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Review of Nonlinear Modelling Parameters and Acceptance Criteria in ASCE 41 for Seismic Evaluation and Upgrading of Steel Structures in Canada

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Abstract: The seismic resilience of structures can be quantified through rigorous seismic assessment. Because no detailed guideline is available for nonlinear modelling parameters and the corresponding acceptance criteria for the seismic evaluation and upgrading of steel structures in Canada, the structural commentary of the National Building Code of Canada (NRC, 2015a) suggests using ASCE 41, which has been adopted as a standard for seismic evaluation and retrofit of buildings in the United States (U.S.). However, because the steel design standards in Canada and the U.S. are different, their applicability to steel structures in Canada needs to be investigated. To this end, this paper critically reviews the nonlinear modelling parameters and acceptance criteria in ASCE 41-17 (2017), then recommends whether these values need to be revised or are adopted as is for the seismic evaluation and upgrading of steel structures in Canada. A numerical example of a steel moment-resisting frame is presented to demonstrate the recommended parameters being used for seismic evaluation.

Keywords: Seismic evaluation and upgrading; structural performance levels; Steel structures; Nonlinear modelling; Acceptance criteria

1. Introduction

The seismic evaluation of existing buildings is paramount in earthquake engineering to improve the seismic resilience of a community. Various standards and guidelines have been developed to help engineers efficiently evaluate the seismic performance of existing buildings. For example, seismic

assessment standards and guidelines have been introduced in several countries such as Canada, the United States (U.S.), New Zealand, and Japan (ASCE, 2017; JBDPA, 2001; NRC, 1993; NRC, 2015; NZSEE, 2017). This paper briefly summarizes the seismic assessment guidelines in Canada and the U.S. Further details of the guidelines in other countries can be found in Fathi-Fazl et al. (2020a).

In the U.S., the state of the practice for seismic assessment of existing buildings is generally informed by NIST *Standards of Seismic Safety for Existing Federally-Owned and Leased Buildings* (NIST, 2011), FEMA P-154 *Rapid Visual Screening of Buildings for Potential Seismic Hazards: A Handbook* (FEMA, 2015), and ASCE 41 *Seismic Evaluation and Retrofit of Existing Buildings* (ASCE, 2017). The NIST standards are intended to be a preliminary screening manual for further evaluation, while FEMA P-154 quantitatively evaluates the conditional probability of structural collapse based on the available building information and a data collection compiled by trained screeners. ASCE 41 is a national standard consisting of three-tiered evaluation procedures. Among the three tiers, Tier 3, systematic evaluation procedure, facilitates detailed seismic evaluation and retrofit of buildings by providing analysis methods, modelling parameters, and acceptance criteria for various structural performance levels.

In a similar manner, the National Research Council Canada (NRC) developed a multi-criteria and multi-level seismic risk management framework for existing buildings in Canada (Fathi-Fazl et al., 2020a), including Level 1 – Preliminary Seismic Risk Screening Tool (PST) (Fathi-Fazl et al., 2018; 2020b; 2020c), Level 2 – Semi-Quantitative Seismic Risk Screening Tool (SQST) (Fathi-Fazl et al., 2020d; 2020e; 2022a), and Level 3 – Seismic Evaluation and Upgrading Guidelines (SEG) (Fathi-Fazl et al., 2022b). Level 1 – PST aims to quickly identify and exempt existing buildings with acceptable seismic risk from further seismic assessment (i.e., Level 2 – SQST or Level 3 – SEG) based on several seismic risk acceptance criteria. Level 2 – SQST intends to follow Level 1 – PST to inexpensively identify and exempt existing buildings with acceptable seismic risk from Level 3 – SEG and prioritize existing buildings with potentially unacceptable seismic risks for Level 3 – SEG. Level 3 – SEG intends to evaluate the potentially hazardous buildings identified in either Level 1 – PST or Level 2 – SQST.

As there are no Canadian-specific guidelines/standards that provide nonlinear modelling parameters and acceptance criteria for the seismic evaluation of structural systems, the commentary L of NBC 2015 (NRC, 2015b) states that ASCE 41 may be used to assess the seismic performance of the structural systems in Canada, provided the ground motions used are as per the NBC 2015 for the seismic hazard levels suggested in the commentary and the intent of the NBC 2015 is satisfied (NRC, 2015a). The nonlinear modelling parameters and acceptance criteria for steel structures in ASCE 41 have been developed based on a large volume of experimental data from various research projects and experts' engineering judgement. However, the application of ASCE 41 for seismic performance assessment of steel structures in Canada needs to be thoroughly reviewed because Canada and the U.S have different design standards and the adoption of ASCE 41 as is may lead to inconsistencies between the guidelines for seismic performance assessment and the seismic design requirements in Canada.

To provide justifications on whether the nonlinear modelling parameters and acceptance criteria in the latest edition of ASCE 41, i.e., ASCE 41-17 (2017), are applicable to steel structures in Canada, it is important to understand how these modelling parameters and acceptance criteria were proposed and applied to the seismic performance assessment. The justifications will help not only Canadian researchers further elaborate on proposing nonlinear modelling parameters and acceptance criteria for steel structures in Canada, but also practicing engineers better understand the steel design standards and assessment procedures of steel structures in Canada. This paper mainly consists of six sections. Section 2 starts with a comprehensive review of the nonlinear modelling parameters and acceptance criteria in ASCE 41-17 (2017). ASCE 41-17 (2017) refers to the design standards for steel structures to determine the nonlinear modelling parameters of steel structural elements, especially the elastic branch of the hysteretic behaviour. Because Canada and the U.S. adopt different steel design standards, Section 3 thoroughly reviews the design standards of the two countries such that the equations used for design and the equations used for the seismic assessment can be consistent upon the adoption of the proposed changes in this paper. After the detailed reviews of the seismic

83 design standards of steel structures in Canada and the U.S. and ASCE 41-17 (2017), Section 4
84 recommends the nonlinear modelling parameters and acceptance criteria for steel structures in
85 Canada. To effectively present the recommendation, this study first classifies the nonlinear modelling
86 parameters and the acceptance criteria in ASCE 41-17 (2017) into five groups depending on the
87 relationship between the design standards of the two countries and their development. Then, in
88 accordance with the attribute of each group, the modelling parameters and acceptance criteria are
89 either adopted as is or replaced by the design equations in the steel design standard of Canada for
90 seismic evaluation of steel structures in Canada. To demonstrate the application of the
91 recommendations to steel structures in Canada, a numerical model of a moment-resisting frame
92 designed based on the NBC 1980 (NRC, 1980) is developed in Section 5. The numerical example is to
93 help practicing engineers and researchers apply the recommendation by following the presented
94 procedures. Finally, a summary and conclusions are presented in Section 6.

95 *2. Nonlinear Modelling Parameters and Acceptance Criteria in ASCE 41-17*

96 In ASCE 41-17 (2017), the modelling parameters and acceptance criteria for the seismic evaluation of
97 structural systems depend on the type of analysis method. To illustrate the modelling parameters
98 and acceptance criteria, this section briefly summarizes the nonlinear analysis methods, which is
99 followed by the definition of acceptance criteria based on Section 7 of ASCE 41-17 (2017). Next, a
100 high-level summary of the modelling parameters in ASCE 41-17 (2017) is provided.

101 2.1. Nonlinear analysis methods

102 In the nonlinear static and nonlinear dynamic analysis methods in ASCE 41-17 (2017), a numerical
103 model of a structure is developed considering the nonlinear force-deformation characteristics of
104 individual structural elements. The nonlinear static procedure (NSP) obtains the global relationship
105 between the base shear force and the roof displacement by applying monotonically increasing
106 equivalent lateral loads to the numerical model. The relationship between the base shear force and
107 the roof displacement is referred to as a pushover curve. Among various NSPs, the coefficient method

is used in ASCE 41-17 (2017) to determine the structural responses for a given seismic event based on the pushover curve. ASCE 41-17 (2017) also suggests the Capacity Spectrum Method in FEMA 440 (2005). On the other hand, in the nonlinear dynamic procedure, a structural system's maximum responses are obtained by performing a nonlinear time history analysis of the numerical model subjected to ground acceleration time histories. Details on the analysis procedures are presented in Section 7.3.1.2 of ASCE 41-17 (2017).

2.2. Acceptance criteria

Acceptance criteria are used to determine a structure's probable performance or its conformance to design/evaluation requirements. ASCE 41-17 (2017) defines the acceptance criteria for three target structural performance levels: Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP), each of which represents a progressively more severe state of damage. Although ASCE 41-17 (2017) defines the performance levels based on global responses of structures considering the anticipated impact of the structural damage and post-earthquake occupancy, the evaluation of the building's seismic performance is conducted at the component action level such as internal moment, shear, torque, axial force, deformation, or rotation.

The acceptance criteria in ASCE 41-17 (2017) are defined based on the following two conditions. The first condition is the ability of a given component's action, such as axial force or shear, to undergo inelastic response, i.e., force- or deformation-controlled, whereas the second condition is the criticality of the element to overall structural performance, i.e., primary or secondary components. When a component's action has measurable ductility and is able to maintain load under inelastic deformation, it is considered deformation-controlled; otherwise, it is referred to as force-controlled. Components that are required to resist seismic forces and accommodate deformations for the structure to achieve the selected performance level are classified as primary, while those that accommodate seismic deformations and are not required to resist seismic forces for the structure to achieve the selected performance level are classified as secondary.

133 *2.2.1. Acceptance criteria for nonlinear procedures*

134 For deformation-controlled actions, primary and secondary components need to have expected
135 deformation capacities corresponding to a structural performance level not less than the maximum
136 deformation demands that each structural element experiences. The expected deformation capacities
137 of primary and secondary components vary along with the selected structural performance level,
138 which are tabulated in Section 9 of ASCE 41-17 (2017). On the other hand, regarding force-controlled
139 actions, primary and secondary components should have lower-bound strengths not less than the
140 force demands. If the force-controlled components' force-deformation relationship is not explicitly
141 modelled, the following equation should be satisfied:

$$\gamma\chi(Q_{UF} - Q_G) + Q_G \leq Q_{CL} \quad (1)$$

142 where γ is the load factor coefficient, χ is the performance level coefficient (taken as 1.0 for CP or
143 1.3 for LS and IO), Q_{UF} represents the force demand caused by gravity and seismic loads, Q_G is the
144 demand from gravity load, and Q_{CL} is the lower-bound strength of a force-controlled action of an
145 element at the deformation level under consideration. Note that $\gamma\chi$ needs not to exceed a value of
146 1.5. An overview of the seismic performance evaluation is presented in Figure 1.

147 *2.2.2. Determination of acceptance criteria*

148 The acceptance criteria for each performance level are defined based on the idealized relationship
149 between the force and deformation of each element as shown in Figure 2, where P and S represent
150 primary and secondary components, respectively. The backbone curve is usually determined based
151 on the experimentally obtained cyclic responses from various studies and experts' opinions. The
152 specified deformation is directly used as the acceptance criteria when nonlinear procedures are used
153 for seismic performance assessment.

