa).

The lower-bound strength of brace connections is also calculated in accordance with the AISC Specification and is taken as the lowest design strength, ϕR_n, considering tension yielding, tension rupture and block shear rupture:

[5] $\phi R_n = A_g F_y$ (Gross section tension yielding)

[6] $\phi R_n = A_n F_u$ (Net section tension rupture)

For block shear rupture:

[7] $\phi R_n = \phi\ [0.6 A_{gv} F_y + A_{nt} F_u] \leq \phi\ [0.6 A_{nv} F_u + A_{nt} F_u]$ when: $F_U A_{nt} \geq 0.6 F_u A_{nv}$

[8] $\phi R_n = \phi\ [0.6 F_u A_{nv} + F_y A_{gt}] \leq \phi\ [0.6 A_{nv} F_u + A_{nt} F_u]$ when: $F_U A_{nt} < 0.6 F_u A_{nv}$

In the above equations, A_g and A_n are respectively the gross and net cross section areas of the member at the connection, A_{gv} and A_{gt} are the gross areas subject to shear and tension, and A_{nv} and A_{nt} are net areas subject to shear and tension, respectively. In equation [6], A_n must not be less than $0.85A_g$. Shear lag effects must be considered to calculate the net area. To evaluate the brace connections lower-bound strength, the strength reduction factor ϕ is set equal to 1.0 and the lower-bound yield and tensile strength values are considered, i.e. F_y = 300 MPa and F_u = 450 MPa.

4.3 Assessment results

Figure 4a shows that all braces have sufficient inelastic deformation capacity except those at the 8th and 9th storey. The inelastic demand concentrated in the upper levels of the structure leading to the formation of soft storeys. This type of behaviour is undesirable and should be prevented.

The connection demand-to-capacity ratio is illustrated in Figure 4b. In all braces, the force demand on the connections exceeded the lower-bound connection strength. The governing failure mode in the connections of the first six storeys was brace gross section yielding whereas block shear failure mode governed the behavior of the connections from the 7th to the 10th level. This is contrary to the behaviour observed in the study by Jiang et al. (2012a) where block shear and net section failure modes were identified as critical. Such inconsistencies can be explained by the differences in the procedures used to calculate the connection resistances in the ASCE 41-06 and CSA-S16-09 standards. In ASCE 41-06, the strength reduction factors are set equal to 1.0 for all failure modes to account for the fact that the evaluation is carried out for an existing structure. In CSA-S16-09, the resistance factor is equal to 0.75 for block shear and net section, and it is equal to 0.9 for gross section yielding.

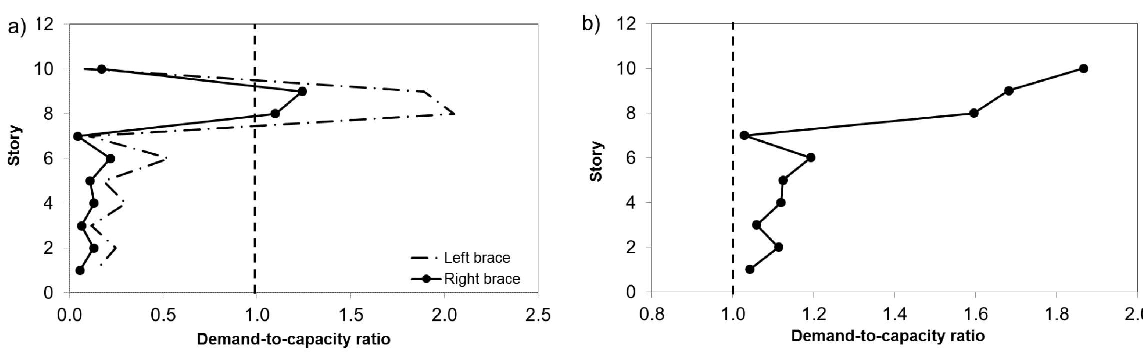

Figure 4: Assessment of: a) the bracing members; and b) the bracing connections

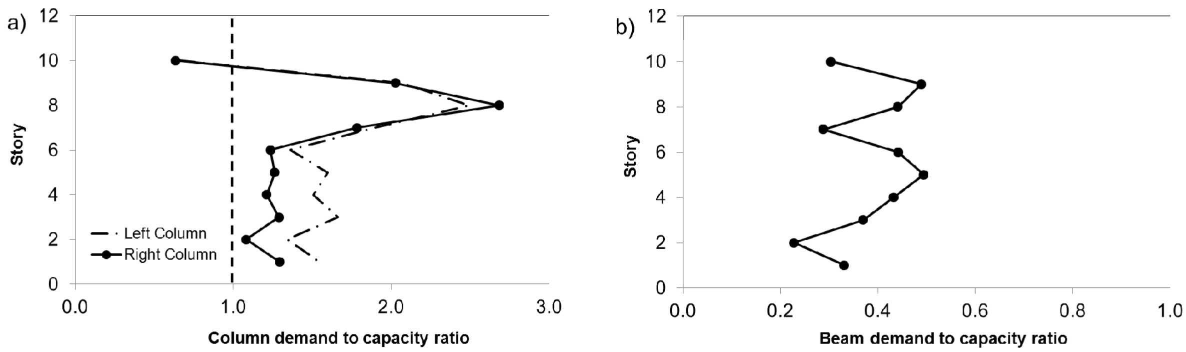

Figure 5: Assessment of: a) the columns; and b) the beams

Demand-to-capacity ratios for the columns are shown in Figure 5a. The value of P_{UF}/P_{CL} is more than 0.2 and less than 0.5 for the columns at the 10^{th} level; thus, Equation [3] was used to evaluate the columns at that storey. At the other levels, the columns were assessed using Equation [4]. None of the columns have sufficient resistance except the columns located at the 10^{th} storey. The beams at the 2^{nd}, 7^{th} and 10^{th} levels were assessed with equation [2] whereas all other beams were evaluated using equation [3]. Figure 5b shows that all of the beams of this tension-only X-braced frame have adequate capacity.

