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ADVANCES IN SEISMIC EVALUATION AND RETROFIT OF STEEL BUILDING STRUCTURES UNDER THE CANADIAN SEISMIC RESEARCH NETWORK

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ABSTRACT: This article presents research activities conducted between 2008 and 2013 on the seismic evaluation and retrofit of steel structures within the Canadian Seismic Research Network (CSRN). The research focused on steel braced frames and steel frames with semi-rigid connections built in the 1960-90 era, prior to the implementation of the special seismic design and detailing requirements for steel structures in Canada. Test programs were conducted to generate experimental data on the cyclic inelastic response and deformation capacity of critical seismic force resisting elements and connections. The seismic performance of typical steel building structures was investigated using linear and nonlinear analysis procedures prescribed in existing guidelines. Retrofit schemes were proposed and verified numerically and experimentally, including brace connections with enhanced ductility, brace ductile fuses and use of supplemental damping. Strengthening of existing metal roof deck diaphragms was also examined.

1. Introduction

Special seismic design and detailing requirements for steel framed buildings first appeared in 1989 in the CSA S16 design standard for steel structures (CSA 1989). Prior to that date, it was implicitly assumed that steel frames could withstand earthquake effects through the inherent ductility and energy dissipation capacity present resulting from traditional steel construction practice. In 2008, the Canadian Seismic Research Network (CSRN) was established to conduct research aiming at managing and mitigating the seismic risk to infrastructure in major Canadian urban centres located in seismic active areas. The assessment of the vulnerability of existing structures and the development of retrofit schemes for deficient critical facilities were among the main objectives and tasks of this 5-year research program.

This article provides a summary of research activities conducted on the seismic assessment and retrofit of existing steel building structures constructed before the implementation of seismic design requirements in CSA S16. The research focused on concentrically braced steel frames and steel frames with semi-rigid beam-to-column connections used in Canada in the 1960-90 period. Several large-scale experimental programs were carried out to generate test data on the cyclic inelastic response of critical members and connections used in these steel framing systems. Test results were used to develop acceptance criteria and numerical models for the seismic assessment of existing steel structures using linear and nonlinear analysis procedures. The provisions of the ASCE 41 standard for the seismic evaluation and retrofit of existing structures in the U.S. (ASCE, 2013) were used in some of the projects to examine their application in Canada.

The studies showed that structures built in seismic active regions of Canada before 1990 generally have insufficient lateral resistance compared to minimum seismic load requirements prescribed in current codes. For braced steel frames, brace connection failure and column buckling are the dominant anticipated undesirable governing failure modes. Premature failure of brace members under cyclic loading is also a possibility. Non-ductile failure is also expected for metal roof deck diaphragms used in low-rise steel buildings. Steel frames with semi rigid connections are expected to sustain excessive storey drifts and rotation demands in their connections under seismic loading. These findings are briefly presented and discussed in the article. Innovative retrofit solutions that have been proposed and experimentally verified to address these issues are also introduced. Those include the improvement of the inelastic deformation capacity of brace connections, introduction of ductile fuses in bracing members and use of supplemental damping devices in semi-rigid frames. The experimental and numerical investigations were performed by a group of six researchers from four different universities in Canada, the majority of the projects being carried out in collaboration between the researchers. Details of these projects can be found in the references listed at the end of article.

2. Testing and Numerical Modelling of Concentrically Braced Steel Frames

2.1. Test Programs on Bracing Members and Brace Connections

Several test programs were performed to characterise the inelastic response and deformation capacities of bracing members and their connections. Most tests were performed on angle braces as those were commonly employed before 1990. Specimens built with both single angle and back-to-back double angle cross-sections were examined. The test specimens did not meet current detailing requirements to delay local buckling and prevent connection failures so that the influence of these limit states on the brace cyclic inelastic response could be observed and assessed. Similarly, the stringent stitch requirements now specified to maintain the integrity of double angle bracing members under cyclic inelastic demand were not satisfied either, which also had impact on the brace seismic response.

A test on an individual double angle brace with staggered bolted end connections is shown in Fig. 1. The angles are L127x76x9.5 and the brace length is 6095 mm. The brace exhibited a ductile overall response with gross section yielding in tension. Failure eventually occurred in tension at the connection net section (Fig. 1b) at a total elongation of 2.2% of the brace length, corresponding to a ductility of 13 based on the measured yield strength (Fig. 1d). Prior to failure, localized yielding took place in both end connections (Fig. 1c), contributing in total 0.32% L axial deformation, i.e. 15% of the global brace ductility (Fig. 1e).

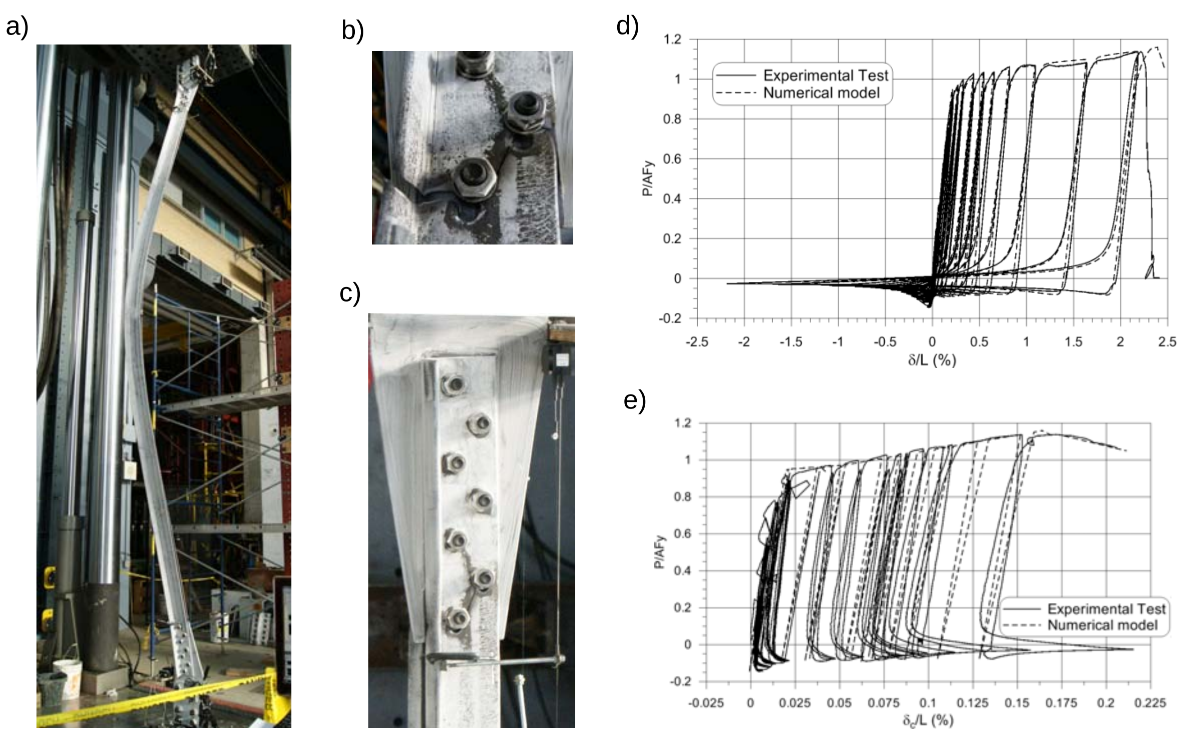


Fig. 1 – Testing of an individual double angle brace with bolted connections; a) Overall buckling; b) Net section failure; c) Plasticity concentrated in connection; d) Overall brace hysteresis; and d) Hysteresis response of the failed connection (Jiang, 2013).

For this brace, calculations predicted that net section failure would occur at a load of 1.05 times the brace gross yield strength. In the test, the connection strength was in fact higher than expected (1.14 AF_y) and connection failure only took place after substantial gross section yielding has developed and higher tension force developed due to strain hardening. In another test performed on an identical specimen built with a single angle (Morrison, 2013), net section failure took place under a tension load of 1.05 AF_y at a 0.62% L axial deformation corresponding to a ductility of 3.7. For this specimen, yielding in the connections contributed to half the brace deformation capacity. Differences between these two tests show that seismic performance assessment can be sensitive to assumptions made in the calculations or modelling, especially when comparable resistances are predicted for different limits states, and that sufficient margin should be considered to cope with unavoidable uncertainties on material and geometrical properties.