154 2.3. Nonlinear modelling parameters

155 For the nonlinear dynamic procedure, the hysteretic behaviour of each element needs to be
156 numerically modelled. ASCE 41-17 (2017) suggests that the hysteretic behaviour of the components

157 is determined experimentally or by other procedures approved by the Authority Having Jurisdiction.
158 The characteristics of the hysteretic loops, including cyclic stiffness degradation in unloading and
159 reloading, cyclic strength degradation, and in-cycle strength degradation, are realistically
160 represented in the component modelling.

161 For the NSP, the envelope of cyclic hysteretic behaviour is generalized as a multi-linear model
162 as shown in Figure 3. In Section 9 of ASCE 41-17 (2017), nonlinear modelling parameters and the
163 corresponding acceptance criteria are provided for each steel structural element based on the
164 generalized force-deformation relationship in Figure 3. The followings steel elements' modelling
165 parameters are available in ASCE 41-17 (2017): (1) beam-column elements, (2) panel zones, (3) beam
166 and column connections, (4) buckling-restrained braces, (5) link beams, (6) steel-plate shear wall, (7)
167 structural steel frame with infill, (8) diaphragms, (9) cast and wrought iron, and (10) cold-formed
168 structural systems.

169 The modelling parameters up to the yielding of each steel structural element (stiffness, yield
170 strength, and yield deformation) are defined based on the design standards of steel structures or
171 based on the first principles. On the other hand, the post-yield force-deformation relationship (points
172 C, D, and E in Figure 3) is derived based on the experimentally obtained cyclic responses from various
173 studies. A brief summary of the modelling parameters of the elastic part of steel structural elements
174 (segment from A to B in Figure 3) are summarized in Table 1, especially the equations of strength and
175 yield deformation that are explicitly presented in ASCE 41-17 (2017). Note that this paper adopts
176 Canadian notations especially for those cross over ASCE 41-17 (2017) and CSA S16 because this paper
177 is written for the practical engineers and researchers who deal with the steel structures in Canada.
178 From the literature review, some of the equations in ASCE 41-17 (2017) are adopted from the design
179 requirements in AISC 360 and AISC 341. More details of parameter definitions, application limits,
180 and commentaries are found in ASCE 41-17 (2017).

181 In ASCE 41-17 (2017), based on the generalized force deformation relationship, the acceptance
182 criteria for nonlinear analysis procedures are defined in terms of the plastic deformation, i.e., a, b,

183 and c in Figure 3. Since the post-yield force-deformation relationship, i.e., C, D, and E in Figure 3, is
184 derived based on the experimentally obtained cyclic responses from various studies which are not
185 affected by design standards, it is logical to directly adopt the acceptance criteria in ASCE 41-17 (2017)
186 to steel structures in Canada. However, ASCE 41-17 (2017) defines the modelling parameters of the
187 elastic region for some steel structural elements by referring to the design standards of the U.S. Note
188 that even though some equations are presented in ASCE 41-17 (2017) as seen in Table 1, it is found
189 that the formulas, especially those that define the strength of the steel components are very similar
190 to the strength equations in the design standards. Thus, the following section reviews the design
191 standards of the two countries, which forms the basis to evaluate whether the nonlinear modelling
192 parameters and the acceptance criteria of ASCE 41-17 (2017) can be adopted as is, or whether they
193 need to be revised.

194 ***3. Review of Steel Design Standards in Canada and the U.S.***

195 Steel design standards in Canada and the U.S. are listed in Table 2. After a comprehensive review of
196 historic design standards of Canada and the U.S. from the early 1900s to current, remarks are
197 provided in the subsequent section for each steel structural element in Table 1. Since the nonlinear
198 modelling parameters and acceptance criteria in ASCE 41 have evolved from ATC 14 (1987), FEMA
199 178 (1992), FEMA 273 (1997), ASCE 31 (2003), ASCE 41-06 (2006), and ASCE 41-13 (2013), and are
200 determined based on the first principles, cumulative experimental data, and experts' opinions, we
201 decided to perform a detailed review of historical development of steel design standards in Canada
202 relative to those of the U.S., to identify if any specific changes in CSA standard resulting in unique
203 component detailing found in Canadian buildings that causes seismic vulnerabilities (Fazileh et al.,
204 2022). Since ASCE 41-17 (2017) uses the steel design standards (AISC 360 and AISC 341) to calculate
205 the strength of the modelling parameters except that the strength reduction factor is taken as 1.0, the
206 equations in CSA S16 are adopted without a strength reduction factor. Note that while conducting
207 this review, AISC 360-16 (2016) and AISC 341-16 (2016) are the latest editions of the design standards
208 in the U.S., while CSA S16-19 (2019) is the latest standard in Canada. After providing an overview of

historic design standards in Section 3.1, a high-level summary of each structural element is presented in Sections 3.2 through 3.9.

3.1. Historical steel design standards

The first design standards in Canada and the U.S. were developed based on allowable (or working) stress design (ASD or WSD). In ASD, the demands to structural elements are calculated based on working, or service loads, then compared to allowable stress limits that are determined by critical stress such as yielding or buckling stress and a factor of safety. After World War II, research into the ultimate strength of steel structures, i.e., plastic design, began. The first plastic design criteria were adopted in the 1961 edition of both CSA S16 and AISC 360. The application of the plastic design philosophy was limited to some structural elements and the procedure was complicated. Based on the review of historic design standards, it is found that the design standards of the two countries are equivalent to each other until 1970.

A new design philosophy based on reliability theory, referred to as limit states design, was first introduced in CSA S16-M78 (1978). Similarly, the limit states design philosophy, named *Load and Resistance Factor Design* (LRFD) was first adopted in 1986 in AISC 360. Note that, in the latest version of steel design standards, CSA S16 only provides the design requirements developed based on the limit states design, whereas both ASD and LRFD are being used in AISC 360. Some discrepancies have been observed in equations and design requirements of the Canadian and the U.S. steel design standards since the new design philosophy was adopted. However, from a thorough review, it is found that the discrepancies are minor and both standards provide similar strength values for each element compared to experimental results.

Although AISC 360 is intended to cover common design criteria, it is not feasible to deal with many special and unique details when steel structures are subjected to an earthquake. To address this issue, the first edition of seismic design standard *Seismic provisions for structural steel buildings* (AISC 341) was released in 1990. In a similar manner in Canada, the first seismic design requirements are introduced in Clause 27 of the 1994 edition of CSA S16 (CSA S16-94, 1994). Figure 4 presents the

design standards of the two countries listed in a chronological order with the major milestones. Note that there are several other historic editions with minor revisions that are not listed in Figure 4. A more detailed review of the historic design standards can be found in Kwon et al. (2022).

Regarding the cold-formed steel structures, in 2001, the first edition of *North American Specification for the Design of Cold-Formed Steel Structural Members* was developed by a joint effort of the American Iron and Steel Institute (AISI) Committee on Specifications, the Canadian Standards Association (CSA) Technical Committee on Cold-Formed Steel Structural Members (CSA S136-M94, 1994), and Camara Nacional de la Industria del Hierro y del Acero (Camara, 1965) in Mexico (AISI, 2001). It includes the ASD and LRFD methods for the U.S. and Mexico together with the limit states design (LSD) method for Canada. This North American specification has been accredited by the American National Standard Institute (ANSI) to supersede the previous editions of *Specification for the Design of Cold-Formed Steel Structural Members* published by AISI and the previous editions of CSA S136 *Cold Formed Steel Structural Members*. Following the successful use of the 2001 edition of the North American Specification for six years, it was revised and expanded in 2007. The latest edition of CSA S136-16 (2016) is equivalent to that of AISI S310 (2016).

3.2. Beam-column elements

The design of beam-column should consider the combined effects of axial and flexural loads, which in general, forms interaction equations. The interaction equations in CSA S16 and AISC 360 are similar, but the coefficients and consideration of the weak axis bending are different. There were various research efforts that compare the design standards in this regard (Bartlett, 2021; Galambos, 1999; Greene 2021). The studies pointed out that even though the final form of the criteria is not equivalent in many cases, the theoretical and experimental bases for such requirements are similar to each other. In addition, Tremblay (2002) compared the design criteria of the buckling strength of the bracing member provided in CSA S136-M94 (1994) and the 1999 edition of AISC 360 (1999) with the experimental results. It was found that even though the equations of CSA S136-M94 (1994) and AISC

(1999) are different from each other, these two design criteria properly capture the mean trend of the compressive strength.

3.3. Panel zones

AISC 360-16 (2016) provides two different criteria for the panel zone along with the consideration of plastic panel zone deformation in the analysis. The strength of the web panel zone for the limit state of shear yielding V_r is given below. Note that CSA S16 and AISC 360 use different mathematical notations to describe the mechanical properties of structures, but this study, hereafter, adopts the notations in CSA S16 for consistency.

(a) When the effect of panel-zone deformation on frame stability is not considered in the analysis

$$V_r = \begin{cases} 0.60F_{yc}d_c w & \text{for } C_u \le 0.4C_y \\ 0.60F_{yc}d_c w\left(1.4 - \dfrac{C_u}{C_y}\right) & \text{for } C_u > 0.4C_y \end{cases} \quad (2)$$

(b) When frame stability, including plastic panel-zone deformation, is considered in the analysis:

$$V_r = \begin{cases} 0.60F_{yc}d_c w\left(1 + \dfrac{3b_c t_c^2}{d_c d_b w}\right) & \text{for } C_u \le 0.75C_y \\ 0.60F_{yc}d_c w\left(1 + \dfrac{3b_c t_c^2}{d_c d_b w}\right)\left(1.9 - \dfrac{1.2C_u}{C_y}\right) & \text{for } C_u > 0.75C_y \end{cases} \quad (3)$$

where F_{yc} is the specified yield stress of the column web, d_c is the column depth, w is the column web thickness, b_c is the width of column flange, t_c is the thickness of the column flange, d_b is the beam depth, C_y is the axial yield strength of the column, and C_u is the required axial strength. Since the strength of column flange is superimposed when the inelastic deformation is taken into account for the frame stability, Eq. (2) is more conservative than Eq. (3).

On the other hand, CSA S16-19 (2019) only provides a single design equation as follows.

$$V_r = 0.55d_c w F_{yc}\left[1 + \frac{3b_c t_c^2}{d_c d_b w}\right] \quad (4)$$

As seen, the design equation in CSA S16-19 (2019) (Eq. (4)) and the equation in AISC 360-16 (2016) (Eq. (3)) are similar, when $C_u \le 0.75C_y$. In addition, because of the reduction factor $\left(1.9 - \frac{1.2C_u}{C_y}\right)$ when $C_u > 0.75C_y$, the equations in AISC 360 are more conservative than those of CSA S16.

