Comparison of Seismic Design Provisions for Buckling Restrained Braced Frames in Canada, United States, Chile, and New Zealand

R. Tremblay[1], M. Dehghani[2], L. Fahnestock[3], R. Herrera[4], M. Canales[5], C. Clifton[6], and Z. Hamid[7]

[1]Professor, [2]Graduate Researcher, Dept. of Civil, Geological and Mining Eng., Polytechnique Montreal, Canada

e-mails: robert.tremblay@polymtl.ca; morteza.dehghani@polymtl.ca

[3]Associate Professor, Dept. of Civil and Env. Eng., University of Illinois at Urbana-Champaign, Urbana, Illinois, USA

e-mail: fhnstck@illinois.edu

[4]Assistant Professor, [5]Graduate Researcher, Dept. of Civil Eng., University of Chile, Santiago, Chile

e-mails: riherrer@ing.uchile.cl, marisol.canales@ing.uchile.cl

[6]Associate Professor, Dept. of Civil & Environmental Eng., University of Auckland, Auckland, New Zealand

e-mail: c.clifton@auckland.ac.nz

[7]Structural Engineer, Steel Construction New Zealand Inc., Auckland, New Zealand

e-mail: Zahid.Hamid@scnz.org

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Abstract. *Seismic design provisions for buildings in Canada, the United States, Chile and New Zealand are presented for buckling restrained braced frames, with focus on design requirements for seismic stability. P-delta effects are explicitly considered in seismic design in Canada, the U.S. and New Zealand. In Chile, stability effects are limited by means of more stringent drift limits. The provisions are applied to a 9-storey building structure located in areas in each country having similar seismic conditions. For this structure, comparable seismic loads are specified in Canada and Chile, whereas significantly lower seismic effects are prescribed in the U.S. In all countries, use of the dynamic (response spectrum) analysis method resulted in lighter and more flexible structures compared to the equivalent static force procedure. Seismic stability requirements had greater impact on designs in Canada and New Zealand. Frame design in the U.S. was only affected by stability effects when applying the stability requirements from AISC 360-10.*

Keywords: Braced frames; Buildings; Buckling restrained brace; Notional loads; Seismic design; Stability; Storey drifts.

1 INTRODUCTION

Buckling restrained braced frames (BRBFs) were introduced in Canada and the United States at the end of the 1990's and their use has since expanded considerably, especially in high seismic regions along the Pacific west coast. In Canada, seismic design provisions for BRBFs were implemented in 2010. The most recent seismic loading provisions are given in the 2015 National Building Code of Canada (NBCC) [1] whereas the latest design and detailing requirements are specified in the 2014 CSA S16 steel design standard [2]. BRBFs were introduced in the U.S. codes in 2005. These codes have since been updated and the latest available set of seismic design provisions are included in ASCE 7-10 [3] and the AISC 341-10 Seismic Provisions [4]. Buckling restrained braced frames have been used in New Zealand with the first application in 1992 to a building expansion and are being introduced in Chile but no specific guidance has been introduced yet in the building codes of these countries [5, 6]. The BRBF system is currently being considered for future editions of these codes. In New Zealand, a draft design guide has been published by Steel Construction New Zealand [7].

A comparative study [8] has shown that the design seismic loads prescribed for a 4-storey concentrically braced steel frame were significantly higher in Canada compared to those specified for the same frame in the U.S., resulting in greater steel tonnage required for the Canadian design. The difference was mainly attributed to the lower force modification factors specified in Canada. This paper presents a study performed to verify if similar conclusions would apply to buckling restrained braced frames used for medium-rise buildings. The seismic analysis is performed using both static and dynamic procedures to investigate possible variations due to the analysis methods. Particular attention is also paid to stability design requirements. Finally, the comparison is extended to also include design solutions obtained for a buckling restrained braced designed in accordance with the current Chilean building code.

The comparisons in this paper are performed for a regular 9-storey office building assumed to be located at sites with comparable seismic conditions in all four countries. The prototype structure as well as the seismic conditions prevailing at the selected sites are first described. The seismic provisions for each country are then summarized with focus on minimum lateral resistance and stability requirements under seismic loading. Static and dynamic analysis methods are described. Although codes in all countries studied include requirements for accidental in-plane torsion effects, they have been omitted in this study as the emphasis is put on lateral strength, stiffness and stability under seismic loading. Design and detailing requirements for BRBFs in Canada, the U.S., and New Zealand are also briefly reviewed. In the last section of the paper, the design of the prototype structure is performed for each country and similarities and differences are highlighted.

2 PROTOTYPE BUILDING

2.1 Geometry and gravity loading

The prototype braced frame building was adapted from the 9-storey model building studied in the SAC steel project [9]. The model structure was however modified as follows: 1) the penthouse structure was omitted, 2) the perimeter moment frames acting in the E-W direction were replaced by buckling restrained braced frames having a chevron bracing configuration, and 3) the orientation of the columns on the E-W perimeter walls was rotated by 90 degrees. The structure plan view and the braced frame elevation are shown in figure 1a. The building is an office building of the normal importance category. The design gravity loads are given in figure 1b. As shown, the building has a single-level basement and a taller first storey height, as commonly found in office buildings.

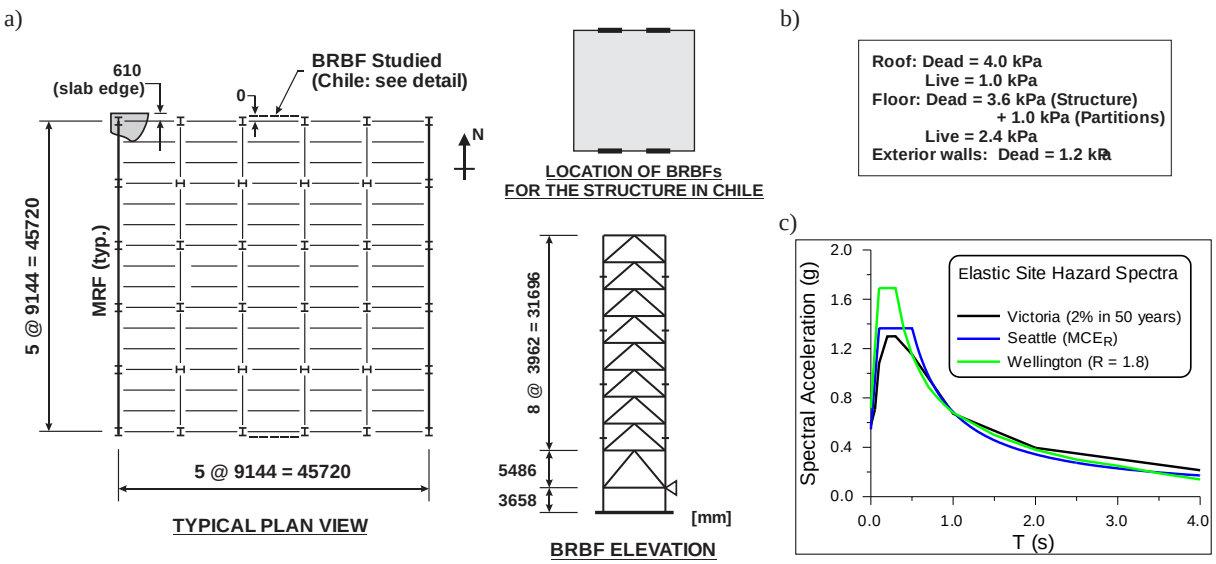

Figure 1: a) Prototype structure; b) Design gravity loads; c) Elastic hazard response spectra.

2.2 Building location and seismic data

The structure is assumed to be located at sites in Canada, United States, Chile, and New Zealand where similar seismic conditions and data prevail. The first three sites are located along the Pacific west coast: Victoria, BC, in Canada; Seattle, WA, in the U.S.; and Valparaiso, in Region V for Chile. Sites in Victoria and Seattle are geographically close to each other and both

are exposed to crustal and sub-crustal earthquakes as well as seismic ground motions originating from the Cascadia subduction zone. The seismicity in Valparaiso is dominated by large subduction earthquakes that occur frequently at the boundary of the Nazca and South American tectonic plates. As is the case for the other three sites, New Zealand is also located on the Ring of Fire rimming the Pacific Ocean. The selected site is located is Wellington, which is situated on a principally strike-slip region of the plate boundary between the Indian-Australian and the Pacific plates, but is also impacted by large subduction earthquakes from the Hikurangi subduction zone situation to the north of Wellington, which is capable of producing magnitude 8 and above earthquakes. For all sites, the structure is assumed to be constructed on soft rock, firm ground or very dense soil conditions, corresponding to site class C with a mean shear wave velocity between 360 and 760 m/s in Canada and U.S. and between 350 and 500 m/s in Chile, and to site class B with a mean shear wave velocity between 300 and 600 m/s in New Zealand.

In Canada, seismic design data consists of mean uniform hazard spectral (UHS) ordinates, S_a, specified at periods 0.2, 0.5, 1.0, 2.0, 5.0 and 10 s for a probability of exceedance of 2% in 50 years. The values for the chosen location are given in table 1. The values specified in the NBCC are determined for a site class C and did not need to be modified for this study. In the U.S., spectral accelerations are specified at short period (0.2 s) and one-second period, S_S and S_1, respectively. These parameters are referred to as risk-targeted maximum considered earthquake (MCE_R) values. They are determined for the same probability of exceedance (2% in 50 years) for a reference ground condition corresponding to site class B with adjustments to achieve a uniform targeted collapse probability. For other site classes, the values are modified using factors F_a and F_v to obtain appropriate spectral values $S_{MS} = F_a S_S$ and $S_{M1} = F_v S_1$. For the site in Seattle, S_S = 1.365 and S_1 = 0.528, and F_a = 1.0 and F_v = 1.30 for site class C. The resulting S_{MS} and S_{M1} values are given in table 1. As shown, they compare well with the values specified in Canada, confirming the similitude between the two chosen locations. In ASCE 7-10, the period T_L is the transition period between the medium- and long-period ranges of the spectrum, as will be discussed in Section 4. For Seattle, T_L = 6 s.

Table 1: Spectral ordinates in the 2015 NBCC, ASCE 7-10 and NZS1105-04.

T (s)	S_a (g) (NBCC)	S_M (g) (ASCE 7)	$C(T) = Z\, C_h(T)\, R$ (NZS1170.5)
0.2	1.30	1.365	1.65
0.5	1.16	-	1.15
1.0	0.676	0.686	0.68
2.0	0.399	-	0.38
5.0	0.125	-	0.11
10	0.0437	-	0.022

In New Zealand, the elastic site spectrum $C(T)$ is defined by the product of the hazard factor, Z, spectral shape factors $C_h(T)$ and the return period factor, R. The hazard factor Z also corresponds to the peak ground acceleration having a probability of exceedance of 10% in 50 years. For Wellington, Z = 0.40. For this study, $C_h(T)$ values for a site class B rock site. For comparison purposes, in table 1, the parameter R is taken equal to 1.8 to obtain $C(T)$ values with a probability of 2% in 50 years. As shown in figure 1c, the elastic response spectra in Victoria, Seattle, and Wellington for this hazard level are very similar. In Chile, the seismic input for design is essentially characterized by the maximum effective ground acceleration A_o at the site. Valparaiso is located in seismic zone 3 where A_o is equal to 0.40 g. Effective accelerations are not based on probabilistic seismic hazard assessment. For the site type studied in Valparaiso, PGA values are expected to reach 0.63-0.65 g for a probability of exceedance of

10% in 50 years [10]. This is larger than the peak ground accelerations for the class C sites in Victoria and Seattle for the same probability level: 0.31 g and 0.36 g, respectively. This is also higher than the factor *Z* (10% in 50 years PGA) specified for Wellington in New Zealand (0.4 g).