5 Conclusion and future work

The seismic performance of a 10-storey commercial steel building located in Vancouver, BC, and designed in accordance with the requirements of Canadian seismic codes of the early 1980's was assessed using nonlinear time-history analysis in order to identify potential deficiencies. The attention was directed to the tension-only X-braced frame with back-to-back double angle braces. The analytical model for the brace members was developed in the OpenSees program and included gross section yielding, in- and out-of plane buckling of the built-up sections as well as individual buckling of the angles. Inelastic out-of-plane flexural and elastic torsional responses of the gusset plates were also considered in the model, together with the rigidity of the connection zones. A parametric study was carried out to determine which modelling parameters have the most significant impact on brace inelastic behaviour. Using a single-section model overestimated the brace compressive resistance. More reliable representation was achieved using a model where each angle was represented individually. Gap elements, which represent the contact between the angles and permit the free movement of individual angle section when the angles move in opposite directions, did not have impact on the brace resistance because brace buckling was governed by in-plane deformations. Therefore, gap elements were excluded from the model. Other structural members were modelled using elastic beam column elements so that the elastic force demand on these members could be evaluated.

Nonlinear time-history analyses carried out for a set of three ground motion records showed that excessive inelastic deformation demand was imposed on the two braces located at the 8^{th} and 9^{th} storeys of the frame. The frame developed soft-storey response in these levels, which is highly undesirable. The resistance of the columns was exceeded in all but the 10^{th} storey. All of the beams were found to have sufficient strength. All brace connections were found to have insufficient capacity. Different connection failure modes were predicted when the connection resistance was calculated according to ASCE 41-06 and CSA-S16-09 standards. This discrepancy can be attributed to different values of the resistance factors considered in the two design standards. Seismic assessment based on the results of linear analysis appears to be more conservative. A more realistic assessment of the seismic behaviour can be obtained in the nonlinear range because the inelastic brace response can be more adequately represented and the forces transmitted to the other structural members by the buckled and yielding braces are reduced. Nonlinear properties of the material such as strain hardening effects can also be accounted for in this type of analysis.

In future work, the modeling of the brace connections will be refined to include inelastic response and different failure modes. In addition, nonlinear beam column elements will be used for the columns and beams to study the impact of possible nonlinear response of these structural members on the global structural behaviour.

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References

Adluri, S.M.R., and Madugula, M.K.S.1995. Residual Stresses and Initial Out-of-Straightness of Hot-Rolled Steel Angles. *Proc. 1995 SSRC Annual Technical Meeting*, Kansas City, MO: 115-126.

ASCE. 2006. *Seismic Rehabilitation of Existing Buildings*, ASCE/SEI 41-06, American Society of Civil Engineers, Reston, Virginia, USA.

ASCE. 2005. *Minimum Design Loads for Buildings and Other Structures*, ASCE/SEI 7-05, American Society of Civil Engineers, Reston, Virginia, USA.

Atkinson, G.M. 2009. Earthquake Time Histories Compatible with the 2005 National Building Code of Canada Uniform Hazard Spectrum, *Canadian Journal of Civil Engineering*, 36: 991-1000

Bertero, V-V., and Bozorgnia, Y. 2004. *Earthquake Engineering from Engineering Seismology to Performance-Based Engineering*, CRC Press LLC, Boca Raton, Florida, USA

CSA. 1978. *Limit States Design of Steel Structures*, CAN3 S16.1-M78, Canadian Standards Association, Toronto, ON, Canada.

CSA. 2009. *Design of Steel Structures*, CSA-S16-09, Canadian Standards Association, Toronto, ON, Canada.

Jiang, Y., Balazadeh-Minouei, Y., Tremblay, R., Koboevic, S., and Tirca, L. 2012. Seismic assessment of existing steel braced frames designed in accordance with the 1980 Canadian code provisions. In Behavior of Steel Structures in Seismic Area: *Proc. of the Seventh International Conference STESSA 2012*, Santiago, Chile: 531-537.

Jiang, Y. and Tremblay, R, and Tirca, L. 2012. Seismic assessment of deficient steel braced frames with built-up back-to-back double angle brace sections using OpenSees modelling, *Proceedings 15WCEE*, Lisbon, Portugal, Paper 4416.

LRFD.1999. *Load and Resistance Factor Design Specification for Structural Steel Buildings*, American Institute of Steel Construction, Chicago, Illinois, USA.

McKenna, F. and Fenves, G.L. 2004. *Open System for Earthquake Engineering Simulation (OpenSees)*. Pacific Earthquake Engineering Research Center (PEER), University of California, Berkeley, CA. (http://opensees.berkeley.edu/index.html)

Mitchell, D., Paultre, P., Tinawi, R., Saatcioglu, M., Tremblay, R., Elwood, K., Adams, J., and DeVall, R. 2010. Evolution of Seismic Design Codes in Canada. *Can. J. of Civ. Eng*., 37(9):1157-1170.

NRCC. 1980. *National Building Code of Canada 1980*, 8th ed. National Research Council of Canada, Ottawa, ON. Canada.

NRCC. 2010. *National Building Code of Canada 2010*, 13th ed., *1980*, 8th ed. National Research Council of Canada, Ottawa, ON. Canada.

Tremblay, R. 2011. Evolution of the Canadian Seismic Design Provisions for Steel Structures since 1989. *Proc. CSCE Annual Conf.,* Ottawa, ON, Canada: 941-956.