3.4. Connections

The 1994 Northridge earthquake revealed the brittle nature of welded beam-column joints (James, 1998; Mahin, 1997). This catastrophic event led to the development of new beam-column connection designs which behave in a ductile manner. From the literature review (Carter & Duncan, 2013) and thorough investigation of the two design standards, it was found that major revisions have been made to the design details of the connection after the Northridge Earthquake. Before this event, Canada and the U.S. design standards had equivalent design criteria for the connections. Although the design details of the connections between Canada and the U.S. in the latest edition are not identical, their difference is minor (Maiorana & Pellegrino, 2013; Pizzuto, 2019). For example, the block shear failure equations in the latest edition of CSA S16 and AISC 360 are different, but Pizzuto (2019) demonstrated that the calculated values from the two equations are similar to each other and to the experimental results.

3.5. Braces in buckling-restrained braced frame

Since the braces in buckling-restrained braced (BRB) frames are intended to resist seismic loads, design detailing requirements are provided in Clause 27 of CSA S16 in Canada and AISC 341 in the U.S. The design detailing requirements about BRBs were first introduced in CSA S16-09 (2009). In CSA S16-09 (2009), the probable tensile and compressive resistance of the bracing members, including strain hardening and friction effects are taken as follows:

$$T_{ysc} = \omega A_{sc} R_y F_{ysc}$$
$$C_{ysc} = \beta \omega A_{sc} R_y F_{ysc} \quad (5)$$

where ω is the strain hardening adjustment factor, A_{sc} is the cross-sectional area of the yielding segment of the steel core, F_{ysc} is the yield stress of the steel core, R_y is the factor applied to yield stress to estimate probable yields stress, and β is the friction adjustment factor. These requirements are also in the latest version of the CSA S16, i.e., CSA S16-09 (2009). The design criteria of the BRB is first introduced in the 2005 edition of AISC 341 and the equivalent design criteria shown in Eq. (5) are given in Section F4 of AISC 341-16 (2016).

3.6. Link beams

Eccentrically Braced Frame (EBF) is developed by combining the merits of a concentrically braced frame and a moment-resisting frame, which brings high elastic stiffness and ductile inelastic responses to the system (David & Sarif, 2020; Kazemzadeh Azad & Topkaya, 2017; Koboevic & David, 2010). EBF has a *structural fuse* which is the link element that dissipates energy through yielding. The first EBF project in Canada was built in the 1980s and EBF design provisions were first introduced in Appendix D of (CSA S136-M89, 1989).

In Section F3 of AISC 341-16 (2016), details of link beam in EBF are described. The available shear strength of the link is equal to the lesser of V_{pa} or $2M_{pa}/e$, where

$$V_{pa} = \begin{cases} V_p & \frac{C_u}{C_y} \leq 0.15 \\ V_p\sqrt{1-\left(\frac{C_u}{C_y}\right)^2} & \frac{C_u}{C_y} > 0.15 \end{cases} \quad (6)$$

$$M_{pa} = \begin{cases} M_p & \frac{C_u}{C_y} \leq 0.15 \\ 1.18M_p(1-(C_u/C_y)) & \frac{C_u}{C_y} > 0.15 \end{cases} \quad (7)$$

in which V_p is the plastic shear resistance, C_u is the axial force in the link, C_y is the axial yield strength, M_p is the plastic moment resistance, and e is the length of the link. Similarly, Clause 27 of CSA S16 provides the design requirements for the link beam as the lesser of V_{pa} or $2M_{pa}/e$, where

$$V_{pa} = V_p\sqrt{1-\left(\frac{C_u}{C_y}\right)^2} \quad (8)$$

$$M_{pa} = 1.18M_p(1-(C_u/C_y)) \leq M_p \quad (9)$$

From this comparison, one can find that the design criteria for the link beam of the two countries are equivalent to each other, except when the axial load ratio is smaller than 0.15.

3.7. Steel plate shear walls

The concept of steel plate shear walls has been recognized in CSA S16 since 1995. In typical designs and as assumed by the seismic provisions, the webs of steel plate shear walls are unstiffened and

slender (Ericksen & Sabelli, 2008). The webs are therefore capable of resisting large tensile forces, but little or no compression. As lateral loads are imposed on the system, shear stresses develop in the web until the principal compression stresses (oriented at a 45° angle to the shear stress) exceed the compression strength of the plate. At this point, the web plate buckles and forms diagonal fold lines.

In the 1994 edition of CSA S16, the design requirements of the steel plate shear wall are provided in a non-mandatory appendix, Appendix M. The design details of the plate wall were well established in CSA S16-09 (2009). The shear resistance of infill plates is taken as

$$V_r = 0.4F_y wL \sin 2\alpha \quad (10)$$

where F_y is the minimum yield stress, w is the thickness of the web, and L is the clear distance between vertical boundary elements flanges, and α is the angle of web yielding. On the other hand, in the U.S., the design criteria for the special plate shear wall are first included in the 2005 edition of AISC 341. The design shear strength in the latest edition of AISC 341 (2016) is given by

$$V_r = 0.42F_y wL \sin 2\alpha \quad (11)$$

As shown in Eqns. (10) and (11), the design equations of the two standards are very similar.

3.8. Cold-formed structures

As described in Section 3.1, the same design standards are used in Canada and the U.S. for cold-formed steel structures since 2001. Thus, this study reviewed the cold-formed steel design standards released before 2001 in Canada and the U.S. Similar to the design standards for steel structures, it was found that equivalent design procedures were provided in two countries before the 1980s. Some discrepancies have been observed in the design standards released in the 1980s and 1990s, but, similarly to the steel design standards, their impact on the seismic assessment procedure is expected to be minor.

3.9. Diaphragms

Cold-formed steel diaphragms consist of profiled steel sheets or panels to resist in-plane shear forces such as seismic and wind-induced lateral loads. Currently, Canada and the U.S. follow the same

design standard for steel diaphragms, AISI S310-16 (2016) *North American Standard for the Design of Profiled Steel Diaphragm Panels*. The design standard provides an analytical method for determining the strength and stiffness of the steel deck diaphragm and test procedures as an option for determining the limit states. The AISI S310 standard has been adopted as a reference in CSA S136 and AISI S100.

Before using the consolidated specification, the U.S. employed both (1) Tri-Services Manual published by the Department of Army and (2) Diaphragm Design Manual published by Steel Deck Institute (SDI), referred to as the SDI method, whose first edition was published in 1981. The Tri-Services method is based on a series of full-scale experiments from which empirical equations for both strength and stiffness were developed. On the other hand, the SDI method was based on the works of Luttrell (Luttrell, 1981; Luttrell, 1995; Luttrell and Huang, 1981) and tests were conducted at West Virginia University. The method adopts an analytical approach that has been confirmed by experiments and allows the tabulation of strength and stiffness values. Note that the AISI S310 is a consensus standard which has been developed based on both the Tri-Services method and SDI method.

The Canadian Sheet Steel Building Institute (CSSBI) adopted the Tri-Service method as the basis for the first diaphragm design standard in 1972. However, there existed some limitations in the Tri-service method, in that it did not indicate the failure mode of the diaphragm and no explicit factor of safety has been provided although a value 2.5 has typically been assumed. Thus, the Design of Steel Deck Diaphragm (CSSBI, 2006) also adopted the SDI method for the fastener configurations that are not covered by the Tri-Services method. From this fact, one can infer that steel diaphragms in Canada and the U.S. are designed based on the equivalent design criteria.

4. Recommendation of nonlinear modelling parameters and acceptance criteria for seismic evaluation and upgrading of steel structures in Canada

Based on the comprehensive literature reviews of design standards and assessment guidelines, this section recommends nonlinear modelling parameters and acceptance criteria for seismic evaluation

and upgrading of existing steel structures in Canada. This recommendation is proposed based on the following reasonings.

First, the backbone curve of each structural element (e.g., a, b, and c in Figure 3), especially the modelling parameters defining the plastic region, has been developed based on the compiled experimental results and experts' opinions according to commentaries of ASCE 41-17 (2017). Based on the review of historic design standards and available literature, there is no clear evidence showing that components of steel structures in Canada would behave differently from the components of steel structures in the U.S. Even if different design standards were used in the past, the assessment is carried out based on the as-built condition of the structural components. Thus, it is deemed appropriate to directly adopt the nonlinear modelling parameters in ASCE 41-17 (2017) for steel structural systems in Canada. It is, however, noted that there is room for improvement as many parameters and coefficients were based on engineering judgement without sufficient experimental results. Updating parameters of a specific type of structural element, though, deserves systematic research with well-controlled test parameters. The following studies show efforts to update the nonlinear modelling parameters and acceptance criteria using available experimental data and numerical simulations: Deierlein et al. (2017, 2018), Hamburger et al. (2017), Lignos et al. (2019), and PEER/ATC (2010).

Second, since the acceptance criteria for each structural performance level (e.g., IO, LS, and CP) in ASCE 41-17 (2017) are defined based on the backbone curve of each component following the procedure in Section 2.2.3, it is deemed appropriate to adopt the structural performance levels as presented in ASCE 41-71 (2017). Even though there is room for further improvement to refine the structural performance levels in terms of probability of exceedance based on reliability theory, such improvement requires independent research.

Third, as discussed in Section 3, most design equations in CSA S16 and the corresponding equations in AISC 360 are similar. Some strength equations show minor differences, though. Because newly designed structures in Canada, which satisfy the latest edition of CSA S16, should also satisfy

396 the requirements in the seismic evaluation guidelines, it is deemed appropriate to use the strength
397 and stiffness parameters in the Canadian design standards for seismic evaluation of steel structures
398 in Canada, rather than those in AISC 360 or AISC 341. Thus, in order to maintain the consistency
399 between the design and the evaluation, it is recommended that the strength and stiffness equations
400 in CSA S16 replace those in AISC 360 and AISC 341 in defining the elastic segment of the backbone
401 curve of various steel structural elements.

402 Based on the above reasonings, the nonlinear modelling parameters in ASCE 41-17 (2017) are
403 revised in order to apply the seismic evaluation procedures to steel structures in Canada. The
404 nonlinear modelling parameters are classified into five types as shown in Table 3. The three types
405 among five do not require revision of ASCE 41-17 (2017), but the rest require minor revision for their
406 applications to steel structures in Canada to maintain consistency with the design equations for new
407 steel structures and those for evaluation of existing structures.

408 To clearly describe the five types of recommended revisions, a high-level summary of
409 application of Types 1 through 5 to the nonlinear modelling parameters is presented for structural
410 elements listed in Table 1. More detailed recommendations can be found in Kwon et al. (2022).