3 SEISMIC DESIGN PROVISIONS IN CANADA

3.1 Seismic loads and analysis methods

In NBCC 2015, the minimum design base shear, *V*, is given by:

$$V = \frac{S(T) M_{\mathrm{v}} I_{\mathrm{E}} W}{R_{\mathrm{d}} R_{\mathrm{o}}} \tag{1}$$

In this expression, *S (T)* is the design spectrum as a function of the structure fundamental period *T*, M_v accounts for higher mode effects on base shear, I_E is the importance factor, *W* is the seismic weight, and R_d and R_o are respectively the ductility- and overstrength-related force modification factors. The design spectrum is determined as follows, using linear interpolation for intermediate values of *T*:

$$\begin{aligned}
&S = \text{ larger of}: F(0.2) S_{\mathrm{a}}(0.2) \text{ and } F(0.5) S_{\mathrm{a}}(0.5) \text{ for } T < 0.2 \text{ s;}\\
&S = F(0.5) S_{\mathrm{a}}(0.5) \text{ for } T = 0.5 \text{ s;}\\
&S = F(1.0) S_{\mathrm{a}}(1.0) \text{ for } T = 1.0 \text{ s;}\\
&S = F(2.0) S_{\mathrm{a}}(2.0) \text{ for } T = 2.0 \text{ s;}\\
&S = F(5.0) S_{\mathrm{a}}(5.0) \text{ for } T = 5.0 \text{ s; and}\\
&S = F(10) S_{\mathrm{a}}(10) \text{ for } T \geq 10 \text{ s}
\end{aligned} \tag{2}$$

Values of $S_a(T)$ are the UHS ordinates at the site (see table 1) and $F(T)$ are site coefficients that depend on the site class and the structure period. Site class C corresponds to the reference ground condition considered for the determination of S_a values and *F* is therefore equal to 1.0 at every period. For steel braced frames, the period to be used in design, T_a, is taken equal to $T_a = 0.025\ h_n$, where h_n is the building height in meters. Alternatively, the period from dynamic analysis can be employed for T_a except that the period cannot be taken longer than $0.05\ h_n$ when determining member forces used to verify strength requirements. The upper period limit does not apply when calculating displacements or drifts. The permissible range of periods for strength design is plotted as a function of the building height in figure 2.

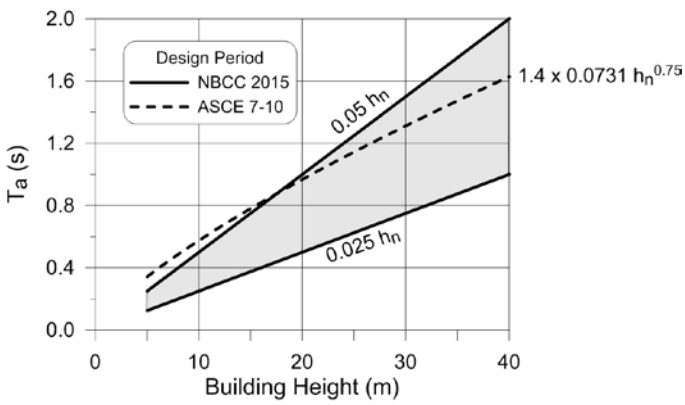

Figure 2: Range of building periods for seismic design of BRBFs in Canada and the U.S.

The M_v factor depends on the ratio $S(0.2)/S(5.0)$ at the site, the period of the structure and the SFRS type. For braced steel frames, M_v = 1.0 in most situations except for ratios $S(0.2)/S(5.0)$ greater than 40 in which case it may reach up to 1.07. For this study, $S(0.2)/S(5.0)$ = 10.4 (see table 1) and M_v = 1.0. The factor I_E takes a value of 1.0, 1.3 or 1.5 for structures of the normal, high or post-disaster importance categories; I_E = 1.0 is therefore considered in this study. In the NBCC, R_d varies between 1.0 for the less ductile SFRSs (e.g. unreinforced masonry) and 5.0 for the most ductile ones (e.g. ductile steel moment frames). A value of 4.0 is assigned to R_d for buckling restrained braced frames, the same as specified for eccentrically braced steel frames. The factor R_o reflects the dependable overstrength present in the SFRS. This overstrength is limited in buckling restrained braced frames as the cross-sectional area of the brace cores at every level in a building can be tailored such that the brace factored axial resistances tightly match the forces from factored load effects. In addition, the steel core can be designed using the actual steel yield strength from coupon testing and the system typically does not develop much additional lateral resistance after brace yielding has been triggered. Hence, a low value of 1.2 is assigned to R_o which only accounts for the difference between factored and nominal resistances (= $1/\phi$ = 1.11) and minimum level of strain hardening anticipated in tension.

For office occupancy, the seismic weight only includes the structure dead load plus 25% of the roof snow load. For the seismic weights at floor levels, it is permitted to include only a portion (0.5 kPa) of the floor dead load assumed for interior partitions.

For short period structures, the value of V from Equation (1) need not exceed the lesser of 2/3 the value computed at a period of 0.2 s and the value at a period of 0.5 s. For braced steel frames with long periods, V must not be less than the value computed at a period of 2.0 s. The resulting base shear ratio V/W for BRBFs at the site under consideration is plotted in figure 3a as a function of the building period. For this site, the upper cut-off (0.181 W) controls up to a period of 0.8 s and the minimum lateral strength of 0.0831 W governs at periods longer than 2.0 s.

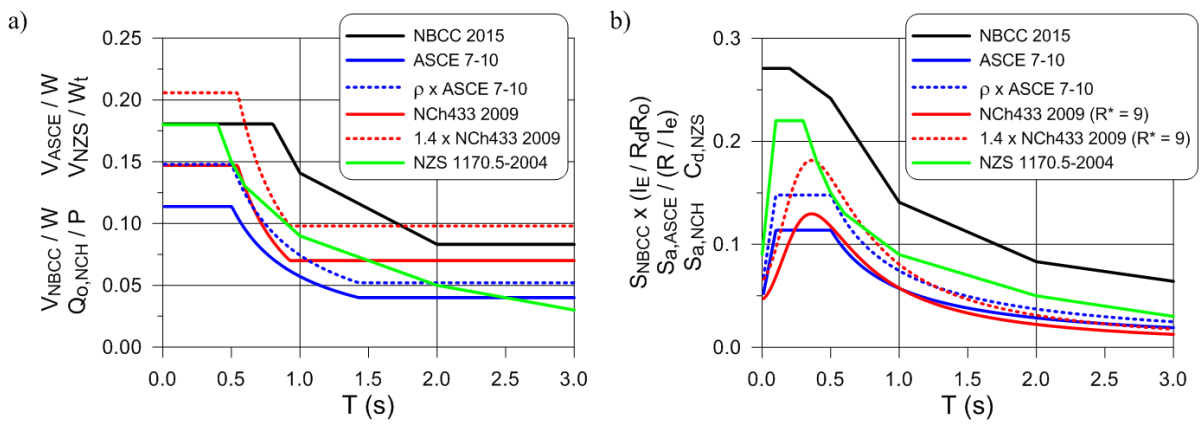

Figure 3: a) Base shear ratios (static analysis); and b) Design response spectra (dynamic analysis) for buckling restrained braced frames at the selected sites.

For regular structures such as the one studied herein, the equivalent static force procedure (ESFP) is permitted to be used if h_n does not exceed 60 m and T_a is less than or equal to 2.0 s. In that case, the lateral loads are distributed along the structure height as follows:

$$F_x = (V - F_t)\left(\frac{W_x h_x}{\sum W_i h_i}\right) \qquad \text{where: } F_t = 0 \quad \text{, for } T_a < 0.7\,s$$
$$= 0.07\,T_a V \le 0.25V \quad \text{, for } T_a \ge 0.7\,\text{s} \tag{3}$$

In this expression, F_t is a concentrated horizontal load applied at the building top to account for higher mode effects. In the NBCC, the dynamic multi-mode response spectrum analysis (RSA) has become the preferred analysis method for multi-storey structures. Dynamic analysis is performed using the spectrum $S(T)$ from Equation (2) and the resulting elastic base shear V_{ed} is used to determine the design base shear V_d:

$$V_d = V_{ed}\left(\frac{I_E}{R_d R_o}\right) \tag{4}$$

The response spectrum, as multiplied by I_E/R_dR_o, is plotted in figure 3b. To prevent excessively low lateral resistances resulting from dynamic analysis, as a minimum, the base shear V_d must be taken equal to 0.8 V for regular structures or V for SFRSs with structural irregularities. Design forces and displacements are then obtained by multiplying the analysis results by the ratio V_d/V_{ed}.

For displacements, the upper limit 0.05 h_n on the structure period need not be applied when determining the base shear force V. Hence, when EFSP is used, lateral displacements are determined with a new force V calculated with the period T_1. Similarly, in RSA, the minimum value of V_d is based on V obtained with T_1. At every level x, storey drifts from analysis under reduced seismic loads, Δ_{xe}, are amplified to obtain design storey drifts Δ_x including inelastic response:

$$\Delta_x = \Delta_{xe}\left(\frac{R_d R_o}{I_E}\right) \tag{5}$$

For buildings of the normal importance category, the design storey drifts Δ_x obtained from the displacements δ_x must not exceed the value of 0.025 h_{sx}, where h_{sx} is the storey height at level x. In the NBCC, gravity dead (D) and live (L) load effects are combined to earthquake effects (E) according to: 1.0 D + 0.5 L + 1.0 E.

3.2 Global stability requirements

In the CSA S16 standard, notional loads N_x must be applied in addition to wind and earthquake lateral loads. At every level, the load N_x is equal to 0.005 times the gravity load contributed by that level. The resulting total lateral load effects are then multiplied by the factor U_2 to account for P-delta effects:

$$U_{2,x} = 1 + \frac{\sum C_{fx}\,\Delta_x}{R_o V_x\, h_{sx}} \tag{6}$$

In this equation, ΣC_{fx} is the sum of the axial compression loads in all building columns and V_x is the storey shear due to seismic plus notional loads. The second term on the right of Equation (6) thus corresponds to the ratio of the overturning moment induced by the gravity loads acting on the laterally deformed storey reaching the anticipated peak inelastic drift to the

frame overturning moment capacity based on the expected frame lateral resistance R_oV_x. Hence, frames designed using $U_{2,x}V_x$ can resist V_x from lateral loads in addition to maximum anticipated P-Δ effects. CSA S16 also specifies an upper limit on U_2 to prevent excessive second-order moments relative to lateral strength from causing progressive drifting of the frame in the inelastic range that may eventually lead to collapse. If the limit is exceeded, the SFRS must be stiffened to reduce Δ as all other parameters in the expression for U_2 (ΣC_f, R_o, V, h_s,) are generally fixed and cannot be modified. Alternatively, the storey shear resistance can be increased to meet $U_2 \leq 1.4$. It is noted that notional loads and P-delta effects are only considered for the design of the braces as beams, columns and connections are designed for lateral loads associated to the probable brace resistances. Displacements and drifts need not be amplified either for P-delta effects.

3.3 BRBF design and detailing

In the CSA S16 standard, the bracing members are designed to have a factored axial resistance in tension and compression, $T_r = C_r = \phi A_{sc} F_{ysc}$, where $\phi = 0.9$, and A_{sc} and F_{ysc} are respectively cross-sectional area and yield strength of the brace steel core, equal to or larger than gravity plus lateral load effects. In the calculation of T_r and C_r, it is permitted to use F_{ysc} obtained from coupon tests, which minimizes brace overstrength. Once braces are designed and detailed, beams, columns and connections are sized to resist gravity loads plus lateral loads that develop when the braces reach their probable tensile and compressive resistances:

$$\begin{aligned} T_{ysc} &= \omega A_{sc} R_y F_{ysc} \\ C_{ysc} &= \beta \omega A_{sc} R_y F_{ysc} \end{aligned} \tag{7}$$

In this expression, R_y is the ratio between the probable and nominal core yield strengths (can be taken equal to 1.0 when the core is designed using coupon test data), ω is a strain hardening adjustment factor on tensile resistance and β is a factor that accounts for friction and Poisson's effect on compressive resistance. Both factors must be determined from qualification testing at 2.0 times the storey drift Δ_x.

