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

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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. 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. P-delta effects are explicitly considered in seismic design in Canada, the U.S. and New Zealand. In Chile, P-delta effects are effectively minimised by means of more stringent drift limits. 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 compared with in the USA

1 INTRODUCTION

This paper summaries the findings of Tremblay et al, [1] which presents a comparison between buckling restrained braced frames (BRBFs) for the same building size and plan, located in similar seismic zones within Canada, the USA, Chile and New Zealand and designed to the relevant country's seismic provisions. 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) [2] whereas the latest design and detailing requirements are specified in the 2014 CSA S16 steel design standard [3]. BRBFs were introduced into 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 [4] and the AISC 341-10 Seismic Provisions [5]. Buckling restrained braced frames have been used in New Zealand with the first application in 1992 to a building expansion, and are now used on a reasonable number of buildings, especially in Christchurch rebuild projects. BRBFs are being introduced in Chile. No specific guidance has been introduced yet in the building codes of New Zealand and Chile [6, 7]. 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 [8].

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 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. 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.

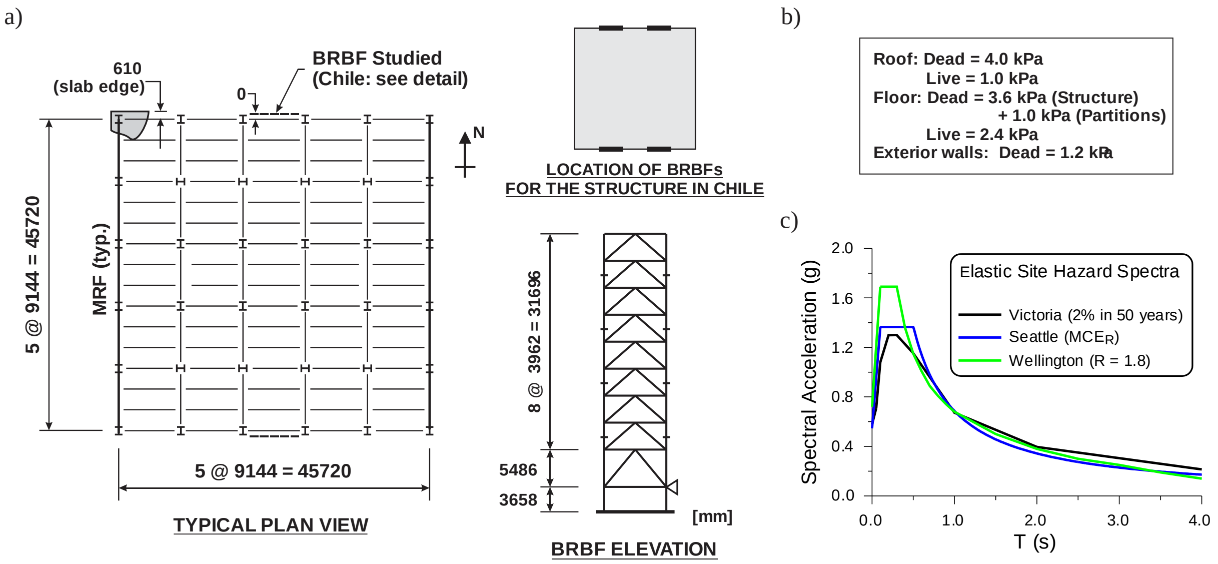

Figure 1. a) Prototype structure; b) Design gravity loads; c) Elastic hazard response spectra.

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. In New Zealand, the selected site is located is Wellington. 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.

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).

Table 1. Spectral ordinates in the 2015 NBCC, ASCE 7-10 and NZS1170.5-04.

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

Figure 2. Base shear ratios and dynamic response spectra for the BRBs from the various sites.

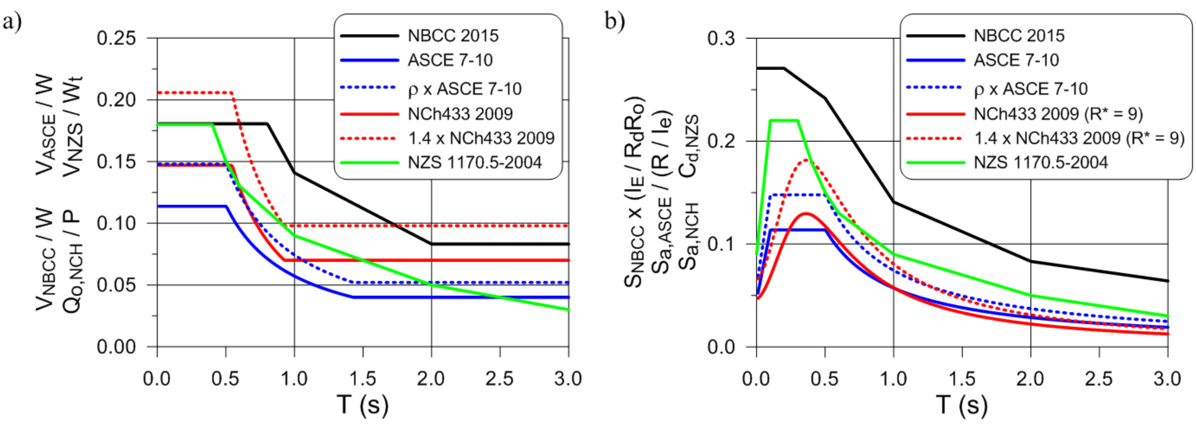

Figure 2. a) Base shear ratios (static analysis); and b) Design response spectra (dynamic analysis) for buckling restrained braced frames at the selected sites.

3 SEISMIC DESIGN PROVISIONS

The determination of seismic loads in each of the four countries is presented in detail in (Tremblay et al [1]), to which interested readers must refer, as the limit on number of pages means they cannot be included herein.