411 4.1. Beam-column elements

412 A beam is defined as a member when the axial force (compression or tension) in the member does
413 not exceed 10% of the expected compressive strength, while when the axial force (compression or
414 tension) in the member equals to or exceeds 10% is classified as a column. AISC 360-16 (2016) presents
415 the interaction equation as follows

(a) For $\frac{C_f}{C_r} < 0.2$

$$\frac{C_f}{2C_r} + \left(\frac{M_{fx}}{M_{rx}} + \frac{M_{fy}}{M_{ry}}\right) \leq 1.0 \tag{12}$$

(b) For $\frac{C_f}{C_r} \geq 0.2$

$$\frac{C_f}{C_r} + \frac{8}{9}\left(\frac{M_{fx}}{M_{rx}} + \frac{M_{fy}}{M_{ry}}\right) \leq 1.0$$

where C_f is the axial strength demand, C_r is the axial strength capacity, M_f is the flexural strength demand, M_r is the flexural strength capacity, and x and y in the subscript are respectively strong and weak axis. By rewriting Eq. (12), it is possible to obtain the strength of the beam-column elements in Table 1. From this relationship, it is found that the interaction equations in ASCE 41-17 (2017) are derived from the design standard, which indicates that it belongs to Type 3 recommendation. Therefore, it is recommended to adopt the corresponding equations in CSA S16 to evaluate the seismic performance of steel structures in Canada. The interaction equation of Class 1 and Class 2 sections of I-shaped members in CSA S16-19 (2019) is given as follows:

$$\frac{C_f}{C_r} + \left(\frac{0.85U_{1x}M_{fx}}{M_{rx}} + \frac{\beta U_{1y}M_{fy}}{M_{ry}}\right) \leq 1.0 \tag{13}$$

where U_1 is the factor to account for moment gradient and for second-order effects of axial force acting on the deformed member, and β is the coefficient for weak axis bending in beam-columns. By rewriting Eq. (13), the following strength equation is recommended for steel structures in Canada.

(a) For strong axis

$$M_{CEx} = M_{pcex} = \frac{M_{pex}}{0.85U_{1x}}\left(1 - \frac{|C|}{C_{ye}}\right) \leq M_{pex}$$

(b) For weak axis

$$M_{CEy} = M_{pcey} = \frac{M_{pey}}{\beta U_{1y}}\left(1 - \frac{|C|}{C_{ye}}\right) \leq M_{pey} \tag{14}$$

Note that similar manner can be employed to other types of sections in CSA S16 to define the strength of the element.

Regarding the yield deformation, as described at the beginning of this section, the equations of yield chord rotation are provided neither in AISC 360 nor in CSA S16 but are present in ASCE 41-17 (2017). Since the yield chord rotation is derived based on the first principles, it is suggested to adopt the equations as is following Type 1 recommendation.

4.2. Panel zones

By comparing the equations in Table 1 and Eqns. (2) and (3), it is found that ASCE 41-17 (2017) adopts Eq. (2) to determine the strength of panel zones. As discussed in Section 3.3, Eq. (2) is more conservative than Eq. (3), but CSA S16 only provides the shear strength that corresponds to Eq. (3) as presented in Eq. (4). Therefore, it is recommended to adopt the shear strength criteria shown in ASCE 41-17 (2017) which is relatively conservative. This recommendation is in accordance with Type 4 recommendation.

4.3. Connections

As discussed in Section 3.4, although the difference between the design details of the connection elements between the standards of Canada and the U.S. are minor, they are not identical. For instance, in AISC 360-16 (2016), the strength for the limit state of block shear rupture along a shear failure path and a perpendicular tension failure path is determined as

$$R_n = 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \leq 0.60F_yA_{gv} + U_{bs}F_uA_{nt} \quad (15)$$

where F_u is the minimum tensile strength, A_{nv} is the net area subject to shear, U_{bs} is the reduction coefficient, A_{nt} is the net area subject to tension, F_y is the minimum yield strength, A_{gv} is the gross area subject to shear. Where the tensile stress is uniform, $U_{bs} = 1$; where the tensile stress is non-uniform, $U_{bs} = 0.5$. On the other hand, the corresponding equation in CSA S16-19 (2019) is given as

$$T_r = [U_tA_{nt}F_u + 0.6A_{gv}\frac{(F_y + F_u)}{2}] \quad (16)$$

where U_t is a factor to account for efficiency of the tensile area listed as a table in CSA S16-19 (2019). Therefore, it is recommended to determine the stiffness and strength of the beam column connections as specified in CSA S16 as a replacement of AISC 360 in accordance with Type 5 recommendation.

In addition to the aforementioned recommendation, ASCE 41-17 (2017) provides equations to define the strength of some partially restrained beam-column connections which are presented neither in AISC 360 nor in CSA S16. According to the commentary of ASCE 41-17 (2017), such equations are derived based on experimental results, especially from the SAC documents (FEMA 350,

2000; FEMA 351, 2000; FEMA 355D, 2000; FEMA 355F, 2000). Therefore, it is recommended to adopt such values as is by following Type 1 recommendation.

4.4. Braces in buckling-restrained braced frame

From the discussion in Section 3.5 and Eq. (5), since identical design criteria are provided in CSA S16 and AISC 341 for braces in buckling-restrained braced frames, it is recommended to adopt ASCE 41-17 (2017) as is by following Type 2 recommendation.

4.5. Link beams

As seen in Table 1 and Eqns. (6) and (7), one can infer that the equations in ASCE 41-17 (2017) adopt the design criteria of link beams in AISC 341. Therefore, it is recommended to adopt the corresponding part in the CSA S16-19 (2019), i.e., Eqns. (8) and (9), to estimate the strength of link beams in Canadian steel structures. This recommendation is in accordance with Type 3 recommendation. In addition, because the shear yield deformation is provided neither in AISC 360 nor in CSA S16, but is proposed in ASCE 41-17 (2017), it is recommended to adopt the equation as is in accordance with Type 1 recommendation.

4.6. Steel plate shear walls

As discussed in Section 3.7, the difference between the design details of the steel plate shear walls between the standards of Canada and the U.S. are minor. It is recommended to estimate the strength and stiffness of the steel plate shear walls as specified in CSA S16 as a replacement of AISC 360 and AISC 341. In other words, Eq. (11) is substituted by Eq. (10). The recommendation corresponds to Type 5 recommendation. Regarding the yield deformation and stiffness of steel plate shear walls, it is recommended to adopt the equations in ASCE 41-17 (2017) as is in accordance with Type 1 recommendation

4.7. Structural steel frame with infill

Structural steel frames with partial or complete infills of reinforced concrete or reinforced or unreinforced masonry need to be evaluated considering the combined stiffness of the steel frame and infill material. It is recommended that the engineering properties and permissible performance parameters for the steel frame components comply with the recommendation specified in Sections 4.1 to 4.6 of this paper, while those for the infill walls should comply with the requirements in Chapter 10 of ASCE 41-17 (2017) for concrete and Chapter 11 of ASCE 41-17 (2017) for masonry. Note that justifications on whether the nonlinear modelling parameters and acceptance criteria of concrete and masonry components of the seismic-force-resisting systems in Chapters 10 and 11 of ASCE 41-17 (2017) are applicable to concrete and masonry structures in Canada are not within the scope of this study.

4.8. Diaphragms

Since Canada and the U.S. share the same design standards for the diaphragms, i.e., AISI S310-16 (2016), it is recommended to directly adopt the nonlinear modelling parameters and acceptance criteria of diaphragms in ASCE 41-17 (2017) for the diaphragms of steel structures in Canada, except that ASCE 41-17 (2017) references AISC 360 to calculate the strength of components of the diaphragms. For example, ASCE 41-17 (2017) calculates the strength of steel-headed stud anchors for steel deck diaphragms with reinforced concrete structural topping by referring to Chapter I of AISC 360, *Design of Composite Members*. In such case, it is recommended to substitute Clauses 17 and 18 in CSA S16 which are respectively *Composite Beams, Trusses, and Joints*, and *Composite Columns* for Chapter I of AISC 360.

4.9. Cast and wrough iron

Since no specific design standards are used to provide the nonlinear modelling parameters and acceptance criteria for cast and wrought iron, it is recommended to adopt the nonlinear modelling parameters and acceptance criteria specified in ASCE 41-17 (2017) as is for steel structures in Canada.

4.10. Cold-formed structures

When computing the stiffness and strength of cold-formed structural systems, ASCE 41-17 (2017) adopt various procedures presented in AISI S400 *North American Standard for Seismic Design of Cold-Formed Steel Structural Systems*. Since Canada and the U.S. share the same design standard for cold-formed structures, it is recommended to adopt the evaluation procedures in ASCE 41-17 (2017) as is for cold-formed structures in Canada.

5. Application example to a steel structure in Canada

The numerical study is conducted to demonstrate the procedure to estimate the nonlinear modelling parameters and acceptance criteria of a steel structure in Canada based on the recommendation provided in Section 4. The aim of this numerical study is not to validate the proposed recommendations by comparing them to the ones in ASCE 41-17 (2017) but to demonstrate the modelling of a nonlinear steel structural system. Using the defined nonlinear modelling parameters and acceptance criteria, it is possible to evaluate the seismic performance of steel structural systems. This section only provides the modelling details and the overall evaluation procedure is provided in Appendix A.

5.1. Reference structure

A representative steel moment-resisting frame (MRF) located in the city of Montreal is selected for this application example. The steel MRF was designed by Gómez et al. (2015) based on the NBC 1980 (NRC, 1980). The plan view of the reference structure is presented in Figure 5(a) which consists of a rectangular layout of 3 by 4 bays. The structure is symmetrical in both directions. The lateral load resisting system in the EW-direction consists of steel MRFs, while in the NS-direction it consists of braced frames. This article only analyzes the EW-direction of the structure, and the corresponding elevation view is illustrated in Figure 5(b). The numbers in the squares in Figure 5(b) represent the node numbers which are assigned for explanation purposes.

According to Gómez et al., (2015), in 1980, steel MRFs were designed with a storey drift limit of 0.20%, but it was not mandatory. Thus, the authors of Gómez et al. (2015) designed two different

structural systems with and without meeting the storey drift limit. This study adopts a design that does not meet the requirement. In other words, the relatively unconservative structural model is selected. The selected wide flange sections are listed in Table 4. The design details including mass, gravity loads, and material and sectional properties can be found in Gómez et al. (2015).

5.2. Nonlinear modelling parameters and acceptance criteria

The numerical model consists of beam, column, and panel zone elements. Using the nonlinear modelling parameter and acceptance criteria tables provided in Section 9 of ASCE 41-17 (2017) and the recommendation in Section 4, the nonlinear backbone curve for each structural element is able to be defined. To effectively present the results, the nonlinear modelling parameters and acceptance criteria of the first storey in the frame structure are presented in this subsection and those of other storeys can be found in Appendix A.

5.2.1. Beam

Following the equation in Table 1 and the recommendation in Section 4.1 with sectional properties shown in Table 4, the yield strength and yield deformation of the beam element can be calculated as shown in Table 5. In calculating the member strength, the expected yield stress of 379 ($= 1.1 \cdot 345$) MPa is employed, where 345 MPa is the nominal yield strength, and 1.1 is obtained from Clause 27.1.7 of CSA S16-19 (2019). Moreover, the moment of inertia of the beam elements is amplified by 1.4 to consider the effect of composite action on beams of the moment frame following the recommendation by Elkady & Lignos (2014).