In CSA S16, beams and columns must satisfy class 2 (compact) section requirements. The columns must also be designed as beam-columns considering a minimum flexural demand in the plane of the frame equal to 0.2 times the plastic moment capacity of the columns. Detail on seismic design provisions in Canada can be found in [11].

4 SEISMIC PROVISIONS IN THE U.S.

4.1 Seismic loads and analysis

In ASCE 7-10, the design base shear, V, is given by:

$$
\begin{aligned}
V = C_s W \quad \text{with: } C_s &= \frac{S_{DS}}{\left(R/I_e\right)} && , for\ \ T \le T_s \\
&= \frac{S_{D1}}{T\ \left(R/I_e\right)} && , for\ \ T_s < T \le T_L \\
&= \frac{S_{D1} T_L}{T^2\ \left(R/I_e\right)} && , for\ \ T > T_L \\
&\ge 0.044\, S_{DS} \\
&\ge 0.01 \\
&\ge \frac{0.5\, S_1}{\left(R/I_e\right)} && ,if\ \ S_1 \ge 0.6
\end{aligned}
\tag{8}
$$

In these expressions, C_s is the seismic coefficient and W is the seismic weight. As shown, three period regions are used to define C_s: plateau for short periods, $1/T$ decay for intermediate period range and $1/T^2$ decay for long periods. Three minimum values are also specified that may govern in the intermediate or long period range. S_{DS} and S_{D1} are the design short-period and one-second spectral acceleration values. They are respectively equal to 2/3 times the modified MCE_R spectral values S_{MS} and S_{M1} defined earlier (see table 1). For the Seattle site studied, S_{DS} = 0.91 and S_{D1} = 0.458. In ASCE 7-10, the force modification factor R varies from 3.0 for steel SFRSs not specifically designed and detailed for ductile seismic response to 8.0 for the most ductile SFRSs. BRBFs classify for an R of 8.0. The importance factor can take values from 1.0 to 1.5 depending on the risk category. In this study, we consider I_e = 1.0.

The fundamental period of the structure, T, can be obtained from dynamic analysis but the so-computed period cannot exceed C_uT_a, where $T_a = 0.0731h_n^{0.75}$ and C_u varies from 1.4 in active seismic regions ($S_{D1} \geq 0.3$) to 1.7 for low-seismic regions ($S_{D1} \leq 0.1$). For the site studied herein, C_u takes a value of 1.4 and C_uT_a is plotted in figure 2. As shown, this upper limit compares well with the maximum period permitted in Canada. As in the NBCC, the upper period limit in ASCE 7-10 need not be applied when determining displacements or drifts. The period T_s is equal to the ratio S_{D1}/S_D, which gives 0.5 s for the case study in this article. The seismic weight is same as defined in the NBCC except that the contribution of the roof snow load is reduced to 20% and needs only be included when the roof snow load exceeds 1.44 kPa.

The variation of the base shear ratio V/W as a function of the period is plotted in figure 3a for BRBFs located at the site considered in the study. Up to a period of 0.5 s (T_s), the design is controlled by the S_{DS} parameter, and the specified minimum lateral strength corresponding to $0.044\ S_{DS} = 0.04W$ applies for periods longer than 1.43 s. This means that the $1/T^2$ decay region does not apply in that particular case (starts at T_L = 6 s). In the figure, base shears from ASCE 7-10 are lower than NBCC values for all periods, with more pronounced differences at longer periods where minimum lateral strength provisions govern. The difference can be mainly attributed to the 2/3 factor applied to the MCE_R spectral ordinates to obtain the design spectrum and the higher R factor for BRBFs (R = 8.0 in ASCE 7-10 *vs* R_dR_o = 4.8 in NBCC 2015). The difference is less for short period structures because of the seismic force cut-off permitted in Canada in that period range. As discussed later, ASCE 7-10 requires that a redundancy factor ρ = 1.3 be applied to seismic loads for the design of the braced frames of the building studied here. The effect of this factor is illustrated in figure 3a. Although it directly impacts SFRS designs, the ρ factor serves other purposes and can take a value of 1.0 for other structures; hence, its effects should be considered with thoughtfulness when comparing basic design seismic demands.

In ASCE 7-10, the ESFP can be adopted for the analysis of regular structures if the height does not exceed 48.8 m and the period is less than 3.5 T_s (= 1.75 s in this study). In that case, the distribution of the lateral seismic forces along the frame height is given by:

$$F_x = V\left(\frac{W_x h_x^k}{\sum W_i h_i^k}\right) \quad \text{where: } k = 1.0 \quad \text{, for } T < 0.5s$$
$$= 0.75 + 0.5T \leq 2.5 \quad \text{, for } T \geq 0.5\text{s} \tag{9}$$

The multimode response spectrum analysis is generally used when static analysis is not permitted. In the analysis, the spectrum is taken equal to $S_a(T)$ divided by (R/I_e), where $S_a(T)$ is given by:

$$\begin{aligned} S_a &= S_{DS}\left(0.4 + 0.6\frac{T}{T_o}\right) && \text{,for } T < T_o \\ S_a &= S_{DS} && \text{,for } T_o \leq T \leq T_s \\ S_a &= \frac{S_{D1}}{T} && \text{,for } T_s < T \leq T_L \\ S_a &= \frac{S_{D1}T_L}{T^2} && \text{,for } T > T_L \end{aligned} \tag{10}$$

In these expressions, the period $T_o = 0.2T_s$. The reduced spectrum $S_a/(R/I_e)$ is plotted in figure 3b. It corresponds to C_s except that a ramp function starting at $0.4S_{DS}$ is included for periods shorter than T_o and no minimum values apply at intermediate and long periods. For strength verification, however, the base shear from response spectrum analysis, V_t, must be compared to the force V from Equation (8): if $V_t < 0.85V$, the force demand from analysis must be multiplied by $0.85V/V_t$. No such adjustment is needed for displacements and drifts, except when V is governed by the term $0.5S_1/(R/I_e)$ in Equation (8), which is not the case in this study ($S_1 = 0.528 < 0.6$). In figure 3b, the ASCE 7-10 design spectrum is lower than the NBCC one, again mainly because it is based on the reduced design spectral ordinates S_{DS} and S_{D1} and the larger R factor.

To account for inelastic effects, displacements from analysis are multiplied by C_d/I_e, where C_d is the deflection amplification factor. In ASCE 7-10, a C_d value of 5.0 is specified for BRBFs and the resulting design storey drifts, Δ, are limited to 0.02 h_s for multi-storey steel frames. Effects from dead and live loads are combined to earthquake effects in accordance with:

$$\begin{aligned} &\left(1.2 + 0.2S_{DS}\right)D + L + \rho Q_E \\ &\left(0.9 - 0.2S_{DS}\right)D + \rho Q_E \end{aligned} \tag{11}$$

In these load combinations, the second term affecting the dead load represents vertical ground motion effects, the ρ factor is the redundancy factor, and Q_E include horizontal seismic load effects. In the first load combination, 0.5 L may be used instead of L when the live load is less than 4.8 kPa, and the specified roof live load (L_r) of 1.0 kPa need not be considered as acting concomitantly with earthquake loading. Both relaxations on live load effects apply to the building studied herein. In Equation (11), the second load combination is used when gravity loads counteract seismic effects.

The redundancy factor is aimed at preventing structural collapse resulting from failure of an individual component or connection in the SFRS. Its value depends on the SFRS type and the

Seismic Design Category assigned to the structure. In ASCE 7, the Seismic Design Category is used to define system limitations (e.g., height limits, prohibited irregularity types, etc.) or trigger special design requirements. The Seismic Design Category depends on the risk category and the design spectral ordinates S_{DS} and S_{D1}. For the structure examined herein, a Seismic Design Category D applies. For braced frames of this Seismic Design Category, $\rho = 1.3$ unless the SFRS consists of a minimum of two braced bays along each perimeter wall or removal of a single brace does not reduce the storey shear strength by more than 33% or create extreme torsional irregularity. The first condition is not met along the E-W exterior column lines and failure of a brace or its connections may result in a deformed configuration exceeding the limit for extreme torsional irregularity, as the moment frames in the orthogonal direction are not expected to contribute significantly to limiting in-plane building rotations. A redundancy factor of 1.3 is therefore considered for the building example. It must be noted that modifying the structure to reduce this factor to 1.0 say, for instance, by adding braced bays on the building perimeter, may lead to less cost-effective solutions compared to just designing for seismic loads increased by 30%.

4.2 Global stability requirements

In ASCE 7-10, global stability effects are accounted for by means of the stability coefficient, θ_x, computed at every storey of the structure:

$$\theta_x = \frac{P_x \, \Delta_x \, I_e}{V_x \, h_{sx} \, C_d} \tag{12}$$

In this equation, P_x is the total axial compression loads in the columns, Δ_x is the design (first-order) storey drift under seismic loads inducing the seismic storey shear V_x, and h_{sx} is the storey height. The stability coefficient then corresponds to the ratio between second-order ($P \times \Delta$) moments to the primary moments ($V \times h_s$). The load P_x is determined using Equation (11) except that the load factors need not exceed 1.0. For the frame examined in this study, P_x is therefore obtained from the load combination D + 0.5L, as in the NBCC. In ASCE 7-10, the stability coefficient must not exceed θ_{max} given by:

$$\theta_{max} = \frac{0.5}{\beta \, C_d} \leq 0.25 \tag{13}$$

where β is the ratio between storey shear demand and capacity. For BRBFs, lateral overstrength is generally small and β can be conservatively taken equal to 1.0 and $\theta_{max} = 0.10$. This limit on θ is aimed at preventing global frame instability in the inelastic range, similar to the upper limit on the factor U_2 in Canadian codes. When comparing the provisions in both countries, the coefficient θ is similar to the second term of Equation (6) for the U_2 factor except that θ is calculated using elastic storey drifts ($\Delta_{xe} = \Delta_x/C_d$) and storey shears V_x, rather than anticipated drifts including inelastic effects ($\Delta_x = R_d R_o \Delta_{xe}$) and expected storey shear resistance $R_o V_x$. The stability coefficient in the U.S. is also determined using consistent Δ_x and V_x values and thus reflects the lateral stiffness of the structure. In the Canadian provisions, the second term in the expression for U_2 aims at reflecting the ratio between the P-delta force demand and the SFRS lateral strength. In spite of these differences, the upper limits specified in both codes dictate comparable, although less severe in the U.S., minimum elastic frame stiffness requirements for BRBFs when one considers that β in Equation (13) essentially corresponds to $1/R_o$, ΣC_{fx} is the same as P_x, and using the specified R_d, R_o and C_d values:

$$U_{2,x} \le 1.4 \;\Rightarrow\; \frac{P_x\left(R_d R_o\,\Delta_{xe}\right)}{\left(R_o V_x\right)h_{sx}} \le 0.4 \;\Rightarrow\; \frac{V_x}{\Delta_{xe}} \ge \frac{2.5\,P_x R_d}{h_{sx}} = 10\,\frac{P_x}{h_{sx}}$$
$$\theta_x \le \frac{0.5}{\beta C_d} \;\Rightarrow\; \frac{P_x\left(C_d\,\Delta_{xe}\right)}{V_x\,h_{sx}\,C_d} \le \frac{0.5\,R_o}{C_d} \;\Rightarrow\; \frac{V_x}{\Delta_{xe}} \ge \frac{2\,P_x C_d}{R_o\,h_{sx}} = 8.3\,\frac{P_x}{h_{sx}} \tag{14}$$