In Fig. 2, full-scale cyclic testing was performed on X-bracing constructed with the same double angle bracing members and connection details. In this case, the reduced unsupported brace length resulted in more pronounced curvature of the brace segments upon brace buckling, which produced longitudinal shear failure of the welded stitches and local buckling of the angle outstanding legs (Fig. 2b). In two specimens, local instability took place in the mid-connection, as shown in Fig. 2c, which prevented buckling of the discontinuous brace segments. This however induced large flexural strain demands in the mid-connecting plates that eventually led to low-cycle fatigue failure of the plates (Fig. 2d). In one specimen, this failure occurred at a brace axial ductility of 3.3. The second specimen was designed with thinner plates that sustained inelastic bolt bearing deformations prior to fatigue plate failure, and the brace overall ductility was increased to 5.0. A third specimen was designed with a thicker plate at its mid-connection and buckling of the discontinuous brace segments was observed. Failure of the specimens occurred in the braces at a ductility of 3.3 due to local buckling and low-cycle fatigue in plastic hinge regions.

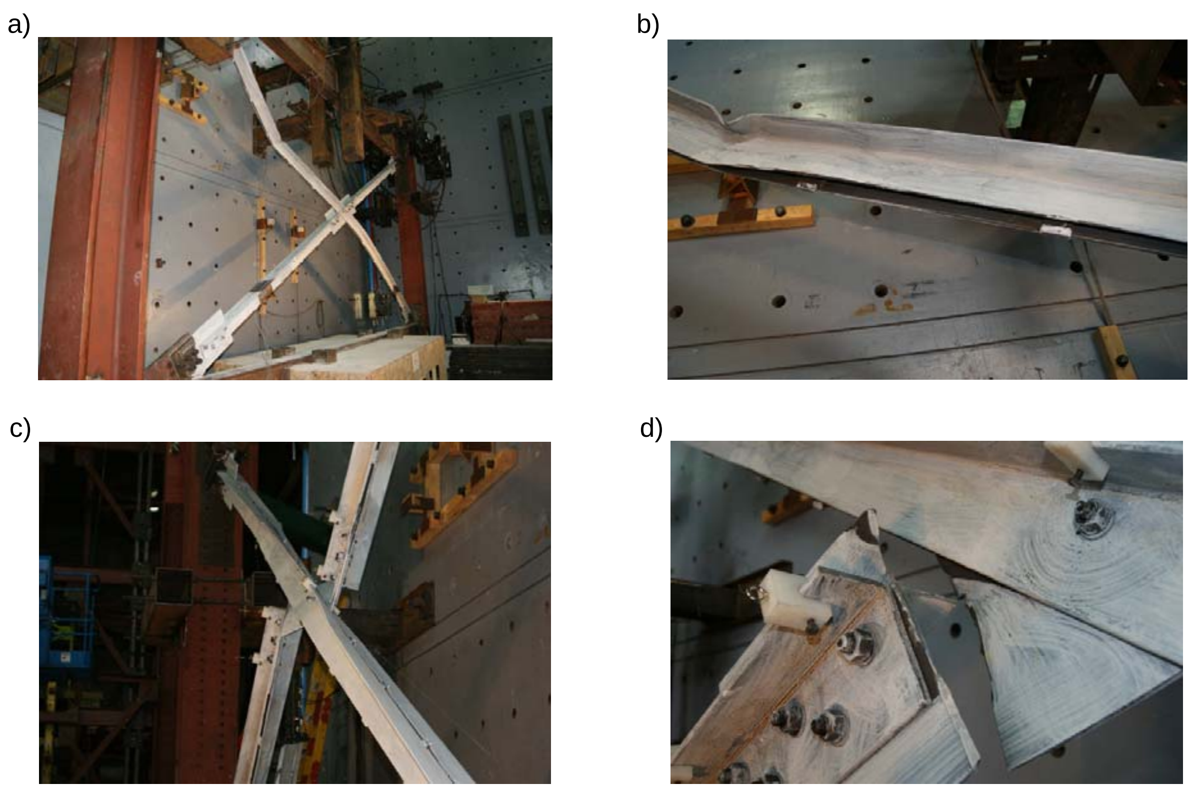
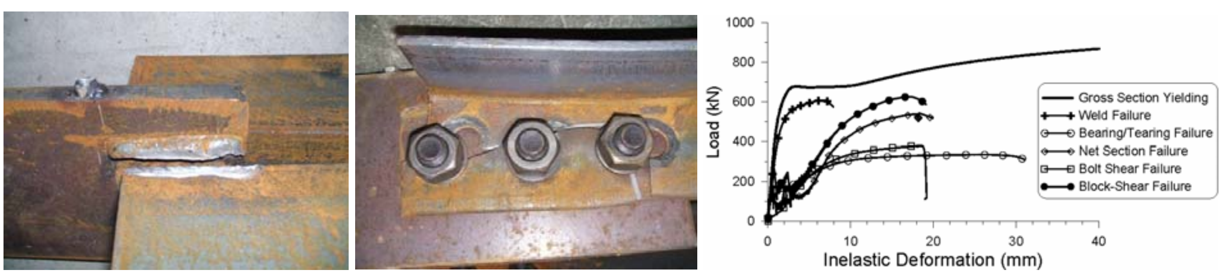


Fig. 2 – Testing of an X-braced frame with double angle braces: a) Overall brace responses; b) Local buckling of angle sections and failure of connecting stitches; c) Instability at mid-connection; and d) Low-cycle fatigue failure in tension on the gross section of the mid-connection plate (Gélinas et al., 2013).

In ASCE 41, seismic performance can be assessed through linear or nonlinear analysis procedures. In the former method, deformation-controlled actions such as axial deformations in braces, ductility (“m”) factors are used to evaluate the adequacy of the structure. For bracing members, the m factor in ASCE 41 varies depending on whether tension-compression or tension-only bracing is used, the type of brace section and connection detailing. The validity of this approach was confirmed by the marked variations in brace ductility observed in the aforementioned test programs. However, discrepancies were also noticed between the ASCE 41 m values and the test results, suggesting that additional research is needed to develop values that more closely reflect the actual inelastic deformation capacity of bracing members and their connections.

In ASCE 41, brace connection response is typically considered as a force-controlled action in view of the limited ductility associated to connection failure, and acceptance is therefore based on strength rather than ductility or deformation capacities. However, it is permitted to classify brace connections as deformation-controlled components when inelastic deformation concentrates in connections, as was observed in the brace test programs and is anticipated in older braced steel frames where connections are not designed to resist the brace yield tensile strengths. Experimental programs were therefore performed on brace connections to assess their inelastic deformation capacities (Hartley et al., 2012). A test series was performed on identical double angle brace specimens with various connections designed to develop pre-determined failure modes so that direct comparison between various limit states could be made (Fig. 3a). Bolt bearing produced the largest ductility whereas weld failure exhibited the lowest. Other failure modes showed comparable intermediate inelastic deformation capacities. As shown, slippage also contributes to deformations of bolted connections. Cyclic tests were also performed on single angle brace with bolted connections collected from existing structures that were demolished or retrofitted. Tension failure on net section dominated the response. As can be seen in Fig. 3b, failure of some specimens was very brittle, with limited plastic deformations in the connections. Coupon testing showed that the steel of these older brace samples generally had high tensile to yield strength ratios but much smaller elongation at rupture compared to minimum values specified today for structural steels (7% for the sample shown in Fig. 3b). Testing of actual samples is therefore recommended if connection ductility is to be considered in seismic performance assessment of a structure. Caution should also be exercised when incorporating connection deformation capacities in the calculation of the ductility (“m”) factor for the braces as these deformation capacities are fixed and do not vary with brace lengths.

a)



b)

Fig. 3 – Tests on brace connections: a) Weld and block shear failures and inelastic deformation capacity under monotonic tension loading (Castonguay and Tremblay, 2010); b) Brittle tension failure on net section for an 1960’s single angle brace (Caruso-Juliano, 2012).

2.2. Numerical Modelling of Steel Braced Frames

Seismic assessment in ASCE 41 can also be performed using nonlinear analysis. As a minimum, the hysteretic response of deformation-controlled actions such as brace axial response is explicitly considered in the analysis and the adequacy of these components is verified by comparing the predicted inelastic deformation demands to deformation capacities determined from idealized backbone force-deformation curves defined from cyclic test results. Force-controlled actions are verified using force demands obtained from analysis. For collapse performance assessment, more refined models that include strength degradation and failure responses of the critical structural components can be used so that structural failure and collapse are directly verified in the analysis. In either case, accurate representation of the inelastic cyclic response of the deformed-controlled components is needed to predict the seismic deformation and force demands in the structure.