Since the connection details govern the plastic behaviour of the beam element, modelling parameters of the connection are introduced. According to Gómez et al. (2015), the connection of the structure is designed such that plates are welded to the bottom and top beam flanges and to the column flanges. Although the modelling parameters of the connection are calibrated using experimental datasets in Gómez et al. (2015), this study uses the provided modelling parameters in ASCE 41-17 (2017). The nonlinear modelling parameters and acceptance criteria for each structural

554 performance level of welded top and bottom haunches can be found in Table 9.7-2 of ASCE 41-17
555 (2017) and presented in Table 6. Note that the provided nonlinear modelling parameters do not vary
556 along with the sectional properties.

557 *5.2.2. Column*

558 Since it is recommended to adopt Eq. (14) to calculate the yield strength of column elements for steel
559 structures in Canada, U_{1x} needs to be first estimated. To this end, the axial force value of the
560 structure is obtained by applying the gravity loads to the numerical model. The gravity loads consist
561 of the dead, live, and snow loads of which details are provided in Appendix A. By performing a linear
562 analysis, the ratio of axial strength demand to capacity for each column is obtained as shown in Table
563 7. Using the equation provided in Clause 13.8.4 of CSA S16-19 (2019) and Eq. (14), the yield strength
564 and deformation of the column are calculated as presented in Table 7.

565 The nonlinear modelling parameters of the columns are defined considering the flange and web
566 slenderness following the procedure provided in Table 9.7-1 of ASCE 41-17 (2017). Since each column
567 member has different axial strength ratios and slenderness values, an interpolation has been carried
568 out to define the nonlinear modelling parameters. Once a, b, and c are determined, the acceptance
569 criteria are calculated using equations given in Table 9.7-1. The results are shown in Table 8.

570 *5.2.3. Panel zone*

571 As described in Section 4.2, it is recommended to adopt the equation provided in ASCE 41-17 (2017)
572 to determine the shear strength of panel zones for steel structures in Canada. Therefore, using the
573 sectional properties of adjacent columns and beams, the nonlinear modelling parameters and
574 acceptance criteria of panel zones can be calculated as presented in Table 9.

575 5.3. Analysis results

576 Using the nonlinear modelling parameters, it is possible to develop a numerical model. Among many
577 NSPs, this study employs the coefficient method to estimate the seismic demand of each structural

component for a given seismic hazard. By comparing the seismic demand and various structural performance levels, the seismic performance of the model is evaluated. It was found that the panel zones are the most vulnerable elements in the MRF structure, but all the demands are below the LS capacity. Detailed procedures and results are provided in Appendix A.

6. Conclusions

This study reviewed the nonlinear modelling parameters and acceptance criteria in ASCE 41-17 (2017) for steel structures in the U.S. to evaluate their applicability to steel structures in Canada. From comprehensive reviews of available seismic design and evaluation standards and guidelines, it was found that ASCE 41-17 (2017) refers to the steel design standards to define some nonlinear modelling parameters, especially the elastic part of the force and deformation relationship of steel structural elements, whereas the plastic region of structural models has been developed based on compiled experimental results and experts' opinion. The acceptance criteria for various structural performance levels are defined based on the backbone curve for each structural element. Thus, the modelling parameters of the plastic region and the acceptance criteria could be directly adopted to evaluate the seismic performance of steel structures in Canada, while care is needed to adopt the modelling parameters of the elastic region because Canada and the U.S. adopt different steel design standards.

After carrying out a comprehensive review of the design standards of the two countries, this study suggested five types of recommendation to apply the nonlinear modelling parameters and acceptance criteria in ASCE 41-17 (2017) to steel structures in Canada. Depending on the design equations, some modelling parameters that are proposed based on the first principles are adopted as is, while others, especially the modelling parameters that are determined based on the steel design standards in the U.S., are replaced by the corresponding steel design standards in Canada. Finally, an application example showed the overall procedure to evaluate the seismic performance of steel structures in accordance with the recommendation proposed in this study. Following this example,

603 the nonlinear modelling parameters and acceptance criteria that are obtained or updated in future
604 studies could be easily implemented in practice.

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609 **Data availability:** All data, models, and codes generated or used during the study appear in the submitted
610 article.

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672

673

674 **Table 1.** Modelling parameters of steel structural elements

Element	Strength	Yield deformation (stiffness)
General assumptions	Unless other procedures are specified in ASCE 41-17 (2017), procedures contained in AISC 360 or AISC 341 to calculate design strength are permitted, except that the strength reduction factor, ϕ, is taken as 1.0.	Component stiffnesses are calculated in accordance with Sections 9.4 through 9.12 of ASCE 41-17 (2017).
Beam column elements–flexural	For flexural actions of beams and columns expected to develop plastic hinges, the expected flexural strength at the hinge locations is Beam $M_{CE} = M_{pe}$ Column $M_{CE} = M_{PCE} = \begin{cases} \text{for} \dfrac{\lvert C \rvert}{C_{ye}} < 0.2 & M_{pe}\left(1 - \dfrac{\lvert C \rvert}{2C_{ye}}\right) \\ \text{for} \dfrac{\lvert C \rvert}{C_{ye}} \geq 0.2 & M_{pe}\dfrac{9}{8}\left(1 - \dfrac{\lvert C \rvert}{2C_{ye}}\right) \end{cases}$ where $M_{CE} =$ expected flexural strength of a member or joint; $M_{pe} =$ expected plastic moment capacity of the section; $M_{pce} =$ expected plastic moment capacity of the section reduced for the effect of axial force. $C =$ axial force in the column; $C_{ye} =$ expected axial yield capacity of the column.	Yield chord rotation of beam $\theta_y = M_{pe}L(1+\eta)/6EI$ where $L =$ length of the component; $E =$ Young's modulus; $I =$ moment of inertia; $\eta =$ Adjustment factor to account for the effect of shear deformation. Yield chord rotation of column $\theta_y = M_{pce}L(1+\eta)/6\tau_b EI$ where $\tau_b =$ stiffness reduction parameter.
Panel zones	For shear actions in panel zones, the expected shear strength, V_{CE} refers to the plastic shear capacity of the panel zone, which is calculated using $V_{CE} = V_{ye} = \begin{cases} \dfrac{\lvert C \rvert}{C_{ye}} < 0.4 & 0.55F_{ye}d_c w \\ \dfrac{\lvert C \rvert}{C_{ye}} \geq 0.4 & 0.55F_{ye}d_c w\left(1.4 - \dfrac{\lvert C \rvert}{C_{ye}}\right) \end{cases}$ where $V_{ye} =$ expected yield shear strength; $F_{ye} =$ expected yield strength; $d_c =$ column depth; $w =$ total thickness of panel zone, including any doubler plates.	The shear yield strain, γ_y, for a panel zone is calculated in accordance with $\gamma_y = \dfrac{F_{ye}}{G\sqrt{3}}\sqrt{1 - \left(\dfrac{\lvert C \rvert}{C_{ye}}\right)^2}$ where $G =$ shear modulus of steel.
Connections (Fully or partially restrained)	**Fully restrained (FR)** Strength of a connection is based on the controlling limit state considering all potential modes of failure. See Section 9.4.2.3 of ASCE 41-17 (2017) for details. **Partially restrained (PR)**	**Fully restrained (FR)** Modelling of a FR connection stiffness is required except for joints that are intentionally reinforced to force formation of plastic hinges within the beam span, remote from the column face

	Equations to estimate the strength of various connection types are provided in ASCE 41-17 (2017). See Section 9.4.3.3 of ASCE 41-17 (2017) for details.	**Partially restrained (PR)** The rotational spring stiffness K_θ of a PR connection is determined by $$K_\theta = \frac{M_{CE}}{0.005}$$ where $M_{CE} =$ expected moment strength of connection of three specific PR connection listed in Section 9.4.3.2.1 of ASCE 41-17 (2017). The stiffness of those not listed in Section 9.4.3.2.1 of ASCE 41-17 (2017) is calculated as $$K_\theta = \frac{M_{CE}}{0.003}$$ Note that an alternative procedure is provided in Section 9.4.3.2 of ASCE 41-17 (2017).
Braces in buckling-restrained braced frame	Braces are modelled with the stiffness of the yielding core segment and transition segment added in series. The transition segment should include the properties of the brace that is stiffened from the end of the core to the gusset connection. It is permitted to assume the gusset and beam–column joint as rigid relative to the brace for axial stiffness.	The expected yield strength is the net area of the core multiplied by the expected yield stress. For strength and modelling parameters, the expected yield stress is taken as the specified minimum yield stress multiplied by R_y from AISC 341, where R_y the is AISC 341 factor to translate from lower-bound to expected values
Link beams; Beam column elements – shear	The expected strength of the link beam is governed by shear yielding, V_{CE}, or flexural yielding, M_{CE}. Where flexural yielding governs, the expected strength is converted to the resultant shear demand V. For shear yielding, the following equation is employed to estimate the expected strength $Q_{CE} = Q_y = V_{CE}$ of the link beam $$V_{CE} = V_{ye} = \begin{cases} \frac{\lvert C\rvert}{C_{ye}} \le 0.2 & 0.6F_{ye}A_s \\ \frac{\lvert C\rvert}{C_{ye}} > 0.2 & 0.6F_{ye}A_s\sqrt{1-\left(\frac{\lvert C\rvert}{C_{ye}}\right)^2} \end{cases}$$ where $A_s =$ Effective shear area of the cross section. For flexural yielding, the following is employed to compute the expected strength $Q_{CE} = Q_y = V$ of the link beam $$V = 2M_{CE}/e$$ where $M_{CE} =$ expected plastic moment capacity of the section; $e =$ length of the link beam.	The yield chord rotation of the link beam is given as $$\theta_y = Q_{CE}/K_e e$$ where $K_e = 12EI/(e^3(1+\eta))$ elastic stiffness of the link beam; $Q_{CE} =$ expected shear strength of the beam; $e =$ length of the link beam.
Steel plate used as shear walls	The wall is permitted to be modelled as the web of a plate girder. If stiffeners are provided to prevent buckling, they are spaced according to the requirements for plate girders given in AISC 360 and the expected strength of the wall is determined by	Unless another method based on principles of mechanics is used, the global stiffness of the wall of a steel plate shear wall K_w is calculated as $$K_w = \frac{Gaw}{h}$$ where

	$Q_{CE} = V_{CE} = 0.6F_{ye}aw$ where $F_{ye} =$ expected yield strength; $a =$ clear width of the wall between vertical boundary elements; $w =$ thickness of plate wall.	$a =$ clear width of wall between vertical boundary elements; $h =$ clear height of wall between horizontal boundary elements; $w =$ thickness of steel plate shear wall. The yield deformation $\Delta_y = \frac{Q_{CE}}{K_W}$ where $Q_{CE} =$ expected shear strength of the steel plate shear wall.
Structural steel frame with infill	The engineering properties and modelling parameters for the infill walls comply with the requirements in Chapter 10 and 11 of ASCE 41-17 (2017) for concrete and masonry, respectively.	
Diaphragms	ASCE 41-17 (2017) provides the detailed modelling parameters by mostly following SDI Diaphragm Design Manual (Luttrell et al., 2004). The SDI Diaphragm Design Manual is the basis of ANSI/AISI S310 *North American Standard for the Design of Profiled Steel Diaphragm Panels*, which is a current design standard for diaphragms that are used in both Canada and the U.S.	
Cast and wrought iron	Since cast and wrought iron components considered as force-controlled behaviour, the following lower-bound compressive strength is used. $C_{CL} = A_g F_{cr}$ where A_g Gross area of column; $F_{cr} =$ critical stress.	Modulus of elasticity are provided in ASCE 41-17 (2017). Because of force-controlled behaviour, no yield deformation is needed.
Cold-formed structure	ASCE 41-17 (2017) provides the detailed modelling parameters by mostly following ANSI/AISI S100 which is *North American Specification for the Design of Cold-Formed Steel Structural Members* used in both Canada and the U.S.	