In ASCE 7-10, P-delta effects are permitted to be ignored when θ is less than 0.10. Otherwise, they must be determined by means of a rational (nonlinear) analysis or taken into account by amplifying seismic member forces and displacements by $1/(1-\theta_x)$. For BRBFs, this P-delta amplification never applies as the stability coefficient is limited to θ_{max} = 0.10. However, the AISC 360-10 Specification for Structural Steel Buildings [12] also must be satisfied for design of BRBFs, and as part of the Direct Analysis Method of Design, it requires consideration of stability, including initial geometric imperfections, second-order effects and stiffness reduction due to inelasticity. Initial geometric imperfections are considered through direct modeling or notional loads, but in many buildings, such as the scenario for the present study, initial geometric imperfections only need to be considered in gravity-only load combinations. Global P-delta effects may be considered directly in the analysis, or through an approximate second-order analysis method such as the B_2 multiplier given by:

$$B_{2,x} = \frac{1}{1-\dfrac{P_{storey}\,\Delta_x}{R_M\,V_x\,h_{sx}}} \tag{15}$$

In this equation, P_{storey} is the total vertical load supported by the storey, Δ_x is first-order storey drift produced by the storey shear V_x, and R_M is a coefficient to account for the influence of member-level P-delta effects on storey-level P-delta effects (R_M = 1.0 for braced frames). This B_2 multiplier is very similar to 1/(1-θ), but in AISC 360-10, there is no threshold below which P-delta effects may be ignored. In addition, the B_2 multiplier will produce a more severe amplification than 1/(1-θ) since P_{storey} is calculated based on the gravity load from Equation (11), $1.2D + 0.2S_{DS}D + 0.5L$, which is larger than the $D + 0.5L$ combination used to calculate θ. Also, storey stiffness (V_x/Δ_x) in Equation (15) is reduced by using $0.8E$ in the lateral system model, where E is the modulus of elasticity, to approximate the effects of inelasticity. Although this reduced stiffness is used in calculating storey stiffness for B_2, AISC 341-10 states that design storey drifts and drift limits are specified by the applicable building code (ASCE 7-10), so the drift check is conducted with unreduced stiffness. Designs with and without application of B_2 are evaluated in the present study since second-order amplification at the design load level, when the structure is still primarily elastic, may commonly be deemed inconsequential for seismic loading, when the structure is expected to develop large inelastic cyclic drifts. Integrated application of AISC 341, AISC 360 and ASCE 7 requirements is discussed in [13].

4.3 BRBF design and detailing

Seismic design requirements for BRBFs in AISC Seismic Provisions are nearly the same as the CSA S16 provisions described in Section 3.3. Detailed design examples can be found in [14]. Differences between CSA and AISC requirements that may impact on member design can be summarized as follows:

- Braces are designed to resist earthquake induced axial loads only, without consideration of gravity induced loads.

- Beams and columns must satisfy the requirements for highly ductile members, which are different from the section class requirements specified in the CSA standard.
- Flexural demand on the columns need not be considered; columns are therefore designed for axial load only.

5 SEISMIC DESIGN PROVISIONS IN CHILE

5.1 Seismic loads and analysis methods

In NCh433, the equivalent static procedure can be used for buildings up to 5 storeys and a height h_n not exceeding 20 m. The procedure can be extended to regular structures up to 15 storeys if special conditions are satisfied. For other structures, multimode response spectrum analysis is required. When static analysis is used, the minimum seismic lateral load, Q_o, is given by:

$$Q_o = C\,I\,P \quad \text{where: } C = \frac{2.75\,S\,A_0}{g\,R}\left(\frac{T'}{T^*}\right)^n \tag{16}$$

, where C is the seismic coefficient, I is the importance factor, P is the seismic weight, A_o is the maximum effective ground acceleration, g is the acceleration due to gravity, R is the response modification factor, T^* is the structure fundamental period, and parameters S, T' and n are used to characterize the effects of local soil conditions on seismic demand. The importance factor varies from 0.6 for occupancy category I representing low risk to occupants to 1.2 for occupancy category IV that includes important or critical facilities. The value I = 1.0 applicable to office buildings is used in this study. The seismic weight P includes the structure dead load plus the full partition dead load and a portion (25%) of the occupancy floor live load (0.25 x 2.4 kPa = 0.6 kPa)..

For the site considered in this study, A_o/g = 0.40 (see Section 2.2). No value of R has yet been adopted in the Chilean code for BRBFs. For eccentrically braced steel frames, R = 6.0 is specified in NCh433. In view of the fact that an R of 8.0 is attributed to both eccentrically braced frames and buckling restrained braced frames in ASCE 7-10, R = 6.0 is tentatively assigned to BRBFs in this study. In NCh433, the period T^* is obtained from analysis, without upper limits. In design, it is common to use $T^* = 0.1N$, where N is the number of storeys. For a storey height of 4.0 m, this expression gives the same period as the period $T_a = 0.025\,h_n$ specified in NBCC 2015 (lower bound in figure 2). This conservative period estimate accounts for the possible stiffening effect of non-structural elements. It also reflects the effect on frame stiffness of the stringent seismic drift limit specified in NCh433, as is discussed below. For site class C, the coefficient S is equal to 1.05, the period T' = 0.45 s and the exponent n = 1.40.

Similarly to the Canadian and U.S. provisions, a minimum value for C is specified in NCh433. This limit is equal to $SA_o/6g$, which gives C = 0.070 for the building site considered herein (A_o = 0.40 g and S = 1.05 for site class C). For BRBFs, this floor value would control the design of BRBFs having a period longer than 0.93 s, as illustrated in figure 3a. For short period structures, NCh433 also proposes a maximum C value that need not be exceeded in design. That maximum value is a percentage of SA_o/g that depends on the R factor. For BRBFs with R = 6.0, this upper limit is set to 0.35 SA_o/g, which corresponds to C = 0.147 for the site conditions selected for this study. For these conditions, figure 3a shows that the cap on Q_o governs for periods up to 0.54 s, which leaves a very limited period range over which Q_o varies with the period. The figure shows that design seismic loads for BRBFs in Chile would lie between those specified in the ASCE 7 and NBCC documents. As described later, a load factor of 1.4 is applied

to seismic loads for design in Chile. When considering this factor, NBCC and Chilean seismic loading requirements become closer to each other.

In the NCh433 ESFP, the vertical distribution of the load Q_o is given by:

$$F_x = \left(\frac{A_x P_x}{\sum A_i P_i} \right) Q_o \quad \text{where:} \quad A_x = \sqrt{1 - \frac{h_{x-1}}{h_n}} - \sqrt{1 - \frac{h_x}{h_n}} \tag{17}$$

The response spectrum analysis is performed using the response spectrum S_a given by:

$$S_a = \frac{S A_o \alpha}{\left(R^* / I \right)} \tag{18}$$

where α is an amplification factor for the maximum effective acceleration that defines the shape of the response spectrum for a given site class and R^* is a reduction factor. These two factors are obtained from:

$$\alpha = \frac{1 + 4.5 \left(T/T_o \right)^p}{1 + \left(T/T_o \right)^3} \tag{19}$$

$$R^* = 1 + \frac{T^*}{0.10 T_o + T^* / R_o} \tag{20}$$

In Equation (19), T_o and p are parameters that depend on the soil type, similar to T' and n in the static force procedure, except that maximum amplification occurs at the period T_o. For site class C, T_o = 0.40 s and p = 1.60. Variation of α for this soil condition is illustrated in figure 4a. Equation (20) applies for frame structures including braced steel frames. In this equation, T^* and R_o are respectively the computed fundamental period and the response modification factor of the structure. For EBFs, the value of R_o in NCh433 is equal to 10. For consistency, the same value is selected herein for BRBFs. As shown in figure 4b, Equation (20) gives small R^* values for short period structures, aiming at preventing excessive ductility demand in the period range where the equal energy principle applies. For longer periods, the R^* factor gradually increases to values close to R_o for the range of periods typically encountered in multi-storey buildings. In Equation (20), R^* also depends on the soil period T_o. The influence of the structure periods on seismic loads is therefore two-fold as the periods in the contributing modes affect the associated spectral accelerations ($A_o\alpha$) and the R^* factor depends on the structure period in its fundamental mode.

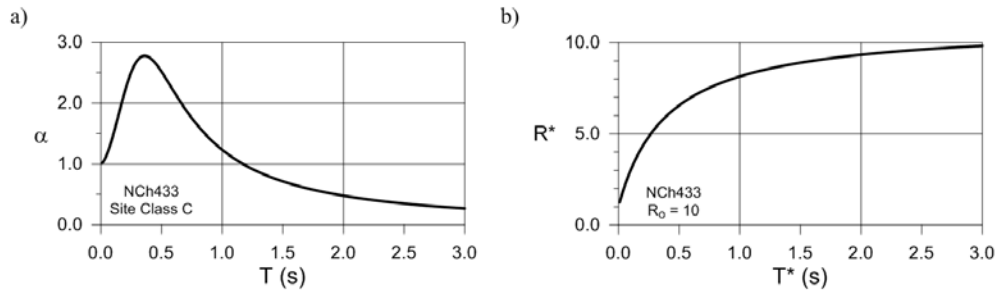

Figure 4: NCh433 design spectrum parameters for the selected site: a) Maximum effective acceleration amplification factor; and b) Reduction factor.

The design spectrum is illustrated in figure 3b for R^* = 9.0, a value close to the one found for the BRBFs studied here, as will be described in Section 6.3. The spectrum compares well

with the one specified in ASCE 7-10. When amplified by the load factor of 1.4, it generally lies between the spectra used in Canada and the U.S. If the base shear from response spectrum analysis, referred to herein as Q_d, is less than the minimum value specified in static analysis ($Q_{o,min} = SA_oIP/6g$), all forces and deformations from analysis must be multiplied by the ratio $Q_{o,min}/Q_d$. As discussed earlier, the minimum value for the building example is equal to 0.070 P. For the spectrum shown in figure 3b, that minimum base shear is likely to govern for structures with a fundamental period longer than approximately 0.9 s. Similarly, the upper limit on base shear specified for the static force procedure, $C_{max}IP$, is permitted to be applied to reduce forces and displacements from response spectrum analysis in short-period structures.

A drift limit of 0.002 h_s is specified for seismic design in the Chilean code. The limit is verified using elastic storey drifts Δ_{xe} directly obtained from the seismic analysis, without amplification for inelastic behaviour and the 1.4 load factor (see below). This stringent restriction is aimed at limiting damage from recurrent strong earthquakes occurring in Chile and therefore can be assimilated to a serviceability limit state. Drift limits in Canada and the U.S. are associated to structural damage and stability of buildings responding in their nonlinear range. Hence, they represent ultimate limit states. They can be compared to the Chilean limit by dividing them by the amplification for nonlinear deformations (R_dR_o = 4.8 in NBCC and C_d = 5.0 in ASCE 7) and the Chilean load factor of 1.4. With these corrections, the NCh433 limit on seismic drifts is respectively 1.9 and 1.4 times more stringent than the drift limits in Canada and the U.S.

For steel structures, NCh433 allows the use of both limit states and allowable stress design approaches. When the former is adopted, a load factor of 1.4 must be applied to seismic effects and the combined effects from gravity and seismic loads are obtained using the following load combinations: 1.2 D + 1.0 L + 1.4 E and 0.9 D + 1.4 E. As in ASCE 7-10, roof live load need not be considered in the first load combination and the second combination applies when gravity load effects are opposed to those from seismic loads. Comparison between building codes in Chile and the U.S. is also presented in [15].

5.2 Global stability requirements

NCh433 does not include any specific stability design requirements to prevent structural collapse by instability under earthquakes. However, application of the severe code drift limit imposes limitations on P-delta overturning moments, thus indirectly controls stability effects under seismic loading.

5.3 BRBF design and detailing

NCh433 refers to the 2005 edition of the AISC Seismic Provisions for the seismic design and detailing of steel SFRSs. To facilitate the comparison between the different codes, however, the 2010 edition of the AISC Seismic Provisions as presented in Section 4.3 will be used with NCh433 for the design of the BRBF located in Valparaiso.

