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CANADIAN PROVISIONS FOR THE SEISMIC DESIGN OF SINGLE-STOREY STEEL BUILDINGS WITH FLEXIBLE ROOF DIAPHRAGMS

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Abstract

The seismic design provisions of the 2015 National Building Code of Canada (NBCC) for single-storey steel buildings with flexible metal roof deck diaphragms are presented. The 2015 NBCC provisions include a new expression for the fundamental period of vibration of buildings used to determine the minimum seismic design loads. The new period equation accounts for the flexibility of both the vertical bracing and roof diaphragm components. The NBCC also requires that the inelastic deformation demand on the vertical bracing elements be evaluated with consideration of the diaphragm in-plane flexibility effects. If needed, the design must be corrected to keep the anticipated deformations within the system deformation capacities. Magnification of roof diaphragm in-plane shears and moments due to diaphragm flexibility must also be taken into account for the design of the diaphragm. The new provisions are applied to a prototype single-storey building. Concentrically braced frames (CBFs) of the moderately ductile category are used and two bracing systems are examined: tension/compression and tension/only bracing. In addition, the building is assumed to be located at two different sites in eastern and western regions of Canada to account for the differences in seismic hazard. The buildings are assumed to be constructed on firm ground sites. In design, diaphragm shears and moments were predicted using a modified response spectrum analysis method. The design is found to significantly influenced by the seismicity level at the site and the bracing system used, which resulted in various properties and overstrength levels among the structures studied. The seismic response of the structures was examined though nonlinear response history analysis. The analyses showed that diaphragm flexibility effects on braced frame deformations were not pronounced for the structures studied. Diaphragm shears and moments can be well predicted using the proposed method.

Keywords: Roof deck diaphragm; Concentrically braced frame; Dynamic magnification; Diaphragm flexibility; Single-storey steel building; Design.

1. Introduction

Structural steel is commonly used in Canada for single-storey buildings employed for industrial, recreational, and commercial applications. In these structures, the roof structure typically includes corrugated steel roof deck panels supported on a roof frame consisting of open-web steel joists and I-shaped steel girders (Fig. 1a). I-shaped or tubular steel members are used for the columns. The roof deck panels are connected to each other and to the supporting framing system to form an in-plane diaphragm that can resist and transfer to the vertical bracing elements lateral wind and seismic loads acting at the roof level. As illustrated in Fig. 1a, vertical bracing elements are generally placed along the building perimeter to minimize obstruction in the structure. Lateral loads induce in-plane shear forces and bending moments in the roof diaphragm, causing horizontal in-plane deformations of the diaphragm Δ_D that add to the deflection of the vertical bracing, Δ_B (Fig. 1b).

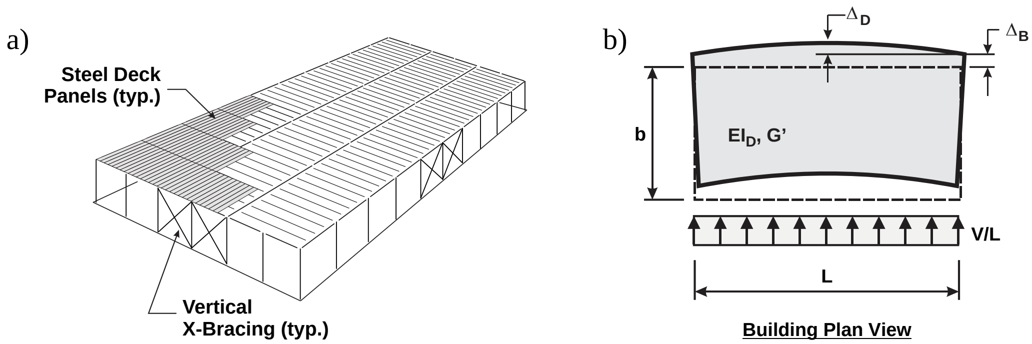

Fig. 1 - Typical single-storey steel building with flexible roof deck diaphragm and perimeter vertical bracing: a) Structure overview; b) Lateral deformation of the roof diaphragm (Δ_D) and vertical bracing (Δ_B) under lateral loading.