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677
678 **Table 2** Comparison between design standards, specifications, and codes of Canada and the U.S.

Country	Seismic Evaluation	Steel Design	Design Load
Canada	NRC seismic evaluation guidelines Commentary L of the NBC	CSA S16 (Structural steel) CSA S136 (Cold-formed steel)	NBC
U.S.	ASCE/SEI 41	AISC 360 (Structural steel) AISC 341 (Structural steel) AISI S310 (Cold-formed steel)	ASCE 7

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682 **Table 3** Classification of recommended nonlinear modelling parameters

Type	Description	Recommendation
1	**Most deformation-related modelling parameters that are not presented in design standards but are only in ASCE 41-17 (2017)** Because the current structural design practice is force-based (i.e., strength capacity needs to be greater than the force demand), only strength capacities are defined in the design standards without definition of yield deformation nor inelastic behaviour. In ASCE 41-17 (2017), the yield deformation and other inelastic deformation parameters are defined primarily based on structural mechanics, experimental results, and experts' opinions. Thus, it is recommended to adopt these modelling parameters as is for seismic evaluation and upgrading of steel structures in Canada.	Adopt as is.
2	**Equations in ASCE 41-17 (2017) with relevant and identical expressions in design standards in the U.S. (AISC 360 and AISC 341) and Canada (CSA S16)** There are equations in ASCE 41-17 (2017) to define various conditions or to calculate strengths. If relevant equations are available in the design standards (AISC 341, AISC 360, and CSA S16), and if the equations in the U.S. and Canadian standards are identical, then it is recommended to adopt the equations in ASCE 41-17 (2017) as is.	Adopt as is if the equations in CSA S16 and AISC 341/AISC 360 are identical to those is ASCE 41.
3	**Equations in ASCE 41-17 (2017) with relevant yet inconsistent expressions in design standards in the U.S. (AISC 360 and AISC 341) and Canada (CSA S16)** There are equations in ASCE 41-17 (2017) to define various conditions or to calculate strengths. If relevant equations are available in the design standards (AISC 341, AISC 360, and CSA S16), and if the equations in the U.S. and Canadian standards are different, then it is recommended to adopt the equations in CSA S16. This is because even though the code equations in the Canada and in the U.S. result in similar strengths, it would not be consistent if practicing engineers use equations in CSA for design and the equations in AISC for evaluation and upgrading.	Adopt the equations in CSA S16 if the equations in AISC 341/AISC 360 and CSA S16 are different.
4	**Equations in ASCE 41-17 (2017) with relevant expressions only in design standards in the U.S. (AISC 341 and AISC 360) not in Canada (CSA S16)** For some equations in ASCE 41-17 (2017), the relevant equations are only in the U.S. standards, not in Canadian standards. In this case, it is suggested to adopt ASCE 41-17 (2017) as is.	Adopt as is.
5	**Modelling parameters that directly refer to design standards.** For many strength and stiffness parameters, as shown in Table 1, ASCE 41-17 (2017) refers to AISC 360 and AISC 341 without providing the equations explicitly. For these equations, it is recommended to substitute the equations with the corresponding equations in CSA S16.	Refer to the corresponding equations in CSA S16.

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685
686 **Table 4.** Member size for 6-storey steel MRF

Level	Column Sections		Beam Sections
	Interior	Exterior	
6	W360×45	W310×45	W360×39
5	W360×45	W310×45	W410×46
4	W360×72	W310×60	W460×60
3	W360×72	W310×60	W460×60
2	W360×122	W310×74	W530×74
1	W360×122	W310×74	W530×74

687

688 **Table 5.** Modelling parameters of elastic region for beam elements

Beam Sections	Yield Strength (kN-m)	Yield Deformation (rad)
W530×74	686.895	0.008

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691
692 **Table 6.** Modelling parameters of plastic region for beam element (a, b, and c refer to Figure 3)

Beam Sections	a	b	c	IO	LS	CP
W530×74	0.024	0.048	0.2	0.015	0.036	0.048

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695
696 **Table 7.** Modelling parameters of elastic region for column element

End Nodes	Axial Strength Ratio	Yield Strength (kN-m)	Yield Deformation (rad)
1 – 2	0.203	420	0.011
8 – 9	0.222	788	0.009
15 – 16	0.222	788	0.009
22 – 23	0.203	420	0.011

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699
700 **Table 8**. Modelling parameters of plastic region for column element (a, b, and c refer to Figure 3)

End Nodes	a	b	c	IO	LS	CP
1 – 2	0.012	0.030	0.717	0.006	0.022	0.030
8 – 9	0.015	0.035	0.700	0.007	0.026	0.035
15 – 16	0.015	0.035	0.700	0.007	0.026	0.035
22 – 23	0.012	0.030	0.717	0.006	0.022	0.030

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702
703
704 **Table 9.** Modelling parameters of panel zone

End Nodes	Yield Strength (kN)	Yield Deformation	a	b	c	IO	LS	CP
2 or 23	322	0.003	0.033	0.033	1.000	0.003	0.033	0.033
9 or 16	521	0.003	0.033	0.033	1.000	0.003	0.033	0.033

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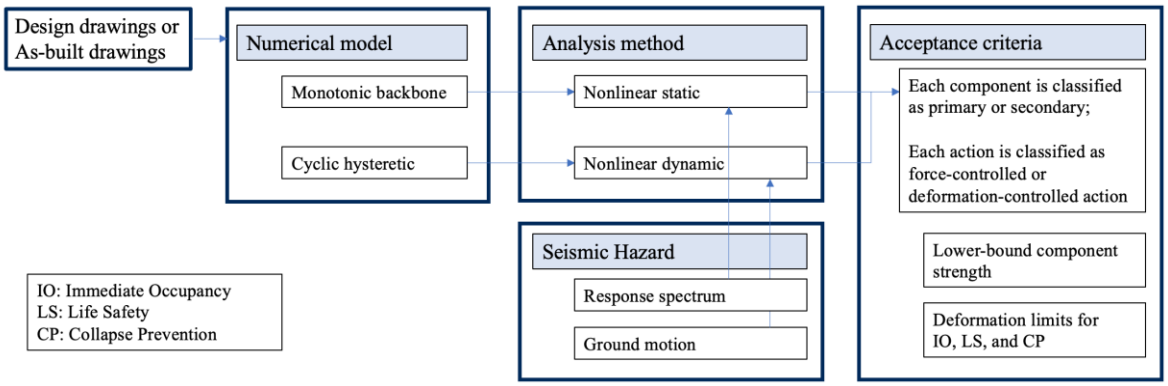


Figure 1. Flowchart of seismic performance evaluation based on ASCE 41-17 (2017)

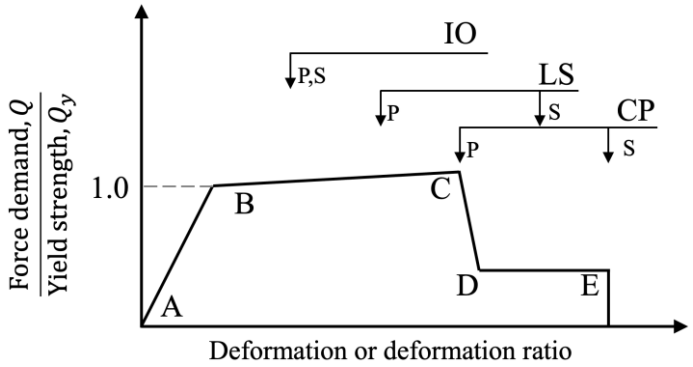


Figure 2. Illustration of acceptance criteria (adapted from ASCE 41-17 (2017))

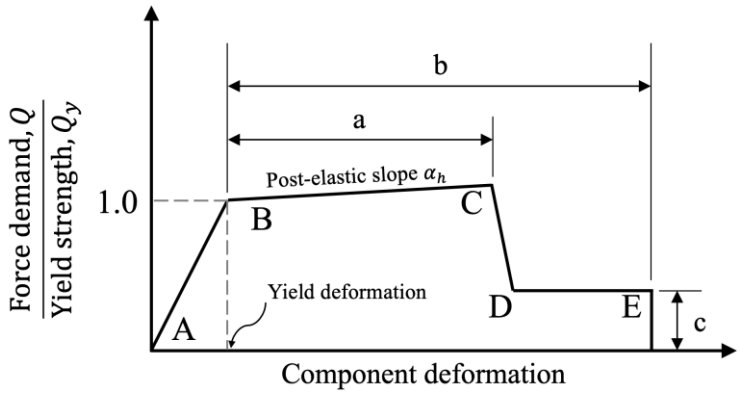


Figure 3. Generalized force-deformation relation for steel components (adapted from ASCE 41-17 (2017))