6 SEISMIC DESIGN PROVISIONS IN NEW ZEALAND

6.1 Seismic loads and analysis methods

The BRBF draft design guide in New Zealand [7] is based on modifying the general design procedure for concentrically braced frames with braces effective in tension and compression, published in HERA Report R4-76 [16], to account for the BRBs being equally strong in tension and compression. The determination of seismic actions is in accordance with NZS 1170.5 [6], which is part of the joint Australia / New Zealand Loadings Standard set [17]. In NZS1170.5,

an equivalent static force procedure (Equivalent Static Method) can be used for regular structures with the largest translational period equal to or shorter than 2 s. As shown below, the design discussed herein just complies. For other structures, response spectrum analysis (RSA) must be used, with sufficient modes included to ensure that at least 90% of the total mass of the structure is participating in the direction under consideration. Three-dimensional RSA is also required for all structures which have a principally torsional first mode of response. Note that the design described herein will not be torsionally sensitive and is taken as vertically regular. The elastic site hazard spectrum for horizontal loading is given by:

$$C(T) = C_{\mathrm{h}}(T)\ Z\ R_{\mathrm{u}}\ N(T,D) \tag{21}$$

In this equation, $C_h(T)$ is the spectral shape factor for the given soil type, specified for periods of 0 to 4.5 s and taken as constant after 4.5 s. Z is the seismic hazard factor representing the uniform hazard spectrum PGA associated with a 500 year return period. As indicated, Z is taken equal to 0.4 for the site chosen in this study. R is the return period factor. For ultimate limit states design, R_u = 1.0 for 10% probability of exceedance in 50 years and R_u = 1.8 for 2% probability of exceedance in 50 years. $N(T,D)$ is the near fault factor. For this design, it has been set at 1.0 for consistency with the other standards, however for the site chosen it would have a value > 1.0 in practice.

For the equivalent static force procedure (EFSP), the horizontal seismic base shear $V = C_d(T_1)\ W_t$. For class B rock site, the horizontal design action coefficient, $C_d(T_1)$, is given by:

$$\begin{aligned} C_{\mathrm{d}}(T_1) &= \frac{C(T_1)S_{\mathrm{p}}}{k_{\mu}} \\ &\geq \left(\frac{Z}{20} + 0.02\right) R_{\mu}\ ,\ \text{but not less than } 0.03R_{\mu} \end{aligned} \tag{22}$$

All symbols are as described above, except for k_μ which is equal to μ for $T_1 \geq 0.7$ s, where μ is the structural ductility factor, which is elaborated on below. The Rayleigh method is specified for determining the fundamental period, T_1. Empirical formulae may be used to initiate the design. This generally give periods lower than obtained from the structural model using the Rayleigh method. The Rayleigh method is the specified method for final design, but no upper limit is specified on the Rayleigh method derived design period in relation to that from the empirical formulae. The total seismic weight of the structure, W_t, is taken as the structure permanent load, including 1.0 kPa for interior partitions, plus 30% of the live load. For the structure studied, an area reduction factor equal to 0.5 also applied to the floor live load when determining the seismic weights. In the NZS 1170.5 equivalent static method procedure, the vertical distribution of seismic load on each floor, F_x is given by:

$$F_{\mathrm{x}} = F_{\mathrm{t}} + 0.92\,V\left(\frac{W_{\mathrm{x}}h_{\mathrm{x}}}{\sum W_{\mathrm{i}}h_{\mathrm{i}}}\right)\quad \text{, where } F_{\mathrm{t}} = 0.08V \text{ at top and zero elsewhere} \tag{23}$$

The response spectrum analysis (RSA) is performed using the response spectrum given by:

$$C_{\mathrm{d}}(T) = \frac{C(T_1)S_{\mathrm{p}}}{k_{\mu}} \tag{24}$$

For structures that are regular, the results from RSA must be scaled so that the base shear is not less than 80% of the base shear from the ESFP for determination of seismic design actions and displacements. For structures that are irregular, the scaling must be to 100% of the base shear from ESFP.

NZS 1170.5 implements the trade-off of strength versus controlled damage through the structural ductility factor μ. For structures with a first mode period $T_1 \geq 0.7$ s in soil classes A through D, the elastic site hazard spectrum value at T_1 is divided by μ and multiplied by S_p to obtain the design action coefficient for ESFP. S_p is termed the structural performance factor and is a structural overstrength factor, analogous to the Canadian R_o. It is, however, linked to the structural ductility factor through the following equation, where the structural ductility used for determining S_p is that relating to the level of detailing used in that system rather than the value used to determine the seismic design actions:

$$\begin{aligned} S_p &= 0.7 && \text{for } \mu > 2 \\ S_p &= 1.3 - 0.3\mu && \text{for } 1.0 \leq \mu \leq 2 \end{aligned} \tag{25}$$

The NZS 3404 steel structures standard [18] specifies 4 categories of ductility demand, through the structural ductility factor, these being:

Category 1: Fully ductile systems, $\mu > 3$
Category 2: Limited ductile systems, $3.0 \geq \mu > 1.25$
Category 3: Nominally ductile systems, $\mu = 1.25$
Category 4: Elastic systems, $\mu = 1.0$

The category of structural ductility demand is a designer's choice, although the commentary to the standard provides some guidance and the detailed seismic design procedures, such as [6, 16] provide more guidance. Following the 2010/2011 Christchurch earthquake series, the upper limit for structural ductility factor is typically $\mu = 3.0$ and that value has been used in this design. In Equations (22) and (24), the combined effect of μ and S_p (through k_μ/S_p) is a reduction of 4.3 for the seismic loads, which compares well with $R_dR_o = 4.8$ in Canada. It is lower than $R = 8.0$ used in the U.S and $R = 6.0$ adopted for Chile.

Once the category of structure is set through the choice of μ, NZS 3404 then specifies the type of design procedure required (e.g., capacity design or not), the level of detailing required for all components of the seismic resisting system and the upper limit actions required to be considered when capacity design is used. For the structure studied herein, dead (D) and live (L) gravity load effects are combined with seismic effects (E) according to: 1.0 D + 0.4 L + 1.0 E. As mentioned, floor live load effects were also multiplied by the area reduction factor of 0.5.

The static base shear V is compared to the values from the other three codes in figure 3a. It is same as in the NBCC for periods up to 0.5 s and then reduces without a minimum floor value to become the lowest among all codes for periods longer than approximately 2.5 s. In figure 3b, the design spectrum for RSA being defined for a 10% probability of exceedance is close to those specified in the U.S. and Chile and is lower than the one prescribed in Canada.

For ultimate limit states, the drift limit in NZS 1170.5 is 2.5% of the storey height, same as in Canada. For structures that are designed using the capacity design method to suppress soft storey formation, the ultimate limit state storey drifts are the elastic deflections from analysis amplified by the structural ductility factor, μ. For regular structures designed by the ESFP, these deflections may be scaled by a reduction factor k_d, which is 0.85 for structures with 6 or more storeys. The drifts must be further multiplied by the drift modification factor k_{dm} to account for

uneven distribution of inelastic deflections over the frame height. For structures taller than 30 m, as is the case for the prototype structure here, k_{dm} = 1.5. For the structure studied herein, considering the inelastic amplification ($\mu k_d k_{dm}$ = 3.83) and the load factor of 1.4 in Chile, the 2.5% h_s limit in New Zealand is 2.4 times higher than the limit in Chile. In New Zealand, a drift limit of 0.0033 is also specified for damage control at the serviceability limit state. The Chilean requirement is 2.2 times more severe than this New Zealand requirement for the level of earthquake associated with serviceability limit state response.

6.2 Global stability requirements

P-delta effects are covered by NZS 1170.5 in three steps. The first step is to determine whether explicit design for P-delta effects is required. There are generic exclusions for low rise or very stiff structures, but the general check is via the stability coefficient, θ_x, given by:

$$\theta_x = \frac{W_x \Delta_x}{V_x h_{sx}} \tag{26}$$

In this expression, W_x is the cumulative seismic weight resisted by the storey under consideration. This differs from the approach used in Equation (6) for Canada and in Equations (12) or (15) for the U.S. where concomitant factored gravity loads instead of seismic weights are used to determine P-delta effects. The storey drift Δ_x includes inelasticity effects and is calculated as described in section 6.1, i.e., by multiplying displacements from analysis by μ and the scale factor k_d. The factor k_{dm} need not be applied for P-delta effects. V_x in this expression is the storey shear strength, which may be taken equal to the design storey shear force. This stability coefficient is the inverse of the inelastic critical buckling load factor. If the value calculated for every storey is less than 0.1, this means the inelastic critical buckling load factor > 10 and P-delta effects can be ignored. If, for any storey, the value ≥ 0.1, then P-delta effects must be considered.

There are two options for this. The procedurally much more simple approximate method A is a multiplier on the structural actions determined from the ESFP or RSA method:

$$\frac{k_p W_t + V}{V} \quad \text{,where } k_p = 0.015 + 0.0075(\mu - 1) \quad \text{with the limits: } 0.015 < k_p < 0.03 \tag{27}$$

This multiplier varies from a minimum value of 1.15 up to 1.5, with typical values of around 1.3 to 1.35. For braced frame systems especially, this method is very conservative. The more complex method B is given in NZS1170.5 and involves the following steps:

- Step 1: Analyse the structure to the ESFP or RSA neglecting P-delta effects.
- Step 2: Obtain the predicted inelastic displacements at the centre of mass at each level by the scaling method described in section 6.1, i.e. by multiplying the displacements from analysis by $k_d\mu$.
- Step 3: Calculate the lateral restoring forces necessary for equilibrium when the seismic weights at each level are displaced by the inelastic deflection profile given from step 2 and calculate the additional elastic displacements generated by these forces.

- Step 4: Calculate the scaling factor β for these forces and displacements, which is specified in NZS 1170.5 as a function of the structural ductility factor, the structure period, and the soil class. The β factor is given in Equation (28) with the *K* factor being plotted in figure 5. For μ = 3, β varies from 1.71 for shorter buildings to 1.0 for taller ones.

$$\beta = \frac{2\mu K}{3.5} > 1.0 \quad \text{for } \mu \le 3.5$$
$$\beta = 2.0K \quad \text{for } \mu > 3.5 \tag{28}$$

- Step 5: Multiply the lateral restoring forces from step 3 by the scaling factor β from step 5 and apply these through the centre of mass as additional lateral loads to those obtained from the ESFP or RSA analysis.
- Step 6: Multiply the additional displacements from step 3 by the scaling factor β from step 5 and apply these through the centre of mass as additional to the inelastic displacements from step 2.

Method B gives P-delta actions that are typically less than 50% of those from method A for braced frames. In the New Zealand design example given in this paper, P-delta actions are required to be considered for both the ESFP and the RSA methods. This is typical for frames designed to μ ≈ 3. When using method B, only the inelastic displacements from step 2 are multiplied by the drift modification factor when verifying compliance to the prescribed drift limit.

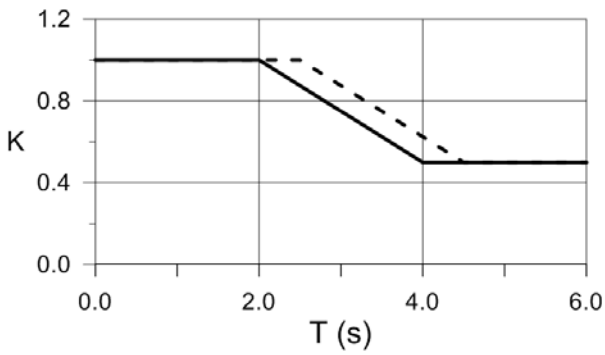

Figure 5: *K* factor used in P-delta analysis method B in NZS 1170.5.