Results from experimental programs performed in the CSRN and previous research projects were used to develop reliable numerical models that can be used for the seismic assessment of concentrically braced steel frames. The models were mainly developed on the OpenSees platform (Mazzoni et McKenna,. 2004), a structural analysis program dedicated to nonlinear seismic analysis, and focus was put on the modelling of braces and brace connections. Brace models typically use force based beam-column elements with fiber discretization of the cross-section, as illustrated in Fig. 4 for a double angle bracing member. A material is assigned to the fibers that accounts for yielding, isotropic and kinematic strain hardening effects, as well as residual stresses. Several elements are used along the member length and initial out-of-straightness is specified so that inelastic buckling response can be adequately reproduced. In Fig. 4, stitches are also modelled to include interaction between the two angles forming the braces. Nonlinear spring elements are used at the brace ends to include the flexural response of the gusset plates upon brace buckling and the inelastic behaviour of the connections. Comparisons with test results are shown in Figs. 1d&e. Prediction of the seismic response of a 4-storey steel braced frame with this model was verified through full-scale hybrid testing (Fig. 4b).

a)

b)

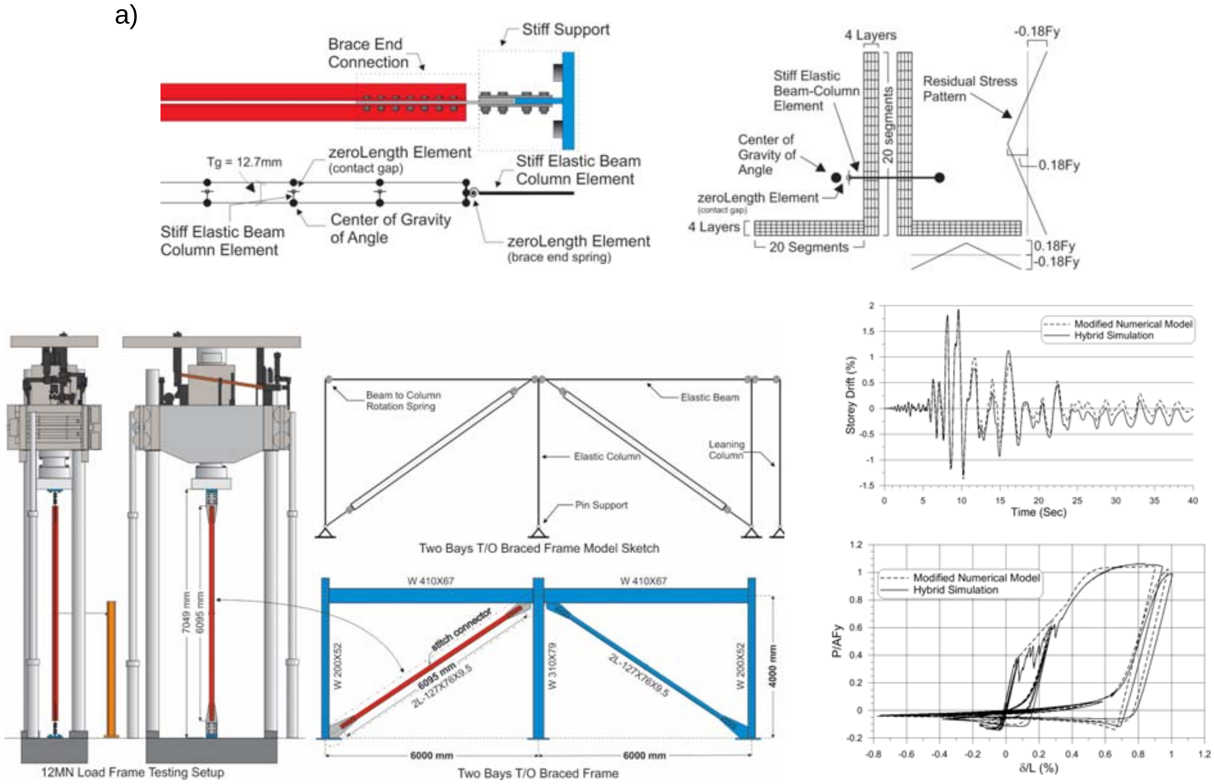


Fig. 4 – Braced frame model with double angle bracing members: a) Brace and connection model (Jiang et al., 2012); b) Model validation using hybrid simulation (Tremblay et al., 2012).

3. Seismic Assessment of Concentrically Braced Steel Frames

3.1. Evolution of Code Provisions

Seismic provisions in Canada have evolved considerably over the last decades, with significant consequences on design seismic loads (Mitchell et al., 2010). The evolution of the design base shear specified in the National Building Code of Canada (NBCC) is illustrated in Fig. 5 for steel braced frames on firm ground sites in Vancouver, BC, and Montreal, QC. For consistency, a load factor of 1.5 was applied to the loads determined using the 1953 to 1985 editions of the NBCC. For the 1990-2010 NBCC editions, force modification factors R and R_d equal to 1.5 were used as systems qualifying for this factor correspond most to those designed before the introduction of special seismic design and detailing requirements in CSA S16 in 1989. For the NBCC 2010, design seismic forces were amplified as required in CSA S16-09 for structure heights above 15 m (CSA, 2009). The comparison in Fig. 5 is not complete as it does not reflect changes to other code provisions such as system restrictions (height limits), importance factors, allowances for reduced design loads when using computed building periods, vertical load distributions, in-plane torsion effects, etc. Nevertheless, it clearly shows that before 1990, structures were designed for much lower earthquake loads compared to the minimum seismic resistance required in 2010 for new buildings.

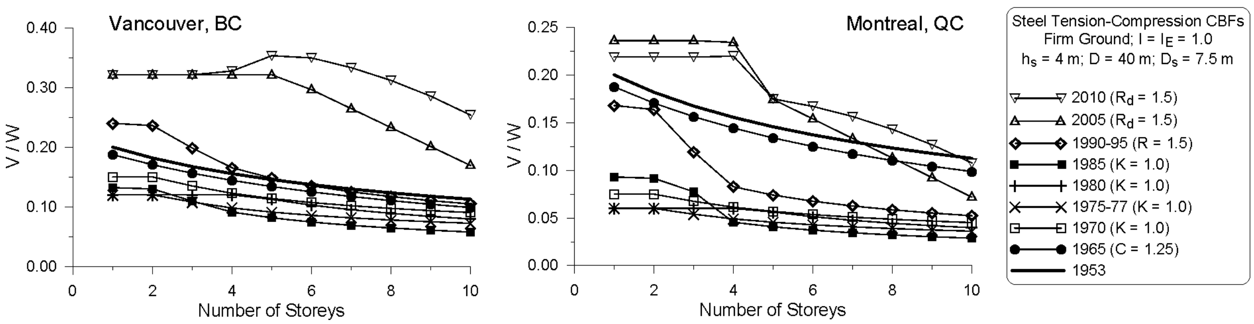


Fig. 5 – Evolution of design earthquake loads in Vancouver and Montreal for tension-compression steel braced frames of the conventional construction on firm ground.

For the cases studied in the figure, lack of lateral resistance is greater for structures designed in accordance with NBCC 1970 to NBCC 1985. For that period, the ratios between design seismic loads and 2010 values vary from 0.25 to 0.53 in Vancouver and from 0.21 to 0.63 in Montreal. For the most part, this is lower than the load factor of 0.6 recommended as a trigger criterion for seismic upgrading in Commentary L to NBCC 2010 (NRCC, 2010). Moreover, detailed assessment of these structures is likely to reveal additional deficiencies in view of the several changes that have also taken place over the last 60 years in the CSA S16 standard. For instance, design equations for shear lag effects and block shear failure in connections were only implemented in the 1989 and 2001 editions of CSA S16, respectively. Lastly, the structures will probably not satisfy the minimum seismic detailing requirements that are now specified in CSA S16 for seismic force resisting requirements designed with a force modification factor R_d of 1.5. It is therefore expected that retrofitting such deficient braced steel frames to current earthquake code requirements will require major strengthening and/or modifications. For some structures, this type of retrofit solution may not be easily applicable and alternative seismic assessment procedures that more explicitly account for the actual ductility or resistance of key structural components may lead to more cost-effective solutions. Methods based on incremental nonlinear dynamic analysis or procedures proposed in ASCE 41 can be used for this purpose. Those have been applied for single-storey and multi-storey braced steel frame buildings as part of CSRN research projects.