New seismic design provisions have been included in the 2015 National Building Code of Canada (NBCC) to account for the influence of the in-plane flexibility of the roof diaphragm on the dynamic seismic response of single-storey buildings [1]. The changes include a new expression for the building fundamental period of vibration, which is function of the building height and diaphragm length. Engineers must also verify that the ductility demand on the vertical elements of the seismic force resisting system (SFRS) remains within acceptable values for the selected system. Lastly, dynamic magnification of diaphragm shears, moments and deformations must be explicitly accounted for in the design of the diaphragm. This article outlines the NBCC seismic provisions for single-storey steel buildings with focus on the changes implemented in 2015. The application of the provisions is illustrated for a typical structure constructed with moderately ductile concentrically steel braced frames. Tension-compression and tension-only designs are examined. The structure is assumed to be located in two seismically active and populated regions of Canada to examine the effects of ground motion characteristics on design and seismic response: Vancouver, British Columbia, in western Canada, and Montreal, Quebec, in eastern Canada. The seismic response of the structure is then examined through nonlinear response history analysis to validate design methods adopted to predict the ductility demands on the vertical bracing elements and the shear forces in the roof diaphragm.

2. Seismic Design

2.1 NBCC 2015 Seismic Design Provisions

In the 2015 NBCC, the minimum design seismic load, V, is given by:

$$V = \frac{S(T_a) M_V I_E W}{R_d R_o} \tag{1}$$

where S is the design spectrum, T_a is the building fundamental period of vibration for design, M_V accounts for higher mode effects on base shear for multi-storey buildings, 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 obtained from the products of site coefficients $F(T)$ and uniform hazard spectral (UHS) accelerations $S_a(T)$ specified at periods T = 0.2, 0.5, 1.0, 2.0, 5.0, and 10 s. Site coefficient values depend on the site class and the reference peak ground acceleration at the site, PGA_{ref}. The S_a values are specified for a probability of exceedance of 2% in 50 years. The design spectra for the two locations considered in this study are plotted in Fig. 2a for site classes A (hard rock), C (soft rock and firm ground), and E (soft soils). In Eq. (1), M_V takes a value of 1.0 for single-storey buildings; for multi-storey buildings, its values depends on the SFRS type, the period T_a, and the ratio between spectral ordinates $S(0.2)$ and $S(5.0)$. The importance factor I_E in Eq. (1) varies from 1.0 for buildings of the normal importance category to 1.5 for post-disaster buildings. The seismic weight W is equal to the dead load plus 25% of the design roof snow load. The factor R_d ranges from 5.0 for the most ductile systems to 1.0 for brittle ones [2]. The R_o factor reflects the dependable lateral overstrength of the SFRS. It varies between 1.0 and 1.5 depending on the SFRS. For short period structures with minimum ductility ($R_d \geq 1.5$), the force V from Eq. (1) need not exceed 2/3 the value computed at a period of 0.2 s, but not less than V at 0.5 s. For tall SFRSs with shear dominated inelastic response, V must be at least equal to the value computed with T_a = 2.0 s.

The period of vibration T_a can be taken as the fundamental period of the structure from dynamic analysis, T_1; however, when determining the load V for verification of strength requirements, T_a cannot exceed upper limits based on empirical period estimates given in the NBCC. For steel braced frames, the empirical expression for T_a is: $T_a = 0.025\, h_n$, where h_n is the building height (in meters), and the upper limit on T_a is equal to two times that value ($0.05\, h_n$). This empirical expression was developed for multi-storey buildings with rigid floor and roof diaphragms [5]. It was recognized in the 2015 NBCC that in-plane deformations of flexible metal deck or timber roof diaphragms can substantially affect the lateral seismic response of single-storey buildings; as such, several modifications were introduced to account for this behaviour. One of these changes is the introduction of new empirical expressions for the period T_a. For single-storey steel building structures with metal roof deck diaphragms and vertical steel bracing, the new equation reflecting period lengthening due to diaphragm flexibility is:

$$T_a = 0.035 h_n + 0.004 L \tag{2}$$

where L is the diaphragm span between adjoining vertical braced frames (in meters). For these structures, the fundamental period T_1 from dynamic analysis can also be used for T_a, except that T_a is limited to 1.5 times the value obtained from Eq. (2). As in previous NBCC editions, the upper limit on T_a does not apply for drift calculations; a reduced seismic load V_Δ obtained from Eq. (1) with the period $T_a = T_1$ is permitted to be used to calculate drifts when T_1 exceeds 1.5 times the value from Eq. (2). For regular buildings with vertical bracing at the diaphragm ends as shown in Fig. 1, the period T_1 can be estimated from [3, 4]:

$$T_1 \approx 2\pi \sqrt{\frac{W}{g}\frac{(\Delta_B + 0.76\Delta_D)}{V}} = T_B \sqrt{1 + 0.76\frac{\Delta_D}{\Delta_B}} \quad \text{, where: } T_B = 2\pi\sqrt{\frac{W}{g K_B}} \tag{3}$$

where Δ_B and Δ_D are respectively the lateral deformation of the vertical bracing and the relative in-plane deformation of the roof diaphragm under uniformly distributed static load V/L (see Fig. 1b), g is the acceleration due to gravity, T_B is the fundamental period of the structure assuming rigid diaphragm conditions, and K_B is the total lateral stiffness of the vertical bracing elements ($\Delta_B = V/K_B$). Deformations Δ_B and Δ_D are recomputed with V_Δ and the total building drift, Δ_T, is obtained by multiplying (Δ_B+Δ_D) by $R_d R_o$ to obtain the total deflection including inelastic response. The lateral deflection Δ_T is then verified against the limits specified in the NBCC: 2.5% h_s for buildings of the normal importance category, 2% h_s for buildings of the high importance category, and 1% h_s post-disaster buildings (h_s is the storey height, it is equal to h_n for single-storey buildings).

In the NBCC, seismic inelastic deformations are expected to develop in the SFRS vertical bracing elements; whereas, the diaphragms are designed to remain essentially elastic to maintain structural integrity and to ensure that lateral loads are distributed among the vertical elements as intended in analysis and design. Diaphragms must therefore resist horizontal load effects corresponding to the probable yield strength of the SFRS vertical elements, which can be achieved by applying capacity design principles. For so-designed single-storey buildings with flexible roof diaphragms, past investigations of the structure's nonlinear seismic response have shown that while Δ_T can predict well the total lateral displacements including inelastic effects, inelastic deformations imposed on the vertical bracing system and in-plane shears and moments in the diaphragm can exceed the values predicted from linear seismic analysis used in design [6-8]. Provisions have also been introduced in the 2015 NBCC to account for these additional effects of roof diaphragm flexibility and ensure that the ductility demand on the vertical bracing elements remains within acceptable values and roof diaphragms have enough in-plane strength. These new requirements only apply when R_d > 1.5 and Δ_D/Δ_B > 0.5, i.e. when sufficient inelastic response is expected and roof diaphragm flexibility is influential. For the SFRS vertical elements, the magnified displacement demand on the vertical bracing system, $\Delta_{B,a}$, can be estimated by deducting from the structure total lateral displacement the contribution from the roof diaphragm:

$$\Delta_{B,a} = \Delta_T - R_o\Delta_D \tag{4}$$

In this equation, Δ_D is amplified by R_o because the lateral load on the building is expected to attain R_oV when Δ_T is reached, i.e., the load corresponding to the system resistance including overstrenth. In design, the actual probable lateral resistance of the SFRS, V_u, can generally be determined and Δ_D may be multiplied by V_u/V_Δ rather than R_o in Eq. (4) to better reflect the anticipated structure response. For instance, a value of V_u/V_Δ larger than R_dR_o would reveal substantial overstrength and elastic response for the vertical bracing, in which case the verification for magnified inelastic deformations would not be required. Conversely, the design would have to be modified if $\Delta_{B,a}$ in excess of the inelastic deformation capacity of the SFRS vertical elements was predicted. In that case, a possible corrective measure consists of improving the detailing at critical locations of the SFRS vertical bracing to accommodate the expected magnified inelastic deformations. Alternatively, the ductility demand on the vertical bracing can be kept within acceptable limits by designing the SFRS for higher seismic loads and a reduced ductility-related force modification factor, referred to herein as R_{dr}, is given in the NBCC for this purpose:

$$R_{dr} = \frac{(R_d + \Delta_D/\Delta_B)}{(1 + \Delta_D/\Delta_B)} \tag{5}$$

This expression is obtained by replacing R_d by R_{dr} in the calculation of Δ_T in Eq. (4) (i.e. $\Delta_T = R_{dr}R_o\Delta$) and solving for R_{dr} when $\Delta_{B,a} = R_d\Delta_{By}$, where Δ_{By} is the displacement at yield of the vertical bracing (= $R_o\Delta_B$). Using R_{dr} should limit the ductility demand on the vertical bracing to the code specified R_d value for the selected SFRS. If the ratio V_u/V_Δ is known, R_{dr} from Eq. (5) can be multiplied by $(V_u/V_\Delta)/R_o$ to account for the SFRS actual overstrength.

In capacity design, the roof diaphragm is designed for a lateral load corresponding to the probable resistance of the vertical bracing elements, V_u. For rectangular buildings with vertical bracing located along the perimeter walls, as studied herein, maximum shear in the roof diaphragm occurs at the diaphragm ends where it is bounded by the capacities of the bracing members in the end walls. Along the diaphragm span, in-plane shears and moments are also constrained by the yield strength of the vertical bracing system, but the values are higher than those predicted from static analysis due to the dynamic response of the diaphragm-bracing system. The 2015 NBCC requires that this dynamic magnification of diaphragm shears and moments be accounted for in the diaphragm design. This can be done by applying dynamic amplification factors to values determined from static analysis. Values of amplification factors have been proposed from nonlinear dynamic analysis of structures exhibiting different Δ_D/Δ_B ratios and ductility levels [9]. A second approach consists of using a modified response spectrum analysis of the SFRS in which the first mode response is scaled such that the first mode shear at the diaphragm ends is limited to the probable resistance of the vertical bracing [10]. Contributions from higher modes are included without adjustment. The method then assumes that only the first mode response is significantly affected by brace yielding and buckling while higher mode response is essentially elastic. The concept is similar to that proposed for reinforced concrete shear walls designed for plastic hinging at their bases [11]. This modal superposition approach is illustrated in the examples presented in Section 3.

The NBCC has provisions for stability (P-delta) and in-plane torsional effects on seismic response. P-delta effects are discussed in the next section. For single-storey buildings with flexible roof diaphragms, resistance to in-plane torsion by the vertical bracing elements oriented perpendicular to the loading direction is limited and is generally ignored. Lateral loads are then generally distributed based on the tributary area concept. However, minimum accidental mass eccentricity corresponding to 5% of the building dimension perpendicular to earthquake loading must still be considered. For symmetrical buildings with identical bracing elements along both end walls, as shown in Fig. 1 and discussed herein, each bracing line is then designed for 55% of *V*.

2.2 CSA S16-14 Seismic Design Provisions

In Canada, steel SFRSs for which R_d in the NBCC is 1.5 or greater must be designed and detailed in accordance with special seismic provisions included in the CSA S16-14 Standard [12]. For steel concentrically braced frames (CBFs), three categories are defined in the NBCC: Type MD (moderately ductile), with R_d = 3.0, Type LD (limited ductility), with R_d = 2.0, and Type CC (conventional construction), with R_d = 1.5. For all three systems, R_o = 1.3. Although stricter design and detailing provisions apply for Type MD braced frames, the system is generally preferred due to the lower specified seismic loads (higher R_d factor). This article is therefore limited to this CBF category. For this system, seismic energy dissipation is expected to develop through inelastic response of the bracing members and strict compliance to capacity design is prescribed in CSA S16 to achieve this behaviour. The braces are therefore selected first in the design process and attention is devoted to selecting the most effective bracing members and brace configuration that minimize the forces that will be imposed to the other SFRS components during a strong earthquake.