6.3 BRBF design and detailing

The NZS 3404 standard] gives general requirements for the design of concentrically braced frame systems with braces effective in tension and compression and for tension-only braced systems. The comprehensive design and detailing provisions for CBF systems in general are given in sections 14 to 19 of HERA Report R4-76 [16]. BRB systems are not expressly covered in that document and a draft design and detailing guide has been produced for SCNZ that has been used to design the New Zealand example presented in this paper [7]. Some of the key points are:

- As with all fixed base seismic resisting systems analysed to NZS 3404, column bases must be considered to have a rotational stiffness, $k_\theta = 1.67EI/L$, where *E*, *I*, and *L* relate to the column at the level above the base.

- The brace yielding core is the primary seismic resisting system element and are designed to resist seismic actions using a strength reduction $\phi = 0.9$. As is done in the U.S. and Chile, the braces are not designed to carry any gravity loading.
- BRB connections and all other SFRS components are designed for the overstrength actions generated by the braces, subject to these not exceeding the upper limit actions specified in NZS 3404. For Category 1 systems with $\mu = 3$ as the frame in this paper, the upper limit actions are those determined from nominally ductile seismic response in conjunction with the appropriate design gravity loads.
- Similar to the practice in other countries, the beams must support the gravity loads ignoring the contribution of the braces. They must also resist the vertical out of balance forces generated by the difference between the overstrength capacities of the brace cores in compression and tension. The beams are designed to carry these loads simply supported; i.e., not relying on any rotational fixity of the collector beam to column connection.
- The strength and stiffness of the brace restraining jacket is determined using the provisions in [18]. These have a strength requirement, based on resisting 2.5% of the overstrength compression capacity of the brace core applied laterally at mid-span of the brace core length and transferring this back to the centre of bearing of the transition region at the end of the brace. They also have an elastic stiffness requirement.

7 SEISMIC DESIGN OF THE PROTOTYPE BUILDING

In this section, the seismic code provisions for the four countries are applied for the design of the buckling restrained braced frames of the 9-storey regular building structure described in Section 2. Lateral resistance along the E-W direction is provided by perimeter chevron braced frames. Two identical frames, one per wall, were used at all sites except for Chile where the structure included a total of four braced frames in the direction studied (figure 1a). The frames have a total height h_n = 37.182 m from the ground level. The columns extend into the basement level. In-plane torsion effects are ignored in the study and lateral loads including stability effects are assumed to be resisted equally by the braced frame(s) in each wall. . Other loads such as wind and snow loads are also ignored in the calculations. Among all four sites studied, it is only in Victoria that a portion of the roof snow load had to be included in the seismic weight; however, that contribution (0.27 kPa) was small compared to the roof dead load (4.0 kPa) and it was omitted to have a uniform comparison basis. Additional design data and assumptions are provided in section 7.1. Section 7.2 presents design solutions obtained when applying the code equivalent static force procedures. In Section 7.3, these designs are then refined using the dynamic (response spectrum) analysis method.

7.1 Design data

The buckling restrained braces of the structures are sized assuming that the yield stress of the core material is known from coupon tests: F_{ysc} = 290 MPa. Hence, R_y equal to 1.0 is used to determine the probable resistances or adjusted strengths. In these calculations, the tension and compression strength adjustment factors, ω and β, are taken equal to 1.4 and 1.1, respectively. In the analyses, the bracing members are assumed to have an equivalent cross-sectional area over the brace workpoint length equal to 1.5 times the core yielding cross-section area A_{sc}. This ratio is typical of braces detailed for high axial stiffness, when drift limits are expected to control the frame design. Beams and columns are assumed to be fabricated from ASTM A992 I-shaped members with a steel yield strength of 345 MPa. It is noted that the same steel grades were considered in all countries, even if they may not be actually available, so that differences in design mainly reflect dissimilarities between code provisions. Column splices

are indicated in figure 1. Beams are non-composite and the frames are analysed and designed assuming that the beam-to-column connections are pinned. The beams are assumed to be vertically braced by the BRB members at mid-length and laterally braced at quarter points and mid-length.

7.2 Design using the equivalent static force procedure (ESFP)

Key design parameters and results for the ESFP are given in table 2 for the four codes used. For NCh433, the results are presented for two different design solutions, strength and drift designs, as will be discussed later.

For the NBCC design, the structure period was initially set equal to the upper limit permitted by the code, i.e. $T_a = 0.05\ h_n = 1.86$ s, which gave a base shear ratio V/W equal to 0.0913 (see figure 3a). Storey drifts Δ_x including inelastic effects were also initially posed equal to 0.01 h_s at every level to calculate preliminary values for the U_2 factors so that P-delta effects could be included in the first design trial. Using these assumptions, the frame members were selected to satisfy minimum strength requirements and the structure was re-analysed to obtain its fundamental period and the storey drifts. The computed period was equal to 1.75 s, shorter than the initially assumed value, and the storey drifts varied from 0.011 h_s at the base level to 0.024 h_s at the uppermost level. Seismic loads and P-delta effects were reassessed using these values and the frame design was modified accordingly. The process was repeated until convergence was reached and the results for the final design are presented in table 2. The structure period is 1.67 s and the associated base shear is equal to 0.102 W. The storey drifts and U_2 factors are all smaller than the applicable code limits, respectively 0.025 h_s and 1.4, and the structure need not be stiffened.

In table 2, the base shear V and the storey shear at the first storey including notional loads and amplified by the U_2 factor are respectively 9094 and 10560 kN, thus an increase of 16% in lateral force demand due to stability design requirements. Effects of notional loads and P-delta amplification on design storey shears are illustrated in figure 6a. As shown, the latter had more pronounced impact on the design: the maximum increase due to notional loads is 6% at the base whereas the factor U_2 varies from 1.07 to 1.12 over the frame height. For this design based on NBCC static force procedure, the amount of steel required for the two BRBFs is 180 t. For all frames, the steel tonnage was evaluated considering brace core cross-sectional areas are multiplied by 3.0 to include the extra material required for the brace end protrusions and the buckling restraining mechanism. For beams and columns, 10% allowance was considered for connections. To further assess the impact of stability design requirements, two additional designs were performed: one where U_2 was set equal to 1.0 and another design where U_2 was equal to 1.0 and the notional loads were omitted. Steel quantities required for these two designs are respectively 161 and 151 t. Thus, for this structure, the steel needed for the BRBFs increased by 7% due to application of the notional loads and by a total of 19% when further increasing lateral loads for P-delta effects.

In table 2, the maximum value of the stability coefficient θ_x as defined in the ASCE 7 code (Equation (12)) is presented to compare the importance of the P-delta effects for all frames. The same parameter is used for all four countries to obtain a uniform comparison basis. The distribution of that parameter over the height is illustrated in figure 6d. For the frame in Canada, the maximum value is 3.2%, which means that P-delta effects could have been ignored had the frame been designed in accordance with the U.S. provisions.

Table 2: Seismic design parameters and results - ESFP (/building).

Parameter	NBCC	ASCE 7 (B_2 = 1.0)	ASCE 7 (with B_2)	NCh433 (strength design)	NCh433 (drift design)	NZS 1170.5[5]
T (s)	1.67	1.54	1.54	1.85	1.37	2.0
Modification factor	R_dR_o = 4.8	R = 8.0	R = 8.0	R = 6.0	R = 6.0	μ/S_p = 4.3
Seismic weight	W = 88840	W = 88470	W = 88470	W = 108230	P = 108230	W_t = 104660
Base shear ratio	V/W = 0.102	V/W = 0.040	V/W = 0.040	Q_o/P = 0.070	Q_o/P = 0.070	V/W_t = 0.050
Base shear (kN)	V = 9094	V = 3542	V = 3542	Q_o = 7576	Q_o = 7576	V = 5177
Design base shear (kN)	10560[1]	4605[2]	5065[3]	10610[4]	10610[4]	5929[6]
Computed T_1 (s)	1.67	2.48	2.36	1.85	1.35	2.0
Maximum drift (/h_s)	Δ_x = 0.024	Δ_x = 0.020	Δ_x = 0.019	Δ_{xe} = 0.0038	Δ_{xe} = 0.002	Δ_x = 0.015
Maximum P-Δ effects	$U_{2,x}$ = 1.12	$B_{2,x}$ = 1.0	$B_{2,x}$ = 1.12	–	–	1.21
Maximum ASCE 7 θ_x	0.032	0.071	0.064	0.038	0.021	0.047
Steel tonnage (t)	180	94	104	153	309	125

Notes: [1]Includes notional loads and P-Δ effects (factor U_2).
[2]Includes the redundancy factor ρ = 1.3.
[3]Includes the redundancy factor ρ = 1.3 and P-Δ effects (factor B_2).
[4]Includes the 1.4 load factor.
[5]Based on ULS design for strength and stiffness at 10% probability of exceedance in 50 years.
[6]Includes the P-delta contribution.

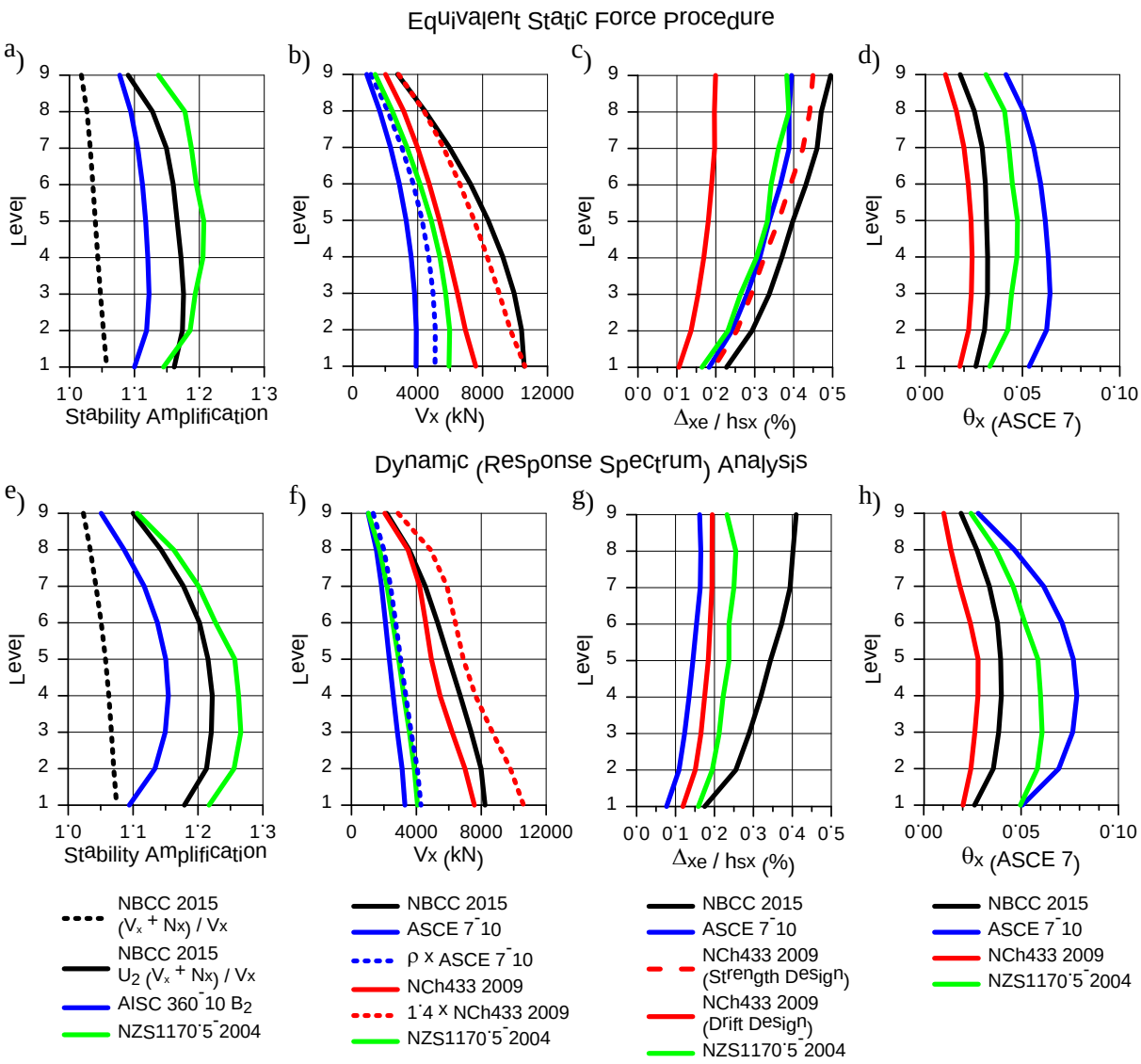

Figure 6: Results from ESFP and response spectrum analysis: a & e) Amplification of storey shears in NBCC, AISC 360, and NZS 1170.5; b & f) Storey shear demands including stability effects; c & g) Elastic storey drifts; and d & h) ASCE 7 stability coefficients.