3.2. Single-Storey Steel Braced Frames

One project examined the seismic performance of single-storey buildings for commercial and light industrial usages that were designed according to the 1965 NBCC (Juliano-Caruso et al., 2014). Lateral loads are resisted by X-bracings with single angle members and a metal roof deck diaphragm (Fig. 6a). In Canada, roof decking is generally built with 38 mm deep x 914 mm deck panels having a trapezoidal section with flutes spaced 152 mm apart. Until recently, deck panels produced with 0.76 or 0.91 mm thick steel were used, as required to resist roof gravity loads, and diaphragm action was achieved by using button punched connections along the panel sidelaps and puddle welds fusing the deck to the supporting steel beams or joists (CSSBI, 1972). The study confirmed that both the vertical bracing and the roof diaphragm of typical structures located in Vancouver and Montreal regions had insufficient lateral resistance when compared to current code minimum strength requirements.

The expected seismic performance of the structures was assessed by determining the collapse margin ratio resulting from failure of the vertical bracing under the design earthquake level. The procedure proposed in FEMA P695 was used to achieve this (ATC, 2009). In the method, nonlinear response history analysis is performed under an ensemble of ground motion time histories that are initially scaled to represent the design level. These analyses are repeated under stepwise incremented ground motion amplitudes to identify the intensity level causing structural collapse. Numerical models capable of accurately reproducing brace hysteretic responses and brace connection failures are required for these analyses, as described in the previous section. For each ground motion intensity level, the number of ground motions causing collapse is noted and the data is used to fit a fragility curve that gives the probability of collapse as a function of seismic intensity. Using this curve, a collapse margin ratio (CMR) is determined which corresponds to the ground motion amplitude relative to the design level that would result in median collapse, i.e. 50% of the ground motion time histories causing collapse. According to FEMA P695, the CMR value is then adjusted to account for spectral shapes of the ground motions and the shape of the fragility curve is modified to account for possible sources of uncertainty affecting the variability in collapse capacity. The seismic performance can then be evaluated using acceptance criteria also proposed in the FEMA P695 procedure.

An example is shown in Fig. 6b. In this case, connection failure and structural collapse were not explicitly modelled in the analyses. Instead, the number of ground motions inducing brace connection deformations larger than the deformation capacity associated to each failure mode was recorded so that fragility curves could be developed for every limit states. The analysis models included the structure roof diaphragm as the flexibility of the diaphragm can significantly affect the demands on the vertical bracing (Tremblay and Stiemer, 1996; Humar and Popovski, 2013). In the figure, NS1 represents the brittle failure mode on net section that was observed in the tests on brace connection assemblies collected from existing buildings (Fig. 3b). As shown, the CMR for this failure is approximately 0.8. Larger CMR values are found for the more ductile failure modes. In this study, the procedure confirmed that the vertical bracing of the structures located in the Vancouver region would have unsatisfactory performance and require seismic upgrade. Conversely, structures located in Montreal would have satisfactory performance and would not need retrofit in spite of insufficient lateral resistance based on NBCC 2010 provisions. This result for Montreal was attributed to the smaller deformation demands imposed by the high frequency ground motions expected in eastern Canada, an effect that cannot be captured when examining force demands.

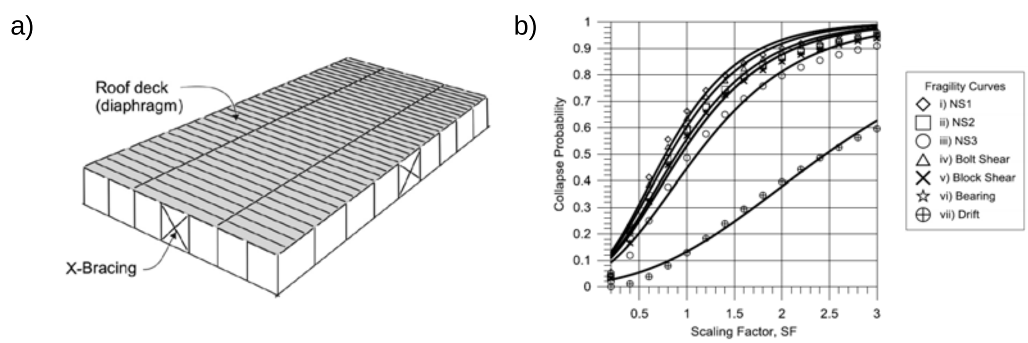


Fig. 6 – Seismic collapse assessment of 1965's single-storey steel buildings using incremental dynamic analysis; a) Buildings studied; b) Fragility curves for different brace connection failure modes (Juliano-Caruso, 2012).

3.3. Multi-Storey Steel Braced Frames

Performance assessment using incremental dynamic analysis was also used for multi-storey steel braced frames designed using the 1980 edition of the NBCC. The example shown in Fig. 7 is a 4-storey frame with double angle tension-only braces located in Vancouver. The braces are connected with high strength bolts. Preliminary assessment using elastic response spectrum analysis had indicated that failure at the net section of the brace connections in the first three storeys would be critical for this structure. Nonlinear analysis was performed to confirm this behaviour and determine if connection failure could cause structural collapse. The brace numerical models described earlier were used in the analyses. In this case, brace connection failure was explicitly modelled in the analysis so that structural collapse could be directly assessed. As illustrated, collapse by global instability occurred as a result of large storey drifts developing at levels where brace connection failures occurred. The analysis model therefore included the gravity system as the continuity of the gravity columns and partial rotational restraint at beam-to-column connections can contribute to frame stability. Inelastic axial-flexure interaction was included in the gravity column models to predict the formation of plastic hinges at columns' top and bottom ends, and the hysteretic response of the beam-to-column joints including contact of the floor slab against the columns was also incorporated in the analysis. The braced frame columns were modelled using the same technique as the bracing members so that inelastic buckling under high axial compression forces induced by the braces combined with flexural demands from storey drifts varying along the structure height could be captured.

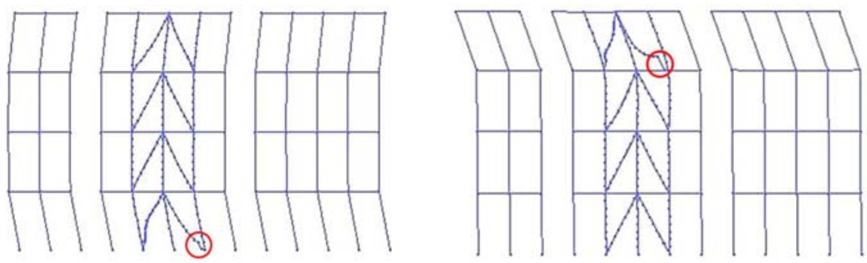


Fig. 7 – Structural collapse of a braced steel frame due to brace connection failure (Jiang, 2012)

Connection failure was critical, as predicted from seismic assessment using linear dynamic response spectrum analysis. However, failure in the fourth storey dominated the response, likely due to inelastic higher mode response, a situation that could not be predicted from linear assessment. Buckling of the braced frame column was not observed but column hinging formed in all columns resulting in the storey collapse mode shown in Fig. 7. Application of the FEMA P695 procedure confirmed the inadequacy of the structure for collapse prevention under the design earthquake level. A similar study was performed by Tirca et al. (2015) on 3- and 6-storey tension-compression braced frames.

Seismic performance assessment was performed using the ASCE 41 linear and nonlinear analysis procedures for the 10-storey building shown in Fig. 8a. ASCE 41 is written in such a way that seismic assessment can be performed for various performance objectives (collapse prevention, life safety, etc.) and level of seismic hazard (2% in 50 years, 5% in 50 years, etc.). Guidance is provided in the document but the performance objectives and hazard levels can be selected by the owner, the engineer, and/or the authority having jurisdiction depending on factors such as the remaining life of the structure and the cost of the repairs *vs* the expected benefits on safety resulting from the retrofit. No such guidance is available in the NBCC to establish proper assessment criteria for the Canadian context. For the example shown herein, collapse prevention under 2% in 50 years earthquakes was considered, as currently required in the NBCC for new buildings. The performance of the tension-only X-bracing in the N-S direction of the building located in Vancouver, BC, is discussed herein. The performance of the chevron bracing for the same building is presented in Balazadeh-Minouei et al. (2014b).