4 SEISMIC DESIGN OF THE PROTOTYPE BUILDING

The seismic code provisions for the four countries are applied for the design of the BRBFs of the 9-storey regular building. 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 Canada 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 enable a uniform comparison basis.

4.1 Design data

The buckling restrained braces of the structures are sized assuming that the yield stress of the core material is: F_{ysc} = 290 MPa. Hence, R_y = 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.

4.2 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.

As shown in table 2 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. 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 3a). The additional storey drifts caused by P-delta effects, 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".

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 2a). 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.

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.

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 3a. 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 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 3d. 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.

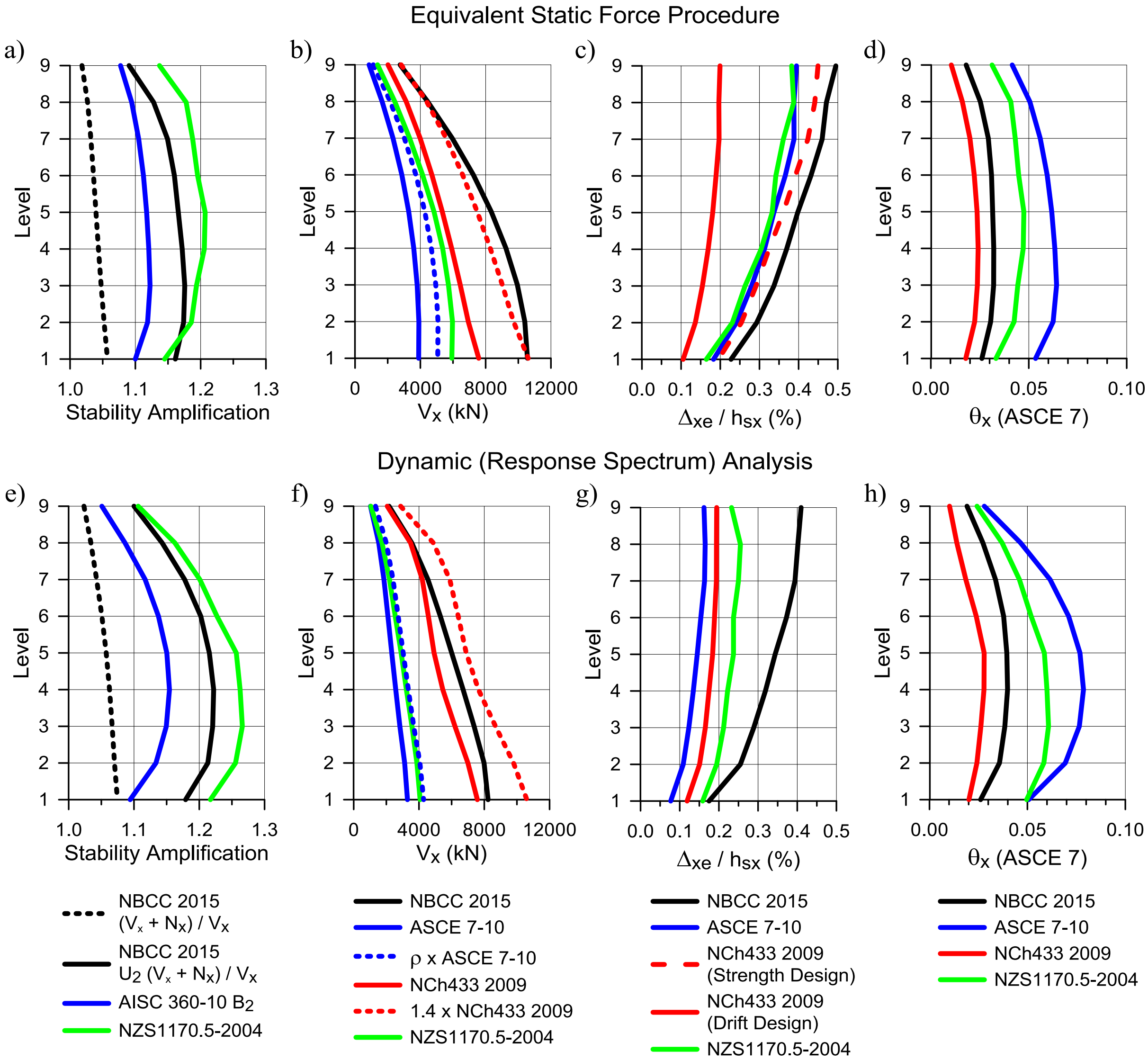

Figure 3. Results from ESFP and response spectrum analysis: a & e) Amplification of storey shears in NBCC, AISC 360, and NZS 1170.5; b & f) Storey shear demands including stability effects; c & g) Elastic storey drifts; and d & h) ASCE 7 stability coefficients.

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 [11] 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 $C_u T_a$ = 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.

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 3a, 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 $1/(1\text{-}\theta)$ would have been less than the B_2 factor in AISC 360. This observation was expected because B_2 is calculated with heavier gravity loads and larger storey drifts (from an analysis with a reduced frame stiffness for inelasticity). In both frame designs, however, the minimum seismic load $V = 0.04W$ applied for periods longer than 1.43 s (see figure 2a) 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 2a, 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 3b. 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 3c 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 ($R_d R_o$ = 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 3d 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 2a). 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 3b, 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 3c, 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 3c&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 2.5, 1.7and 3.0 times heavier than those in New Zealand, 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.

4.3 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.

The differences between the frames designed to each countries' provisions were similar to those shown for the ESFP and discussed in detail above. Space limitations mean only one of the two sets of results can be presented herein, except as given in Figure 3 e) to h) so readers will have to refer to [1] for the detailed comparison of the RSA designs.

5 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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