The same approach was adopted for the design of the structure in the U.S., except that two BRBF designs were examined: one for which the stability requirements of AISC 360-10 were omitted (labelled B_2 = 1.0 in table 2) and one for which these requirements were considered. For the first design, only one iteration was required to reach the final design described in table 2. As shown, the computed period for this frame (T = 2.48 s) is longer than the upper limit C_uT_a = 1.54 s that must be used for determining member forces, meaning that the seismic force demands would not change in subsequent iterations as the period limit would then control the base shear. In addition, the computed maximum stability coefficient and storey drift values over the frame height are 0.071 and 0.02 h_s, respectively. Hence, P-delta amplification on member forces and displacements could be omitted, as permitted in ASCE 7-10 when θ is smaller than 0.10, and the frame lateral stiffness was sufficient to meet the code prescribed drift limit (0.02 h_s) and the θ_{max} value (0.10). It is noted that member forces are determined with ρ = 1.3 while drifts and stability coefficients do not include the redundancy factor.

In the second design, P-delta effects were considered using the AISC 360 B_2 multiplier. This required iterative design as storey drifts and, thereby, B_2 factors and member forces varied with member sizes. In table 2 and figure 6a, the B_2 multiplier for the converged design solution varies between 1.08 and 1.12, which resulted in a heavier (104 *vs* 94 t) and stiffer frame (T = 2.36 *vs* 2.48 s). The increased stiffness also led to smaller storey drift and stability coefficient values. Note that drifts are not amplified for P-delta effects as the stability coefficient is less than 0.1 at every level. The B_2 factor was only applied to storey shears used to calculate the required axial strengths for the braces. Had P-delta amplification been required by ASCE 7, the amplifier 1/(1-θ) would have been less than the B_2 factor in AISC 360. For instance, θ = 0.064 at level 3 where P-delta effects are maximum, which gives an amplification of 1.07, less critical than B_2 = 1.12. This observation was expected because B_2 is calculated with heavier gravity loads (from Equation (11)) and larger storey drifts (from an analysis with a reduced frame stiffness for inelasticity). In ASCE 7-10, the computed structure period is permitted to be used without upper limit for determining the seismic loads used to calculate storey drifts. In both frame designs, however, the minimum seismic load $V = 0.04W$ applied for periods longer than 1.43 s (see figure 3a) and it was not possible to take advantage of the actual frame longer periods to reduce the loads used for the calculation of the displacements.

As could be expected from figure 3a, the design earthquake load for this structure is markedly lower than the one prescribed in the NBCC: 3542 *vs* 9094 kN in table 2. As mentioned, this difference is mainly due to the lower design spectral ordinates and higher R factor specified in ASCE 7-10. When applying the stability requirements in the NBCC and AISC 360-10 and the redundancy factor of ASCE 7-10, the Canadian frame is designed for approximately 2 times the lateral loads used for the same structure in the U.S. Differences in storey shear demands are illustrated in figure 6b. These differences resulted in much a lighter braced frame design in the U.S. (104 t) compared to Canada (180 t). The computed storey drifts are compared in figure 6c for the two designs. In the figure, elastic drift values before amplification for inelastic effects (Δ_{xe}) are shown to allow direct comparison with Chilean drift values. The amplification in Canadian and U.S. codes for BRBFs being very similar (R_dR_o = 4.8 *vs* C_d = 5.0), the figure also permits comparison between these two codes. As shown, storey drifts are nearly the same for both North American designs. The U.S. building is however more flexible as the drifts for this structure are computed using lower seismic loads. This higher lateral flexibility is apparent in figure 6d where the ASCE 7-10 stability coefficients for each

design are compared. As was indicated, the stability coefficients satisfy the limit θ_{max} over the height of the U.S. frame but the values for this structure are significantly higher than those computed for the BRBFs in the other three countries.

In Chile, after completing the design iterations considering both strength and drift requirements, it was found that the sizes obtained for beams and columns were too large to be practical. Therefore, it was decided to consider two independent braced bays per edge instead of one, as illustrated in figure 1a. The design was initiated assuming a fundamental period $T = 0.1N = 0.9$ s and members were first selected to meet strength requirements. The building period was computed (1.82 s) and used to determine a new set of lower seismic loads. The reduction was small as the base shear is unchanged past a period $T = 0.93$ s (see figure 3a). The design could be refined slightly to obtain the "strength design" presented in table 2. For this structure, $T = 1.85$ s and the base shear Q_o is equal to $0.070\ P = 7576$ kN. When including the 1.4 load factor, the design base seismic load is 10610 kN, which is nearly the same as the base shear including stability effects in Canada. Such a high shear force demand in Chile is partly due to the larger seismic weight specified in NCh433 which includes a portion of the floor live load. In figure 6b, the vertical distribution of storey shears is however relatively less severe than those obtained in Canada and the U.S.

For this design, storey drifts Δ_{xe} vary from $0.0019\ h_s$ to $0.0038\ h_s$ from the first to ninth levels. As shown in figure 6c, these values compare well with the drifts of the two North American designs but exceed the $0.002\ h_s$ limit of NCh433 at all but the first level. For this frame, examination of the response showed that brace axial deformations contribute $0.0013\ h_s$ drifts at every level while column axial deformations induce drifts up to $0.0023\ h_s$ at the top level. Column straining in the basement level alone results in $0.0004\ h_s$ drift at all levels above, i.e. 20% of the code allowable drift, indicating that frames with columns extending under ground level may not represent an effective configuration when tight drift limits have to be met. In contrast, beam axial deformations were small. Hence, the structure was stiffened to obtain the "drift design" in table 2 by increasing the cross-sectional area of all BRBs. Column sections were also increased, with large changes in all but the top two levels. Column and beam sizes were also adjusted as needed to resist the higher force demands imposed by the stronger braces. As a result of the modifications, the period reduced from 1.85 to 1.37 s. This change had no effect on seismic loads as minimum base shear requirements still controlled for the shortened period. The storey drifts and stability coefficients computed over the frame height are plotted in figures 6c&d for this "drift design". As anticipated, seismic induced drifts and θ values are smaller than those obtained with the other three codes. Stability coefficients in Chile and Canada are however comparable for this structure. For this example, satisfying the stringent drift limitations had a major impact on design as it required two times more steel than the amount necessary to meet strength design requirements (309 *vs* 153 t). The final frame design in Chile is also 1.7 and 3.0 times heavier than those in Canada and the U.S., respectively. Note that steel tonnage for the structure in Chile structure is for four braced bays but the steel required for two exterior gravity bays was deducted from the total to allow direct comparison with the structures in the other countries.

As in Chile, the seismic weight in New Zealand is higher than in North America as it includes the full dead load plus a fraction of the floor live load. The period used to determine the base shear V is also the computed frame fundamental period T_1, without an upper limit. That period is equal to 2.0 s, the limit beyond which ESFP is no longer permitted. In table 2, the resulting force V is however lower compared to Chile due to the longer design period, the absence of a minimum value for $C_d(T_1)$ and the use of the 1.4 seismic load factor in Chile. In spite of the larger seismic weight, V in NZS is also much lower than the Canada value due to the differences in design periods and hazard levels. It is higher than the base shears in the U.S., essentially

because of R (8.0) being larger than μ/S_p (4.3). The stability coefficients determined using the unmodified storey drifts Δ_x obtained with the lateral displacements from analysis multiplied by μk_d = 2.55 range between 0.08 and 0.12 and P-delta effects had to be included in design. Method B of NZS1170.5 was used for this purpose. In step 3, the additional P-delta storey shears ($\theta_x V_x$) were calculated together with the additional displacements they induced. In step 4, the scaling factor was β = 1.71, as obtained from Equation (28) with μ = 3 and K = 1.0 (T_1 = 2.0 s and class B site). The resulting amplification in design storey shears due to P-delta effects varies between 14 and 21% over the structure height, which is the largest among all structures studied (figure 6a). The additional storey drifts caused by P-delta effects (from step 3, as amplified by the β factor), represent between 4.1 and 5.2% of the total storey drifts. As shown in table 2, the maximum storey drift is 1.55% h_s, including the k_{dm} factor of 1.5, which is less than the code limit of 2.5% h_s. At the end, the required steel tonnage is approximately 20% higher than in the U.S., comparable to the Chile "strength design" and significantly lower than the material needed in Canada or for the Chilean "drift design".

7.3 Design using the dynamic (response spectrum) analysis

For all four countries, response spectrum analysis (RSA) was performed using a structural model of the final design resulting from the ESFP. Key design results are summarized in table 3. For the structure in Chile, only the design that satisfies the drift limits is presented and discussed.

For the structure in Canada, response spectrum analysis of the BRBFs gave a base shear V_d = 7781 kN, a value between the base shear V from static analysis (9094 kN in table 2) and the minimum required base shear 0.8V = 7275 kN. Force and displacement results from the analysis could then be used without adjustments, which permitted to diminish member sizes. The re-designed frame had a longer fundamental period (1.79 s), which allowed further reduction in seismic loads. Analysis and re-design steps were repeated until the process converged, and the properties for the final design are given in table 3. When compared to the ESFP design, the frame period changed from 1.67 to 1.87 s, leading to smaller base shears V and V_d. The latter is equal to 6984 kN, 23% less than the design base shear from the static force procedure. Comparing figures 6b and 6f reveals that the storey shear demand at intermediate levels is also relatively less critical from dynamic analysis. This reduced lateral force demand from RSA diminished the required steel tonnage by 20%, from 180 to 144 t. In table 3, the stability design requirements increased the storey shear at the frame base by 18% to reach 8232 kN. The maximum storey drifts reduced compared to static force procedure values (from 0.024 to 0.020 h_s) but the maximum U_2 factor slightly increased from 1.13 to 1.15 because the frame lateral strength from RSA is lower. Similar to the EFSP design, the maximum ASCE 7 stability coefficient θ_x for this frame is less than 0.10 (= 0.04) and no lateral strength increase would have been needed had the structure been located in the U.S.