The frames are built with back-to-back double angle braces with bolted connections. Again, brace connections were found to be critical for this structure. However, the assessment was performed assuming that the connections had sufficient strength so that seismic performance of members could be evaluated. Demand-to-capacity ratios for the bracing members as obtained from different methods are plotted in Fig. 8b. Preliminary assessment using NBCC 2010 and CSA S16-09 with R_d = 1.5 showed that braces at all levels but the roof would need to be strengthened to match current code requirements. Compared to the equivalent static force method (ESFM), lower demand is obtained from dynamic response spectrum analysis (RSA). Response spectrum analysis is also adopted when following the

ASCE 41 linear assessment procedure and a similar trend is obtained. However, using the expected member resistances and the ductility (“m”) factor for the braces in ASCE 41 significantly reduced the required upgrade effort, for both the number of braces and the required level of strengthening. Nonlinear assessment was also performed using response history analysis under an ensemble of ground motions scaled to the design level. In this case, only the braces at levels 8 and 9 need to be reinforced when comparing brace axial deformation demands and capacities, clearly showing the possible benefits from using a more refined method of analysis.

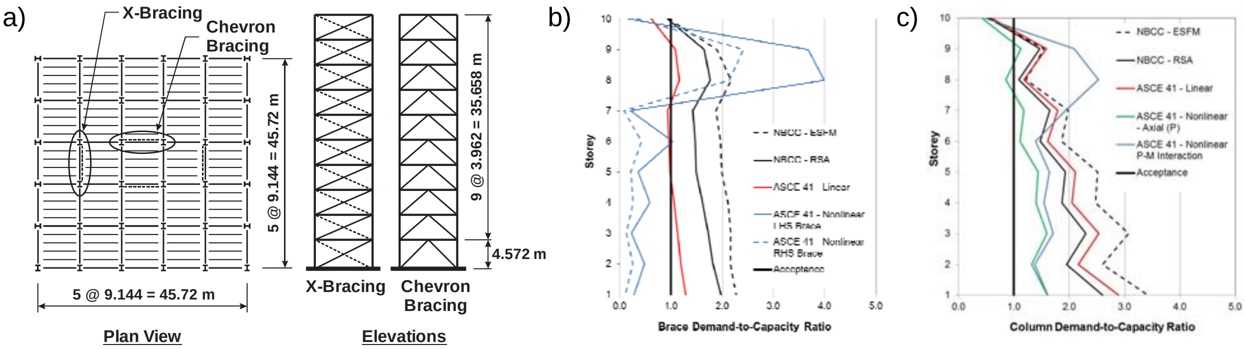


Fig. 8 – Seismic assessment of 10-storey X-braced frame: a) Building studied; b) Assessment of the braces; and c) Assessment of the columns (Balazadeh-Minouei et al., 2013, 2014a).

Assessment results for the braced frame columns are presented in Fig. 8c. According to current codes, the columns would need to be reinforced at every building levels except the 10^{th} level. The same conclusion is drawn from the ASCE 41 linear assessment procedure, essentially because column axial response is a force-controlled action and column axial loads corresponds to the forces from elastic response spectrum analysis that are reduced using the demand-to-capacity ratios of the braces located above the level under consideration. For this frame, the resulting forces are similar to those obtained by dividing elastic forces from analysis by the code force modification factors R_o and R_d. When performing nonlinear assessment using hysteretic models for the braces, the axial load demand on the columns reduce considerably, especially towards the structure base where the likelihood of having all braces reaching simultaneously their capacities is reduced. Nonlinear analysis however revealed large flexural demands on the columns at the 8^{th} and 9^{th} levels due to large storey drifts resulting from the large brace ductility demands at these levels (Fig. 8b). Such a behaviour cannot be predicted from elastic analysis. Additional nonlinear analysis was run with nonlinear models for both the braces and the columns to examine the consequence of this high flexural demands on column stability. In these analyses, column buckling occurred at the first storey, when the axial load exceeded the column axial strength. Plastic hinging formed at the column ends at the 8^{th} and 9^{th} levels but no buckling occurred likely because the axial load remained smaller than the column buckling strength and the columns deformed in double curvature, a less critical condition for buckling. Column stability under such loading conditions is still an area of research at the time of writing.

In general, the CSRN studies indicated that seismic performance using nonlinear assessment gives a more realistic representation of the seismic demands and allows to better target areas of the structure that really need to be fixed. Nonlinear assessment however requires reliable numerical models that can properly reproduce the hysteretic response of deformation-controlled elements under cyclic inelastic seismic loading. For both the linear and nonlinear assessment procedures, the studies also showed that the results are sensitive to material properties and the deformation or force capacities assigned to the elements. Proper assessment therefore also relies on a good knowledge of these parameters.

4. Seismic Retrofit of Steel Braced Frames

4.1. Retrofit Design

Different retrofit strategies were examined for the deficient braced frames studied in the projects described in the previous section. For the most part, it was found that a retrofit design follows the steps illustrated in Fig. 9. Once the assessment has been completed, deficient deformation-controlled actions are first addressed because corrective actions taken for these components will likely affect the force demand imposed on the force-controlled components of the structure. This is similar to the situation encountered when performing seismic capacity design for new structures. For a deficient deformation-controlled action, retrofit can be performed by either: 1) increasing the strength of the component with the objective of satisfying minimum strength requirements and/or reducing the inelastic deformation demand imposed to that component; or 2) increasing the deformation capacity or ductility of eh component so that it can accommodate the anticipated deformation demand. For a brace, strengthening may consist in reinforcing the existing member or replacing the member by a new, stronger one. The main drawback of this first option is that the modification will typically result in higher force demand being imposed to force-controlled components such as brace connections, columns, beams, roof and floor diaphragms, and foundations. Hence, improvements to the deformation-controlled elements may worsen the situation for the remaining elements along the lateral load path.

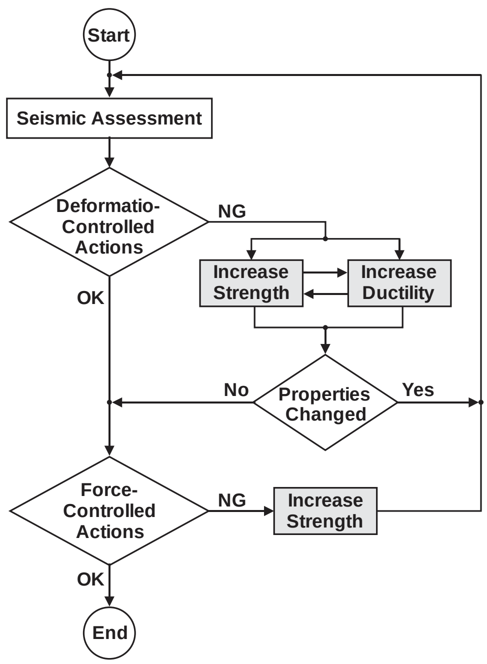


Fig. 9 – Retrofit design procedure

In the second option, the bracing member can be repaired or replaced by a new member so that it has a higher "m" factor or exhibits larger inelastic deformation capacity. In that case, the force demand on adjacent force-controlled components can be kept the same as, or close to the value in the existing structure. This may limit the extent of retrofit required for the adjacent force-controlled actions. As shown in Fig. 9, the adopted retrofit scheme may be two-fold to limit further the force-demand in other critical components. For instance, a strength-deficient brace can be replaced by a stronger and more ductile member so that the minimum required strength for the new brace is reduced when considering its greater ductility. This could be the case when non-ductile weak braces are replaced by buckling restrained braces instead of regular tubular braces. Similarly, enhancing the ductility of a bracing member can be accompanied by a decrease in the brace actual resistance. This could be achieved, for example, by introducing ductile fuses in existing braces.