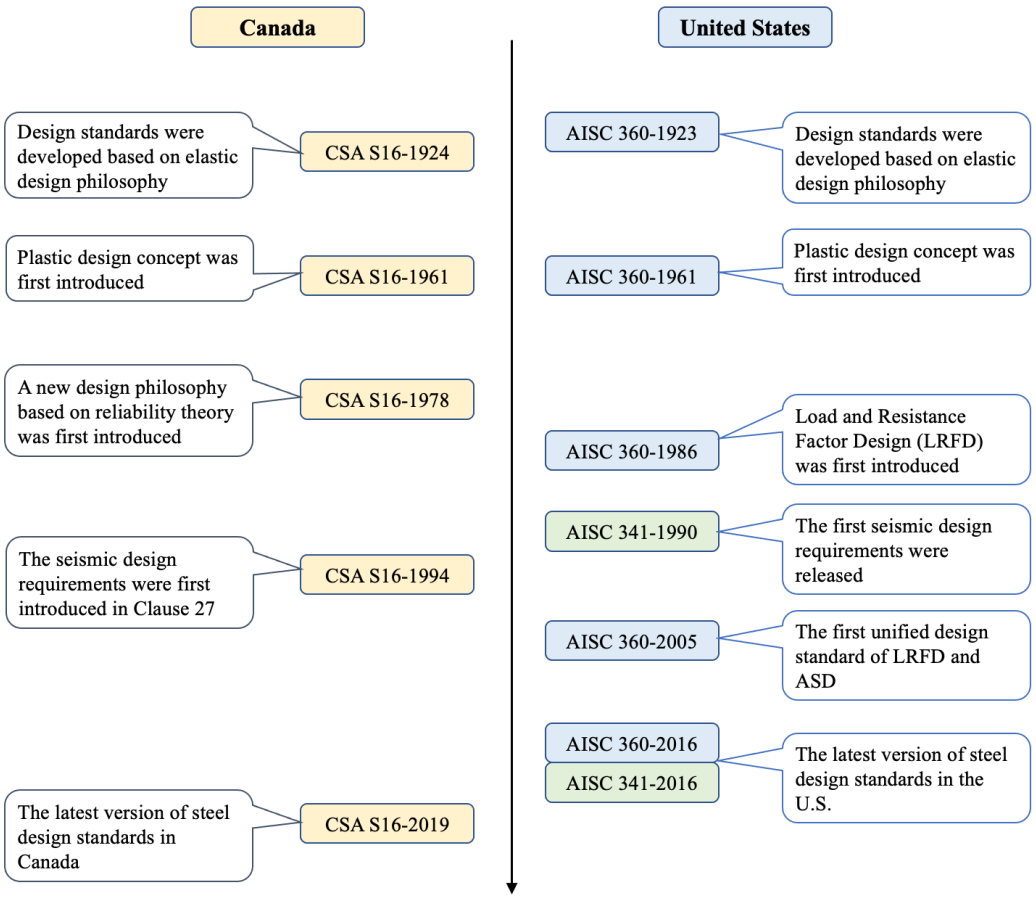
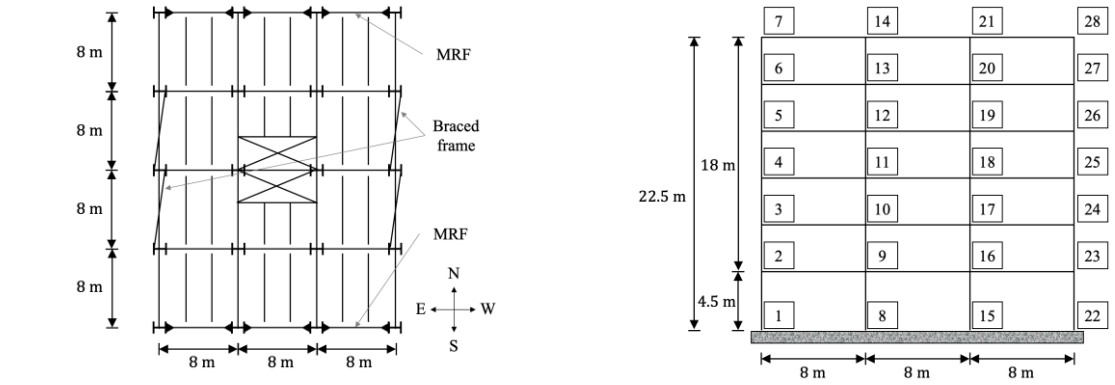


Figure 4. Evolution of steel design standards in Canada and the U.S.



(a) (b)

Figure 5. Reference 6-storey steel MRF

Appendix A

The seismic performance of a steel structure is evaluated using the defined modelling parameters and acceptance criteria in Section 5. In order to illustrate the procedure to both practicing engineers and researchers, two different structural analysis models were created. The first model was developed with Perform 3D (Computers & Structures (CSI) Inc., 2006), while the second model was developed using the *Framework for Integration of Seismic Models* (SFIM; Acosta, 2019). SFIM is open-source and it was implemented as a series of Python scripts (https://github.com/joacostar/SFIM_v2) that can be used to create a nonlinear analysis model for OpenSees (McKenna, 2011), starting from an elastic model developed with the commercial software S-FRAME Analysis (Stylianou, 2015).

A.1. Seismic hazard

According to Commentary L of NBC 2015 (NRC, 2015b), different levels of hazard are employed along with the assessment/upgrading levels. However, for the purpose of illustration, this study adopts a seismic hazard with a probability of exceedance of 2% in 50 years (return period: 2,475 years) at Montreal region in NBC 2015 (2015a). The corresponding design spectrum is presented in Figure A1 assuming site class C. Later, when assessing the developed numerical models, the seismic demand corresponding to the seismic hazard with 2,475 years of return period is not enough to develop the nonlinear behaviour of the structure. Thus, a scale factor of 3 is used when assessing the structures (i.e., amplification factor 3 is multiplied to the design spectrum in Figure A1) to demonstrate the NSP.

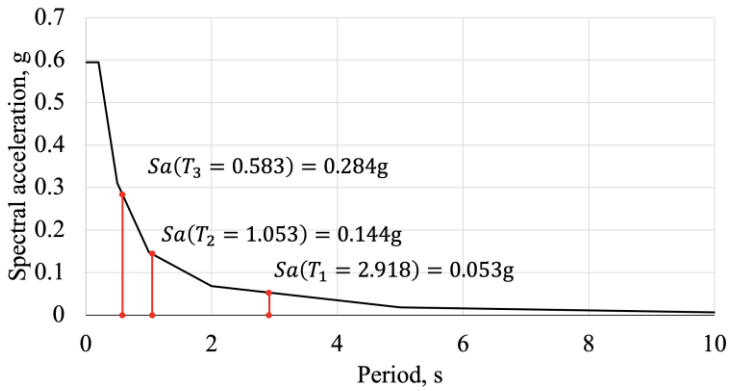


Figure A1. Design spectrum of Montreal where T_1, T_2, and T_3 are the first three periods of the frame structure

753 A.2. Modelling parameters and acceptance criteria

754 Detailed procedures for defining the modelling parameters and acceptance criteria of the beam,
755 column, and panel zone are provided in Section 5. Since Section 5 only provides modelling
756 parameters and acceptance criteria of some structural elements of the frame structure, this subsection
757 provides those of other structural elements that are not handled in Section 5. The results are
758 summarized in Tables A1 to A5.

759

760 **Table A1.** Modelling parameters of elastic region for beam elements

Levels	Beam Sections	Yield Strength (kN-m)	Yield Deformation (rad)
6	W360×39	251.229	0.012
5	W410×46	335.858	0.011
4	W460×60	485.760	0.009
3	W460×60	485.760	0.009
2	W530×74	686.895	0.008
1	W530×74	686.895	0.008

761

762 **Table A2.** Modelling parameters of plastic region for beam element

Levels	a	b	c	IO	LS	CP
6	0.024	0.048	0.2	0.015	0.036	0.048
5	0.024	0.048	0.2	0.015	0.036	0.048
4	0.024	0.048	0.2	0.015	0.036	0.048
3	0.024	0.048	0.2	0.015	0.036	0.048
2	0.024	0.048	0.2	0.015	0.036	0.048
1	0.024	0.048	0.2	0.015	0.036	0.048

763

764 **Table A3.** Modelling parameters of elastic region for column element

End nodes	Axial strength ratio	Yield strength (kN-m)	Yield deformation (rad)
1 – 2	0.203	419.810	0.0105
8 – 9	0.222	788.297	0.0092
15 – 16	0.222	788.297	0.0092
22 – 23	0.203	419.810	0.0105
2 – 3	0.166	439.390	0.0093
9 – 10	0.181	829.993	0.0083
16 – 17	0.181	829.993	0.0083
23 – 24	0.166	439.390	0.0093
3 – 4	0.164	348.045	0.0094
10 – 11	0.241	433.802	0.0076
17 – 18	0.241	433.802	0.0076
24 – 25	0.164	348.045	0.0094
4 – 5	0.119	354.074	0.0095
11 – 12	0.175	471.531	0.0083
18 – 19	0.175	471.531	0.0083
25 – 26	0.119	354.074	0.0095
5 – 6	0.098	268.686	0.0091

12 – 13	0.173	287.662	0.0080
19 – 20	0.173	287.662	0.0080
26 – 27	0.098	268.686	0.0091
6 – 7	0.038	268.686	0.0091
13 – 14	0.070	295.631	0.0082
20 – 21	0.070	295.631	0.0082
27 – 28	0.038	268.686	0.0091

765

766 **Table A4**. Modelling parameters of plastic region for column element (a, b, and c refer to Figure 3)

End Nodes	a	b	c	IO	LS	CP
1 – 2	0.012	0.030	0.717	0.006	0.022	0.030
8 – 9	0.015	0.035	0.700	0.007	0.026	0.035
15 – 16	0.015	0.035	0.700	0.007	0.026	0.035
22 – 23	0.012	0.030	0.717	0.006	0.022	0.030
2 – 3	0.013	0.037	0.667	0.008	0.028	0.037
9 – 10	0.017	0.044	0.683	0.009	0.033	0.044
16 – 17	0.017	0.044	0.683	0.009	0.033	0.044
23 – 24	0.013	0.037	0.667	0.008	0.028	0.037
3 – 4	0.009	0.035	0.591	0.005	0.027	0.036
10 – 11	0.009	0.023	0.683	0.005	0.017	0.023
17 – 18	0.009	0.023	0.683	0.005	0.017	0.023
24 – 25	0.009	0.035	0.591	0.005	0.027	0.036
4 – 5	0.011	0.039	0.624	0.005	0.031	0.041
11 – 12	0.009	0.029	0.604	0.006	0.022	0.029
18 – 19	0.009	0.029	0.604	0.006	0.022	0.029
25 – 26	0.011	0.039	0.624	0.005	0.031	0.041
5 – 6	0.008	0.032	0.629	0.005	0.024	0.032
12 – 13	0.004	0.029	0.414	0.002	0.024	0.032
19 – 20	0.004	0.029	0.414	0.002	0.024	0.032
26 – 27	0.008	0.032	0.629	0.005	0.024	0.032
6 – 7	0.010	0.038	0.702	0.005	0.029	0.038
13 – 14	0.005	0.035	0.465	0.003	0.031	0.042
20 – 21	0.005	0.035	0.465	0.003	0.031	0.042
27 – 28	0.010	0.038	0.702	0.005	0.029	0.038

767

768 **Table A5.** Modelling parameters of panel zone

End Nodes	Yield Strength (kN)	Yield Deformation	a	b	c	IO	LS	CP
2	321.751	0.003	0.033	0.033	1.000	0.003	0.033	0.033
9	521.051	0.003	0.033	0.033	1.000	0.003	0.033	0.033
16	521.051	0.003	0.033	0.033	1.000	0.003	0.033	0.033
23	321.751	0.003	0.033	0.033	1.000	0.003	0.033	0.033
3	321.751	0.003	0.033	0.033	1.000	0.003	0.033	0.033
10	521.051	0.003	0.033	0.033	1.000	0.003	0.033	0.033
17	521.051	0.003	0.033	0.033	1.000	0.003	0.033	0.033
24	321.751	0.003	0.033	0.033	1.000	0.003	0.033	0.033
4	215.819	0.003	0.033	0.033	1.000	0.003	0.033	0.033
11	285.859	0.003	0.033	0.033	1.000	0.003	0.033	0.033
18	285.859	0.003	0.033	0.033	1.000	0.003	0.033	0.033
25	215.819	0.003	0.033	0.033	1.000	0.003	0.033	0.033
5	215.819	0.003	0.033	0.033	1.000	0.003	0.033	0.033

12	285.859	0.003	0.033	0.033	1.000	0.003	0.033	0.033
19	285.859	0.003	0.033	0.033	1.000	0.003	0.033	0.033
26	215.819	0.003	0.033	0.033	1.000	0.003	0.033	0.033
6	173.767	0.003	0.033	0.033	1.000	0.003	0.033	0.033
13	204.301	0.003	0.033	0.033	1.000	0.003	0.033	0.033
20	204.301	0.003	0.033	0.033	1.000	0.003	0.033	0.033
27	173.767	0.003	0.033	0.033	1.000	0.003	0.033	0.033
7	152.208	0.003	0.033	0.033	1.000	0.003	0.033	0.033
14	178.954	0.003	0.033	0.033	1.000	0.003	0.033	0.033
21	178.954	0.003	0.033	0.033	1.000	0.003	0.033	0.033
28	152.208	0.003	0.033	0.033	1.000	0.003	0.033	0.033

A.3. Numerical structural model

A two-dimensional numerical model was developed in Perform 3D, while a three-dimensional model was generated with SFIM. An overview of the S-FRAME model is presented in Figure A2. Since the model from Perform 3D is two-dimensional, leaning columns (gravity columns) are introduced at the rightmost part of the structure in order to take into account P-Delta effects from gravity columns. The gravity columns are modelled as a linear element whose sectional properties are tabulated in Table A6. The masses obtained from Gómez et al., (2015) are applied to each node. The first three natural periods of the structure are 2.918 s, 1.053 s, and 0.583 s. The corresponding spectral acceleration values are shown in Figure A1.