In the U.S., the frames designed with the static force procedure had long periods (2.48 and 2.36 s) and response spectrum analysis resulted in base shears V_t lower than the minimum base shear of 0.85V = 3011 kN, where V = 3542 kN obtained with $C_u T_a$ = 1.54 s, as specified in ASCE 7-10. Frame members could then be redesigned for the forces from the analysis, after scaling the analysis results up to obtain a base shear of 3011 kN. Two designs were obtained depending whether or not stability provisions from AISC 360-10 were included. As shown in table 3, these two frame designs are lighter and more flexible (have longer periods) than their counterpart designed with the static force procedure. This is essentially because member forces are associated to a base shear of 0.85 V in dynamic analysis compared to V in the static force method. For both frames, the computed storey drifts and stability coefficients after satisfying minimum member strength requirements were within the applicable limits; hence frame

stiffening was not necessary. It is noted that RSA was performed with structural models with unreduced stiffness properties and the storey drifts from analysis were used as is, without base shear adjustments. The latter explains the much smaller values compared to those from the static force procedure, as can be seen in figure 6g for the case where AISC 360-10 stability provisions were considered. For that case, storey drifts were not amplified for second-order effects because the stability coefficients are less than 0.10. However, for strength design, the B_2 multiplier was applied to member forces. As is done for θ, B_2 was determined using storey drifts multiplied by the base shear ratio 3011/1568 to have consistent storey shear and storey drift values reflecting storey stiffness. In addition, storey drifts for B_2 were multiplied by 1/0.8 to simulate reduced stiffness effects. As shown in figure 6e, the B_2 multipliers vary between 1.05 and 1.15, which resulted in a 7.5% increase in steel tonnage compared to the case where AISC 360-10 provisions are omitted in design (86 *vs* 80 t).

As was the case when the ESFP was used, the design storey shears for the frame in the U.S. when including the redundancy factor and P-delta effects are approximately half the values considered for the frame in Canada (4284 *vs* 8232 kN), and the steel weights required in both countries exhibit similar proportions (86 *vs* 144 t). The lighter frames obtained from RSA in the U.S. have however less capacity against second-order overturning moments, as reflected by the higher stability coefficients in figure 6h. In tables 2&3 and figures 6a&e, the factors U_2 (Canada) and B_2 (U.S.) take comparable values for both analysis methods; however, this similarity is coincidental as the two factors are computed with different gravity load, storey drift and storey shear values.

Table 3: Seismic design parameters and results - Response spectrum analysis (/building).

Parameter	NBCC	ASCE 7 (B_2 = 1.0)	ASCE 7 (with B_2)	NCh433 (drift design)	NZS 1170.5[5]
T (s)	1.87	2.74	2.63	1.55	2.5
Modification factor	R_dR_o = 4.8	R = 8.0	R = 8.0	R^* = 8.95	μ/S_p = 4.3
Seismic weight	W = 88840	W = 88470	W = 88470	P = 108230	W_t = 104663
Base shear ratio	V/W = 0.0906	V/W = 0.0400	V/W = 0.0400	P/Q_o = 0.070	V/W_t = 0.040
Base shear (kN)	V = 8052	V = 3542	V = 3542	Q_o = 7576	V = 3345
Min. base shear (kN)	0.8 V = 6442	0.85 V = 3011	0.85 V = 3011	$Q_{o,min}$ = 7576	0.8 V = 4130
Analysis base shear (kN)	V_d = 6984	V_t = 1506	V_t = 1568	3372	3345
Design base shear (kN)	8232[1]	3914[2]	4283[3]	10610[4]	4071[6]
Max. drift (/h_s)	Δ_x = 0.020	Δ_x = 0.008	Δ_x = 0.008	Δ_{xe} = 0.002	Δ_x = 0.010
Max. P-Δ effects	$U_{2,x}$ = 1.15	$B_{2,x}$ = 1.0	$B_{2,x}$ = 1.15	-	1.27
Maximum ASCE 7 θ_x	0.040	0.086	0.079	0.028	0.061
Steel tonnage (t)	144	80	86	255	91

Notes: [1]Includes notional load and P-Δ effects (U_2 factor).
[2]Includes the redundancy factor ρ = 1.3.
[3]Includes the redundancy factor ρ = 1.3 and P-Δ effects (B_2 factor).
[4]Includes the 1.4 load factor.
[5]Based on ULS design for strength and stiffness at 10% probability of exceedance in 50 years.
[6]Includes the P-delta contribution.

In Chile, contrary to Canadian and U.S. codes, the same minimum base shear requirement ($ISA_oP/6g$ = 7576 kN) is specified for both the ESFP and the response spectrum analysis. Consequently, when this limit applies, as is the case here, no reduction in design seismic loads is obtained when using dynamic analysis. However, for this frame, relatively smaller lateral displacements were computed with the response spectrum analysis and since the NCh433 drift limit governed the design, member sizes could be reduced while still satisfying storey drift limit. As shown in table 3, the building fundamental period T^* is 1.55 s for the final design,

which gave a force reduction factor R^* of 8.95 for the site class C conditions. When applied to the design spectrum, this high factor resulted in a low base shear (3372 kN) but displacement and force results from analysis had to be scaled up to meet the minimum base shear of 7576 kN. The building in Chile therefore requires the heaviest BRBFs among the four countries (255 t), even if a large force reduction factor was permitted. The frame design in Chile resulted in high lateral stiffness and, as illustrated in figure 6h, smaller stability coefficient values.

Similar to the North American designs, the results from RSA in New Zealand are adjusted with respect to 80% of the base shear from the EFSP, which leads to reduced seismic actions. The frame is therefore more flexible than its EFSP counterpart (T_1 has increased from 2.0 to 2.5 s) and P-delta amplification for storey shears is more pronounced, as evidenced by comparing figures 6a and 6e. Method B was also used to account for P-delta effects in this design and the maximum amplification of V_x is equal to 1.27 at the third level (table 3). Coincidently, figure 6f shows that the design storey shears for the New Zealand frame are very close to those required in the U.S. when including the 1.3 redundancy factor. Compared to the NZ frame designed with the ESFP, the amplification of the storey drifts due to P-delta effects also increased to reach 7.4%. However, the total storey drifts, including inelastic effects were still well within the code limit (maximum = 0.010 h_s in table 3) and strength governed the design of the frame. In table 3 and figure 6 h, the maximum ASCE 7-10 stability coefficient is 0.061, much higher than for the frames in Canada and Chile, but smaller than the values obtained from the U.S. structures. The frame in New Zealand is also lighter that the Canadian and Chilean ones but slightly heavier than the U.S. designs.

8 CONCLUSIONS

Seismic design provisions for steel buckling restrained braced frames (BRBFs) in Canada, the U.S., Chile, and New Zealand were reviewed and compared for locations having similar seismic data and local site conditions. The seismic design requirements were applied to a single-bay, chevron BRBF used in a 9-storey office building located at the selected four sites. Earthquake effects were determined using the equivalent static force procedure (ESFP) and the dynamic response spectrum analysis (RSA) method. The main conclusions of the study can be summarized as follows:

- All codes have similar seismic loading and analysis requirements. However, differences in hazard level, spectral shape and treatment of higher modes, design period, seismic weight, ductility factor, consideration of redundancy and overstrength, and minimum force levels resulted in different design seismic loads. For the frame studied, the seismic loads in Canada and Chile were comparable. Seismic loads in the U.S. were the lowest by a significant margin, essentially due to a higher force modification factor. Those in New Zealand were higher than the U.S. values but markedly lower than the Canadian and Chilean values. The use of dynamic analysis generally resulted in lighter frame designs compared to the ESFP.
- Design seismic provisions in Canada, the U.S., and New Zealand include minimum stability requirements that are aimed at preventing structural collapse by dynamic instability in the nonlinear range. All three country's codes require amplification of the seismic effects and an upper limit on the P-delta overturning moments. In Canada, these requirements are based on the frame lateral strength whereas the frame lateral stiffness is used in the U.S and New Zealand. Notional loads are also specified in Canadian codes. In Chile, stability under lateral seismic demand is provided indirectly by means of stringent drift limits.

- Similar seismic design and detailing requirements are provided for BRBFs in steel design standards in Canada and the U.S. Codes in Chile and New Zealand do not include specific provisions for BRBFs but in Chile the system can be designed using U.S. standards as currently done for other steel seismic force resisting systems. In New Zealand a design and detailing guide has been published.
- Other design provisions such as drift limits (Chile), load combinations, consideration of gravity loads in the design of the BRB members (Canada) also influenced designs. In Chile, drift limits resulted in reduced stability effects.
- For the structure studied, impacts of the code seismic stability requirements were more pronounced for the structures located in Canada and New Zealand. In the U.S., the frame design was only affected when applying the stability provisions from AISC 360-10 and the effects were less significant.
- Due to the lower design seismic loads, the frames in the U.S and New Zealand are more flexible and have higher P-delta moments relative to the primary seismic overturning moments.

In Canada and New Zealand design standards, there is a requirement for all columns in the structure to be spliced such that the columns are continuous. This means the columns in the gravity system can assist in overall frame stability and in self-centering following an earthquake. In the 2010/2011 Christchurch earthquake series, all multi-storey steel frame buildings situated on stable ground effectively self-centered. The benefits of column continuity on frame stability should be investigated further in future studies.

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REFERENCES

[1] NRCC. *National Building Code of Canada 2015*, 14th ed., National Research Council of Canada, Ottawa, ON, 2015.

[2] CSA. *CSA S16-14, Design of Steel Structures*, Canadian Standards Association, Mississauga, ON, 2014.

[3] ASCE. *ASCE/SEI 7-10, Minimum Design Loads for Buildings and Other Structures*, American Society of Civil Engineers (ASCE), Boca Raton, FL, 2010.

[4] AISC. *AISC-341-10, Seismic Design Provisions for Steel Structures*, American Institute of Steel Construction (AISC), Chicago, IL, 2010.

[5] INN. *NCh433.Of1996, Diseño sísmico de edificios, including 2009 modifications and 2011 Decreto 61*, Norma Chilena Oficial, Santiago, CH, 2009.

[6] NZS1170.5:2004, *Structural Design Actions - Part 5: Earthquake Actions – New Zealand*, Standards New Zealand, Private Bag 2439, Wellington 6020, New Zealand, 2004.

[7] Wijanto, S. *Guide to Seismic Design of Buckling Restrained Braced Frames*. Steel Construction New Zealand, Auckland, New Zealand, 2015.

[8] Sabelli, R., and Dean, B. "Comparison of US and Canadian code requirements for seismic design of steel buildings", *Proceedings of the 9th U.S. National and 10th Canadian Conference on Earthquake Engineering*, Toronto, ON. Paper No. 296, 2010.

[9] Gupta, A., and Krawinkler, H. *Seismic demands for performance evaluation of steel moment resisting frame structures*. Report No. 132, The John A. Blume Research Center, Dept. of Civil and Environmental Engineering, Stanford University, San Francisco, CA, 1999.

[10] Leyton, F., Ruiz, S., and Sepulveda, S.A. "Reevaluacion del peligro sismico probabilistico en Chile central", *Andean Geology*, 37, 2: 455-472, 2010.

[11] Filiatrault, A., Tremblay, R., Christopoulos, C., Folz, B., and Pettinga, D. *Elements of Earthquake Engineering and Structural Dynamics*, 3rd ed., Presses Internationales Polytechnique, Montreal, QC, 874 p., 2013.

[12] AISC. *AISC-360-10, Specification for Structural Steel Buildings*, American Institute of Steel Construction (AISC), Chicago, IL, 2010.

[13] Nair, S.R., Malley, J.O., and Hooper, J.D. "Design of Steel Buildings for Earthquake and Stability by Application of ASCE 7 and AISC 360", *Engineering Journal*, AISC, 3rd Quarter, 199-204, 2011.

[14] AISC. *Seismic Design Manual*, 2nd ed., American Institute of Steel Construction (AISC), Chicago, IL, 2012.

[15] NEHRP. *Comparison of U.S. and Chilean Building Code Requirements and Seismic Design Practice 1985–2010*, Report No. NIST GCR 12-917-18, 2012.

[16] Feeney, M.J., and Clifton, G.C. *Seismic Design Procedures for Steel Structures*, Report No. R4-76 New Zealand HERA, Manukau City, Auckland, New Zealand, 2001.

[17] AS/NZS 1170 Set, *Structural Design Actions, Standards New Zealand*, Wellington 6140, New Zealand, 2004.

[18] NZS3404:1997/2001/2007, *Steel Structures Standard*, Incorporating Amendment No. 1 and Amendment No. 2, Standards New Zealand, Wellington 6140, New Zealand, 2007