After an initial retrofit solution is proposed for the deformation-controlled actions, seismic performance assessment of the retrofitted structure should be redone because the changes made are likely to affect the structure response and the seismic demands on members and connections. If linear assessment was used in the first iteration, nonlinear analysis may be performed for the second one to improve further the initial retrofit scheme for the deformation-controlled actions. Force-controlled actions are then verified and reinforcement is designed as required from the analysis results. Seismic performance assessment of the structure is performed again until no further modification is required. As discussed, although brace connection response is usually considered as force-controlled, it may be treated as a deformation-controlled action when dependable plastic deformation capacity already exists or can be easily mobilized in brace connections. It is noted that the chart in Fig. 9 would apply for the common case where structural deficiencies can be addressed without major alterations to the existing lateral load resisting system. Examples of such retrofit schemes for steel braced frames that have been studied in the CSRN projects are discussed in the next section. Other retrofit approaches may be more suitable for certain situations. For instance, excessive seismic deformation demands on critical components could be more effectively addressed by laterally stiffening the structure (addition of braced frames or shear walls) or adding supplemental damping. Application of supplemental damping is discussed in Section 5 for moment frames with semi-rigid connections.

4.2. Retrofit Solutions Studied for Steel Braced Frames

Tirca et al. (2015) examined the seismic performance of retrofitted 3- and 6-storey braced steel frames that had been designed in accordance with the 1980 codes for Vancouver, Montreal, and Quebec City. Nonlinear assessment was performed using the FEMA P695 procedure for the structures in the existing and retrofitted conditions. For the existing structures, the brace connections were too weak and their failure caused collapse of the buildings. The connections were reinforced which, in turn, required strengthening of the beams and columns in view of the higher forces imposed by the braces reaching their axial compressive and yield tensile strengths. Fragility curves for low, moderate, and severe damage limit states are plotted for the 3-storey frame in Quebec City in Fig. 10a. As illustrated, the retrofit scheme improved significantly the performance of the structure, especially for the moderate and severe damage limit states.

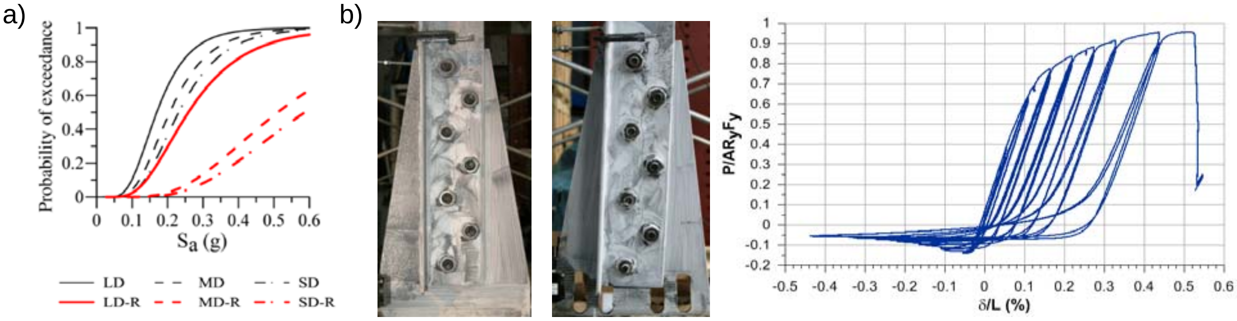


Fig. 10 – Fragility curves for low damage (LD), moderate damage (MD) and severe damage (SD) for a 3-storey braced frame in the existing and retrofitted conditions in Quebec City (Tirca et al., 2015); b) Retrofit of 4-storey tension braced frame through ductile tension yielding in the brace gusset plates (Jiang, 2013).

A retrofit scheme where short ductile fuses are created in the gusset plates of brace connections was examined for the 4-storey tension-only braced frame discussed earlier and shown in Fig. 7. The retrofit was verified experimentally in the cyclic test programs performed on individual 6.095 m long braces as shown in Fig. 1a. The original and retrofitted brace connections are shown in Fig. 10b. For the latter, four slotted holes parallel to the brace longitudinal axis were created in the end gusset plates to concentrate yielding in the gusset plates at a tension load smaller than the strength of the bolted brace to gusset plate bolted connections. The slots were 25 mm in diameter and had a total length of 76 mm. They were fabricated as would be done in the field, i.e. by first drilling four rows of three 25 mm circular holes and then cutting and grinding the remaining material between the holes. The overall brace-connection hysteric response is shown in Fig. 10b. As intended, yielding in tension only took place in the end gusset plates while brace buckling occurred in compression. The peak tension load remained below the existing bolted brace connection capacity and failure took place in the reduced section of one of the gusset plates. At failure, the total deformation in both connections reached 23 mm, representing 0.37% of the brace length. That retrofit produced a more ductile behaviour compared to that measured for existing braces in Fig. 3b.

Within the CSRN program, significant efforts were devoted to the development and testing of ductile fuses that can be implemented in bracing members. Similar to the gusset plate retrofit, the objective is to achieve ductile brace response while reducing the forces applied to the brace connections and other force-controlled components of the structure. The fuses developed can be used for the retrofit of existing structures as well as for the construction of new braced frame structures (Tremblay et al., 2011). The various schemes studied are illustrated in Fig. 11. For HSS bracing members, a fuse design was proposed in which the brace is cut near one of its end connections and the two brace segments are connected together using four angles with reduced cross sections that control the force and develop ductile yielding in tension (Fig. 11a). The fuse is enclosed in built-up box section to prevent outward buckling of the angles and maintain the overall brace buckling compressive resistance. In Fig. 11b, locally reducing the cross section was also proposed as a fuse for single angle and double angle bracing members. As shown, ductile failure of the angle occurs in the reduced section segment, away from the brace connection, as intended in design. In a subsequent test program, Desrochers (2014) showed that the ductility of the fuse detail could be improved further by inserting the reduced brace segment within a HSS member to prevent fuse local deformations. A third fuse detail is shown in Fig.11c which consists in a ring element that can be simply obtained by plasma cutting of a steel plate. The system can be used in single diagonal braces as well as at the intersection of braces in X-bracing. The system is intended for tension-only bracing; however, the shape and details of the ring can be tailored to achieve force and deformation capacities to match the specific retrofit requirements for a wide range of applications. Nonlinear seismic assessment using the FEMA P695 procedure showed that this fuse design would result in satisfactory seismic performance for the deficient 1965's single-storey buildings described earlier and shown in Fig. 6 (Morrison and Rogers, 2013).