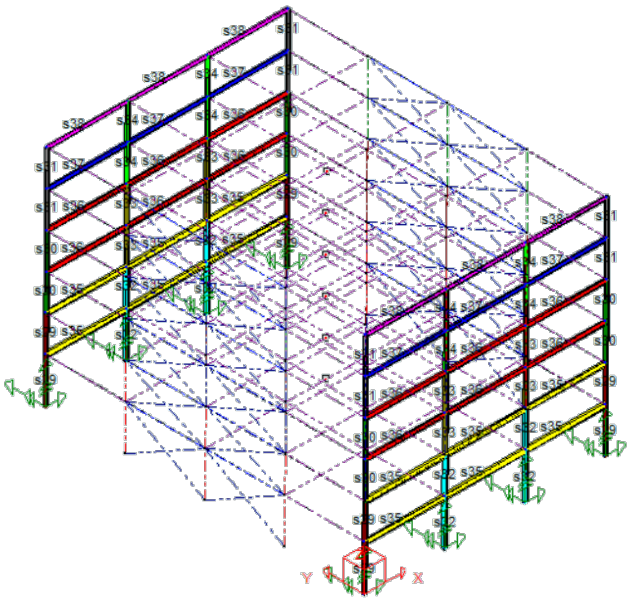


Figure A2. 3-dimensional view of S-FRAME model. Only elements to be modelled with nonlinear components are shown.

784 **Table A6.** Member size of gravity columns

Level	Gravity Column Sections	
	Interior	Exterior
6	W310×38.7	W310×23.8
5	W310×38.7	W310×23.8
4	W310×52	W310×38.7
3	W310×52	W310×38.7
2	W310×74	W310×44.5
1	W310×74	W310×44.5

785

786 Gravity loads were applied in each model according to the load combinations from Section 7 of

787 ASCE 41-17 (2017) given by

$$Q_G = Q_D + 0.25Q_L + 0.2Q_S \quad \text{(A1)}$$

788 where Q_G, Q_D, Q_L, and Q_S respectively represent the gravity, dead, live, and snow load. The loads
789 taken from Gómez et al., (2015) are applied uniformly distributed on beam elements of the MRF.
790 Although this numerical investigation follows the load combination rule presented in ASCE 41-17
791 (2017), further investigation is needed to examine whether the provided load combination is able to
792 fully consider the characteristics of loading scenarios for steel structures in Canada. The numerical
793 models of Perform 3D and SFIM can be downloaded at (URL is provided once the paper is accepted.)

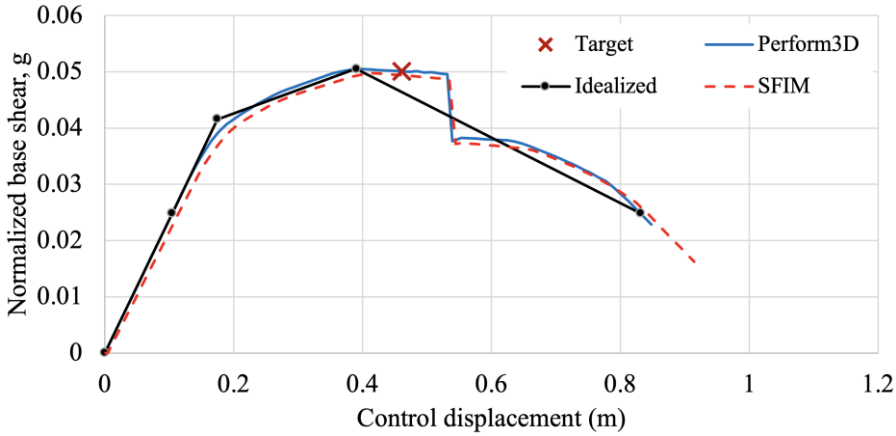
794 A.4. Seismic performance assessment

795 The NSP was employed to assess the performance of the building. To this end, a pushover analysis
796 was first carried out. The results are presented in Figure A3 with the idealized force and displacement
797 relationship obtained by following the procedure in ASCE 41-17 (2017). Using the idealized bilinear
798 force-deformation relationship, the target displacement can be estimated using the coefficient method
799 which is given by

$$\delta_t = C_0 C_1 C_2 S_a \frac{T_e^2}{4\pi^2} \mathrm{g} \quad \text{(A2)}$$

800 where C_0 is the modification factor to relate spectral displacement of an equivalent single-degree-of
801 freedom (SDOF) system to the roof displacement of a multi-degree-of-freedom (MDOF) system, C_1
802 is the modification factor to relate expected maximum inelastic displacements to displacements
803 calculated for a linear elastic response, C_2 is the modification factor to represent the effect of pinched

804 hysteresis shape, cyclic stiffness degradation, and strength deterioration on the maximum
805 displacement response, S_a is the response spectrum acceleration at the effective period, g is the
806 acceleration of gravity. Note that as described in Section 2.1, other NSP methods could be employed
807 in this regard such as Capacity Spectrum Method in FEMA 440 (2005) or deep learning-based
808 approach by Kim et al., (2019, 2020).
809



810
811 **Figure A3.** Pushover curves of steel MRF model from Perform 3D (blue solid), SFIM (red dashed),
812 and idealized relationship (black circle)
813

814 According to ASCE 41-17 (2017), since the coefficient method is not permitted when the ratio of
815 elastic strength demand to yield strength coefficient $\mu_{strength}$ exceed the maximum strength ratio
816 μ_{max}, the two ratios are calculated before estimating the target displacement. By carrying out the
817 linear analysis with and without including the P-Delta effect in accordance with Eq. (7-32) of ASCE
818 41-17 (2017), μ_{max} is calculated as 4.24. Since $\mu_{strength} = 3.81$ according to Eq. (7-31) of ASCE 41-17
819 (2017), which is smaller than μ_{max}, it is allowable to use the coefficient method to estimate the target
820 displacement.

821 Since the effective period $T_e = 2.92\,s$ from the idealized force-deformation relationship in
822 Figure A3, the corresponding period of spectral acceleration from the defined hazard is calculated as
823 $S_a = 0.158$ g, where g is the gravitational acceleration. Note that as described in Section A.1, this
824 spectral acceleration is obtained after amplifying the design spectrum three times. Following the
825 procedure in ASCE 41-17 (2017), the coefficients in Eq. (A2) are calculated as: $C_0 = 1.38$, $C_1 = 1$, and

826 $C_2 = 1$. By substituting them into Eq. (A2), the target displacement (i.e., performance point) is
827 calculated as 0.461 m which is denoted as the red x mark in Figure A3.

828 In order to evaluate the seismic performance of the structure, maximum demands of each
829 structural component experienced up to the target displacement point are calculated. Note that since
830 the performance levels are determined in terms of the plastic rotation angle, the yield rotation is
831 deducted from the total rotation from analysis. The corresponding demand values from Perform 3D
832 and SFIM are summarized in Table A7, with the capacity limits of different performance levels. To
833 effectively represent the results, structural elements that exceed their IO capacity are shown in the
834 table. Note that the node numbering refers to Figure 5(b).

835
836 **Table A7.** Capacity of each performance level and demand at the target displacement

Element	End	Capacity			Demand	
Type	Nodes	IO	LS	CP	Perform 3D	SFIM
Column	1 – 2	0.006	0.022	0.030	0.006	0.004
Column	8 – 9	0.007	0.026	0.035	0.009	0.005
Column	15 – 16	0.007	0.026	0.035	0.008	0.005
Column	22 – 23	0.006	0.022	0.030	0.007	0.004
Panel Zone	2	0.003	0.033	0.033	0.011	0.008
Panel Zone	9	0.003	0.033	0.033	0.016	0.014
Panel Zone	16	0.003	0.033	0.033	0.015	0.013
Panel Zone	23	0.003	0.033	0.033	0.015	0.012
Panel Zone	3	0.003	0.033	0.033	0.011	0.010
Panel Zone	10	0.003	0.033	0.033	0.016	0.016
Panel Zone	17	0.003	0.033	0.033	0.015	0.014
Panel Zone	24	0.003	0.033	0.033	0.015	0.014
Panel Zone	4	0.003	0.033	0.033	0.012	0.011
Panel Zone	11	0.003	0.033	0.033	0.021	0.021
Panel Zone	18	0.003	0.033	0.033	0.019	0.019
Panel Zone	25	0.003	0.033	0.033	0.019	0.018
Panel Zone	5	0.003	0.033	0.033	0.007	0.007
Panel Zone	12	0.003	0.033	0.033	0.016	0.016
Panel Zone	19	0.003	0.033	0.033	0.014	0.014
Panel Zone	26	0.003	0.033	0.033	0.014	0.015
Panel Zone	13	0.003	0.033	0.033	0.009	0.008
Panel Zone	20	0.003	0.033	0.033	0.007	0.007
Panel Zone	27	0.003	0.033	0.033	0.008	0.008

837
838 As shown in Table A7, Perform 3D and SFIM produce similar maximum demand values, which
839 indicates that it is possible to simulate the behaviour of the lateral load resisting system in 2D and 3D
840 numerical models. It is seen that the damage has been observed at the base of the first storey columns

841 and in panel zones except for the roof top of the frame. However, all the demands are below the LS
842 capacity of each component. Since the demands of most of the panel zone exceed the IO performance
843 level, it can be concluded that the panel zones are the most vulnerable elements in the MRF structure.
844 Although this study uses all three performance levels defined in ASCE 41-17 (2017), further study is
845 needed to determine a more refined performance level for steel structures in Canada considering the
846 importance of structure, uncertainties of modelling parameters, and seismic hazard of Canada.
847