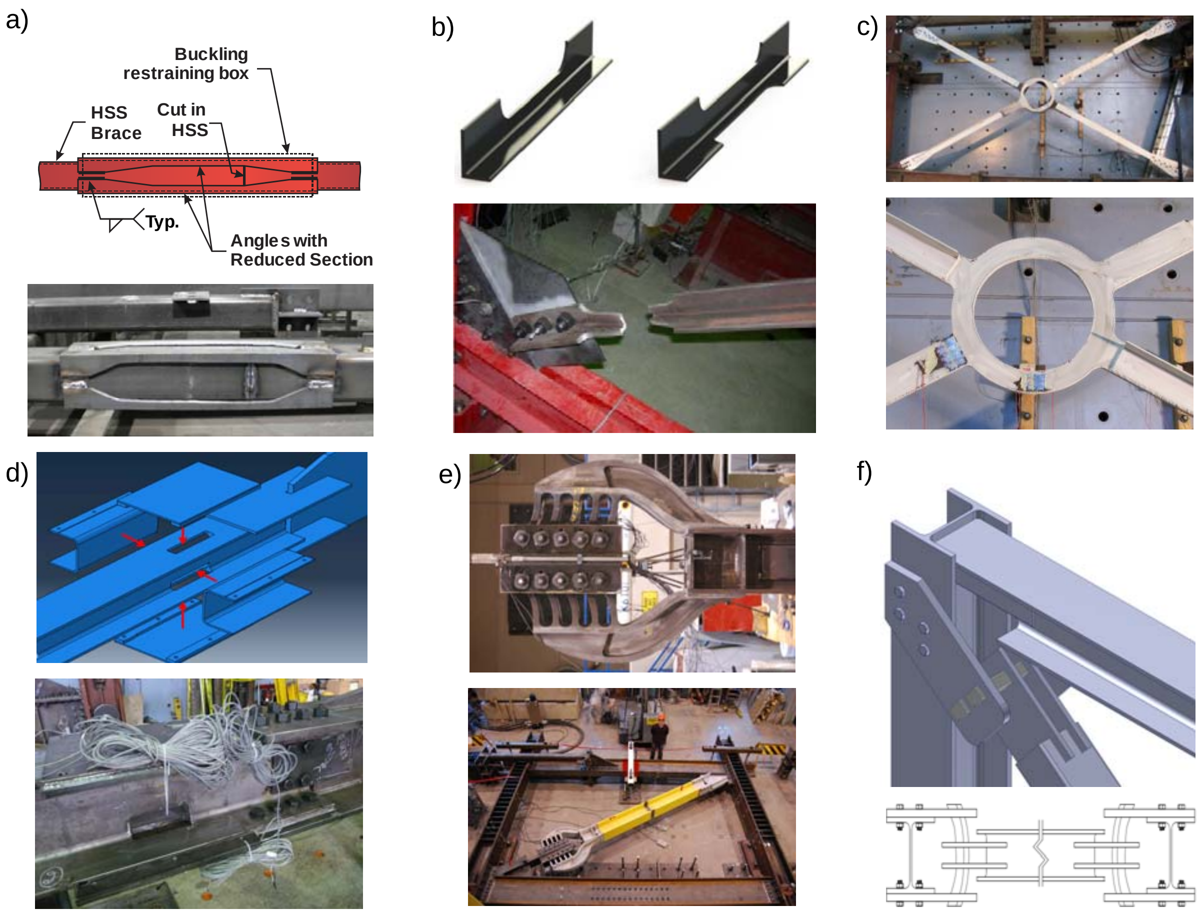


Fig. 11 – Ductile fuse systems studied: a) HSS fuse with reduced angle sections (Kassis and Tremblay, 2008; St-Onge, 2012); b) Angle braces with locally reduced cross-sections (Légeron et al., 2014); d) Ring fuse system (Morrison, 2014); d) Fuse design for W shaped braces (Egloff, 2013); e) Cast steel yielding brace system (Gray et al., 2014); and f) Pin fuse (Tirca et al., 2012).

The fuse design in Fig. 11d was developed for heavy W-shaped bracing members. As shown, the axial strength of the brace member is reduced by locally removing material at the flange to web intersections to minimize the impact on the brace flexural stiffness and buckling resistance. The fuse segment is also confined using back-to-back cold formed C-shapes and outer plates to prevent local buckling of the brace cross-section in compression. The design methodology proposed for the fuse was validated through finite element studies prior to performing physical testing of brace fuse assemblies (Egloff et al., 2012). In the ductile brace fuse system shown in Fig. 11e, seismic energy is dissipated by the yielding fingers of a specially engineered cast steel connector. In contrast with the previous fuse designs, the fuse is designed to yield both in tension and compression. Brace buckling is therefore prevented and the system exhibits a symmetrical hysteretic response. In Fig. 11f, the fuse is built with ductile pins connected to the bracing and other framing members with steel plates. The assembly is designed so that the pins yield in flexure prior to reaching the buckling and tensile yield strengths of the brace. Hence, this system also has symmetrical response in tension and compression. The single-pin detail was originally developed in Europe and the research focused on designs with multiple pin arrangements aimed at increasing strength and deformation capacities as well as system redundancy.

As discussed, seismic assessment studies revealed that metal roof deck diaphragms with button punched sidelap connections and frame puddle welds as used in older low-rise steel buildings generally have insufficient lateral seismic resistance. Previous cyclic tests of steel deck diaphragms by Tremblay et al. 2004 had also revealed that roof deck diaphragms of this type have limited inelastic deformation capacity, meaning that they would classify as force-controlled components and would need to be strengthened for retrofit. As part of the CSRN project, an ongoing diaphragm dynamic testing program on large-scale diaphragm assemblies was expanded to include tests on a retrofitted diaphragm. The test specimens were 7.3 m x 21 m (Fig. 12a) and the diaphragms were built 38 mm deck panels and seismic excitation was introduced by two high performance dynamic actuators located at the diaphragm ends. In the tests, horizontal inertia forces were therefore induced along the diaphragm span as would be the case in a real building. The results of two specimens are shown in Fig. 12c. The original diaphragm (Test 10) was built with 0.76 mm thick steel deck panels connected with 16 mm puddles welds and button punches at 305 mm o/c. For the retrofitted specimen (Test 11), power actuated fasteners were installed from underneath next to every welds and the sidelaps were reinforced using 1.21 mm thick, 50 mm wide steel strips with pairs of self-tapping screws spaced 305 mm o/c (Fig. 12b). Additional frame fasteners were also used at the sheet end overlaps to reduce warping deformations and increase shear resistance. As shown, the retrofit studied significantly increased both the shear stiffness and shear strength of the diaphragm.

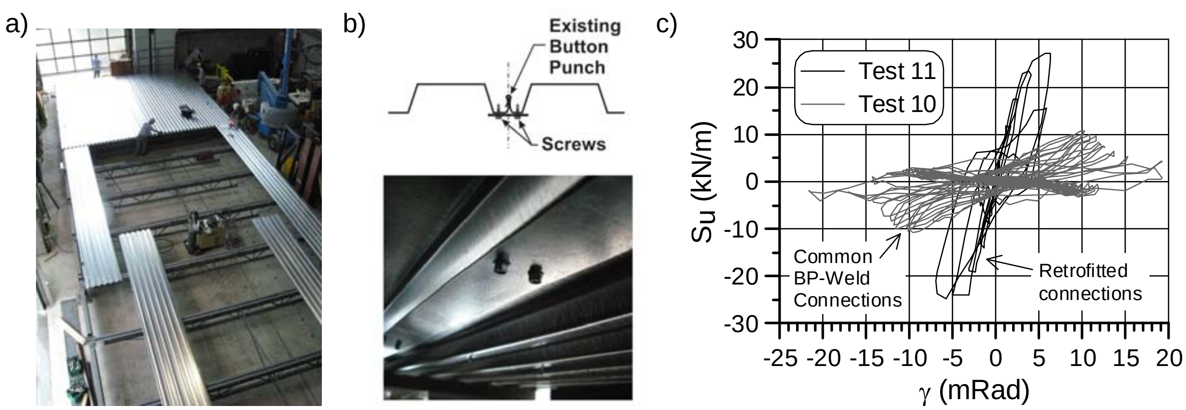


Fig. 12 – Diaphragm dynamic testing : a) Installation of the 7 m x 21 m test specimen; b) Screwed retrofit of the sidelap connections; and c) measured hysteretic responses for the original (Test 10) and retrofitted (Test 11) diaphragms (Massarelli, 2010).

5. Semi-Rigid Constructions

In the 1960's, Type 3 (semi-rigid) construction was permitted to be used in the CSA S16 Standard (CSA 1961, 1965). In this system, which is also referred to as Type 2 (simple) construction with wind connections, lateral resistance is achieved by means of semi-rigid beam-to-column connections designed to develop a plastic moment capacity greater than the moments induced by lateral loading. The beam-to-column joints are built with angles or plates connecting the beam top and bottom flanges to the column (Fig. 13a). The connections are not designed to resist the moments induced by gravity loading and the beams are therefore designed as simply supported members assuming that the connections have sufficient rotation ductility to accommodate the gravity induced end rotations. This simple and popular design approach was allowed in CSA S16 until the 1989 edition. In 1989 and subsequent S16 standards, lateral resistance and stability of simple constructions must be provided by a lateral bracing system.

In view of the lower seismic loads specified prior to 1990 (Fig. 5), existing frames designed using semi-rigid connections in seismic active areas most likely suffer from lack of lateral resistance against earthquake effects. As part of the CSRN project, the seismic performance and retrofit of a prototype steel frame of Type 3 construction designed as per the 1965 code was studied. This prototype structure corresponds to an existing 9-storey hospital building that had been designed and constructed in the 1967-68 period. Detail of the study can be found in Kyriakopoulos and Christopoulos (2013) and Kyriakopoulos (2012). A site visit showed that all frame members were W sections and all beam-column joints had moment capacity in both orthogonal directions provided by steel plates welded to the beam top and bottom flanges. The girders were connected to the column flanges and the secondary beams were connected to column continuity plates. During the site visit, it was also noted that the top connecting plates were locally bent to accommodate geometric imperfections and actual weld sizes in the connections.

Seismic performance of the structure was performed for using linear and nonlinear analysis procedures for two hypothetical locations in eastern and western Canada. The structure was found adequate to resist wind and gravity loading at the assumed sites but response spectrum analysis showed that it did not have sufficient lateral strength to resist seismic design loads as specified in the 2005 NBCC. Linear analysis also showed a tendency for soft-storey response at the structure first level as well as excessive storey drifts (more than 1%) for a post-disaster building. Nonlinear static and dynamic analysis confirmed those findings with excessive drift demand and soft-storey response near the structure base (Fig. 13a). Seismic collapse was not observed under the design level earthquake. Limited incremental dynamic analysis indicated that collapse would occur under ground motions amplified by 1.5 in Vancouver. Stable response was still predicted for the Montreal site at a scaling factor of 3.0.

Due to the lack of cyclic test data on Type 3 construction, the hysteretic beam-to-column joint model in nonlinear analysis was based on data obtained from tests on rigid connections designed and fabricated in accordance with recent codes. Full-scale quasi-static cyclic testing was therefore performed on realistic semi rigid beam-to-column connections. The specimens were fabricated to reflect original construction drawings and as-built conditions. Geometric defects observed during the site visit were also reproduced so that the top connecting plates were initially misaligned as was the case in the actual reference structure. Two tests were performed to investigate the behaviour for the columns bent about their strong and weak axes and, thereby, evaluate the seismic performance along both building directions. As shown in Fig. 13b, the test setup included one beam and two columns. Hence, two connections were tested simultaneously. In all tests, yielding of the connecting plates could be reached in tension and compression except for the top connecting plates which buckled in compression because the longitudinal fillet welds connecting these plates to the beam flanges were interrupted at some distance from the column face, which created unsupported length conditions for the plate. The initial misalignment of the plates and an actual yield stress much higher than indicated on mill test certificates, as obtained from coupon tests performed after the frame tests, probably also contributed to the observed buckling response. For the specimen with strong axis column bending, buckling of the top plates was first observed at a storey drift of 2% (Fig. 13c) and failure eventually took place in the welds connecting these plates to the columns at a drift of 2.6% (Fig. 13d). In the column weak axis test, top plate buckling started at a drift of 1.5% and failure occurred in the fillet welds connecting these plates to the column continuity plates at a drift of 2%. A typical connection hysteresis is presented in Fig. 13e. In all cases, stable ductile response was observed until failure of the connecting top plate welds. Beyond this point, significant strength degradation was measured.

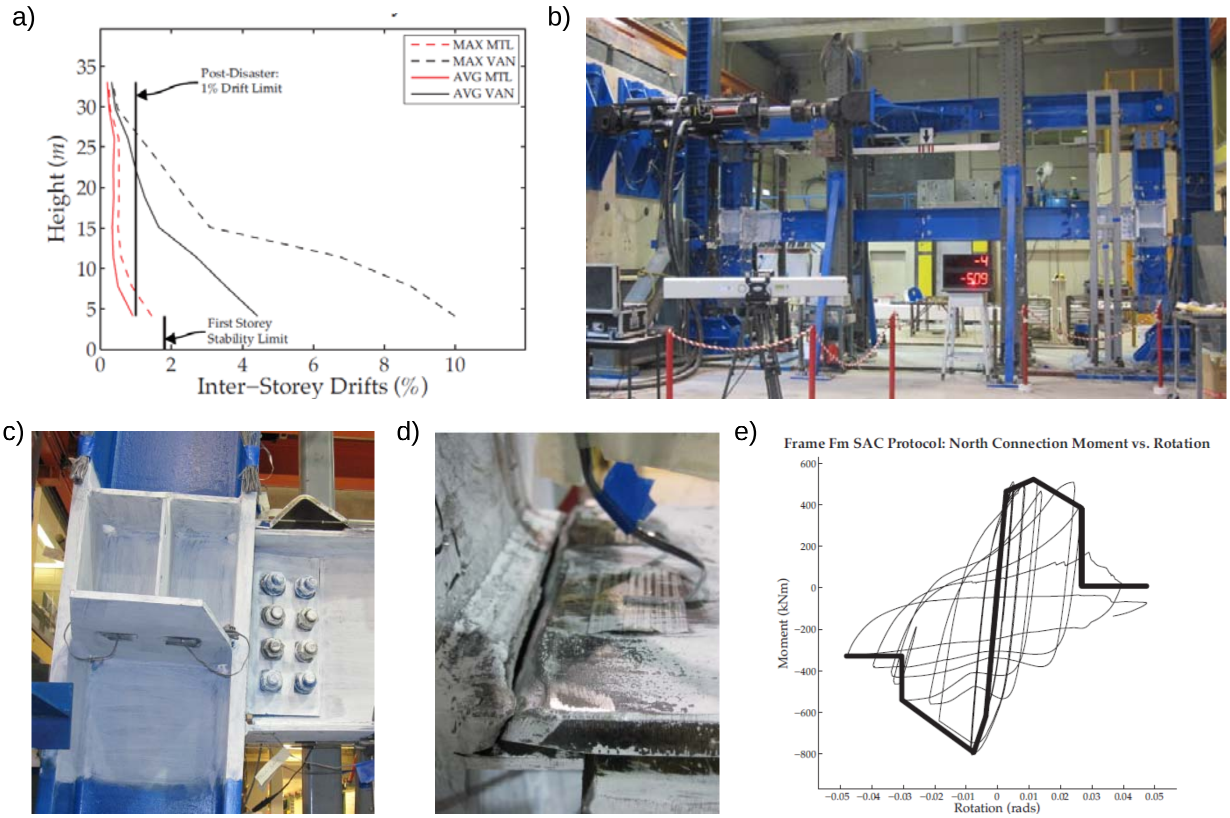


Fig. 13 – Testing of beam-to-column connections of a Type 3 (semi-rigid) construction: a) Seismic drift demand from nonlinear dynamic analysis; b) Test setup; c) Buckling of top flange plate; d) Failure of top flange plate weld; and e) Typical connection hysteresis (Kyriakopoulos, 2012).

For this post-disaster structure, one of the main goals for the retrofit was to reduce storey drifts so that the building could maintain its functionality after a strong earthquake and collapse due to global instability could be prevented in case of an extreme event. Higher performance objectives for higher seismic hazard levels were therefore established. For instance, peak storey drifts were to be limited to 1% under 1.5 times the 2% in 50 years earthquake level. The drift limits were also established to avoid connection rotations exceeding the ductile range. Acceptance criteria were set for residual drifts and peak floor accelerations. The objectives were to be met while limiting the impact on floor accelerations and base shear forces. The chosen retrofit solution consisted of adding supplemental damping to the structure. Both hysteretic and linear viscous dampers were examined. For the Vancouver site, the column base connections were also modified from pinned to fixed to reduce drifts in the first storey. The retrofit design was performed using performance spectra developed by Guo and Christopoulos (2013a&b) as part of the CSRN research effort. With this graphical tool, the design engineer can evaluate the effect of added damping and stiffness on all structural response parameters of interest for the different seismic hazard levels considered. The seismic performance of the retrofitted structures was then verified using nonlinear assessment procedure.

6. Conclusions

The research carried out under the Canadian Seismic Research Network (CSRN) has permitted to develop and disseminate a large amount of technical information that can be used for the seismic assessment and retrofit of steel structures in Canada. Extensive large-scale test programs have been conducted to generate reliable test data on the hysteretic behaviour of members and connection details representative of the Canadian steel industry practice. The studies focused on steel braced frames and frames with semi-rigid beam-to-column connections. Different procedures have been applied to assess the seismic performance of a number of deficient structures. These procedures were also used for the design of retrofit solutions. A novel performance spectra based design technique was used to develop a retrofit strategy with supplemental damping for a frame with semi-rigid connections. Several test programs were also conducted to verify the performance of promising retrofit techniques for deficient steel structures.

The studies showed the key importance of dependable and representative test data in seismic assessment. In particular, the experimental research showed that the response of structural components and connections can be sensitive to material properties and fabrication details. Seismic performance assessment was also found to be sensitive to properties assumed in the analysis of the structure. Data collection therefore also represents a critical phase in a seismic assessment project and caution should be exercised when using or extrapolating from test data generated for similar but not identical construction details. The use of modification factors and/or bounding analysis techniques should be considered to cope with unavoidable uncertainties and the variability in the properties and capacities of components. The studies performed clearly showed the need for additional experimental work to improve our current database on deformation capacities and hysteretic responses for key structural components. The research showed that seismic performance assessment using nonlinear analysis provides more realistic seismic demand estimates when compared to linear procedures. In many instances, it revealed responses that could not be predicted from linear techniques. Nonlinear analysis would therefore lead to more appropriate and targeted definition of the required retrofit measures. For the cases studied, it generally resulted in reduced upgrade requirements compared to linear assessment methods.

Additional guidance is needed on minimum performance objectives and hazard levels that should be considered for the seismic evaluation and retrofit of structures in Canada.

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