In CSA S16, braces in Type MD CBFs must be arranged and proportioned so that the lateral resistance provided by tension acting braces along any bracing line is similar in both opposite directions to mitigate non symmetrical response due to brace compressive strength degradation in the post-buckling range. X-bracing is commonly adopted to satisfy this requirement. With this configuration, the effective length of the compression brace is reduced to half the brace full length due to in-plane and out-of-plane restraints provided by the tension-acting brace at the intersection points. In CSA S16, Type MD CBFs are permitted to be designed as tension-compression (T/C) or tension-only (T/O) systems. For the former, the lateral load is shared between compression and tension acting braces and the braces are sized to resist compression induced axial loads. In T/O bracing, the contribution of the compression braces is neglected and braces are selected to resist in tension the entire applied lateral load. T/O bracing is generally more effective when braces are slender (braces are long and seismic loads are low). All braces must also satisfy an upper limit on global slenderness, $KL/r \leq 200$, to achieve minimum energy dissipation capacity together with stringent slenderness limits for cross-section elements to delay local buckling in plastic hinges forming during buckling.

The braces must have factored axial resistances equal to or greater than member forces induced by NBCC seismic loads *V* plus in-plane torsion and stability effects, as discussed in the previous section. In CSA S16, stability effects include the effects from initial out-of-plumbness, inelasticity effects, and P-delta effects. The former two are accounted for by means of notional horizontal loads *N* equal to 0.5% of the total concomitant gravity load contributed by the level under consideration. For single-storey buildings, the concomitant gravity load is the total roof dead load plus 25% of the roof snow load. For rectangular buildings with perimeter vertical bracing studied herein, the total lateral load along each end wall is therefore 55% *V* plus 50% *N*. Lateral load induced forces are then multiplied by $(1 + \theta)$ to account for P-delta effects, where θ is the stability coefficient corresponding to the ratio between the storey shear required to resist the P-delta overturning moment and the structure probable storey shear strength. For symmetrical single-storey buildings, θ can be estimated from:

$$\theta = \frac{\left(\Sigma C_{\mathrm{f}}\right)\left(\Delta_{\mathrm{B,a}} + \Delta_{\mathrm{T}}\right)/2}{\left(R_{\mathrm{o}} V + N\right) h_{\mathrm{n}}} \tag{6}$$

where ΣC_{f} is the sum of the concomitant column axial loads due to the same concomitant roof gravity loads as used for notional loads. Displacements $\Delta_{\mathrm{B,a}}$ and Δ_{T} are as defined in the previous section. In Eq. (6), it is assumed that the roof gravity load is displaced by the average roof lateral displacement and the SFRS has a minimum dependable total lateral resistance equal to $R_{\mathrm{o}}V$ plus the notional load. In NBCC, it is permitted to ignore P-delta effects when θ is less than 0.10.

Once the braces are selected, the roof diaphragm is designed to resist in-plane forces induced by gravity loads plus the lateral load V_{u} that will develop when the braces reach their probable resistances in tension (T_{u}) and compression (C_{u}) given by:

$$\begin{aligned} T_{\mathrm{u}} &= A R_{\mathrm{y}} F_{\mathrm{y}} \\ C_{\mathrm{u}} &= \frac{1.2\, A R_{\mathrm{y}} F_{\mathrm{y}}}{\left(1+\lambda_{\mathrm{y}}^{n}\right)^{1/n}} \le A R_{\mathrm{y}} F_{\mathrm{y}} \quad \text{where: } \lambda_{\mathrm{y}} = \frac{KL}{r}\sqrt{\frac{R_{\mathrm{y}} F_{\mathrm{y}}}{\pi^2 E}} \end{aligned} \tag{7}$$

In these equations, A is the brace cross-section area, $R_{\mathrm{y}}F_{\mathrm{y}}$ is the probable brace yield strength, KL/r is the brace effective slenderness, and E is the Young's modulus. In case of CBFs exhibiting large overstrength, the NBCC and CSA S16 allow to limit the force V_{u} to the force V obtained from Eq. (1) with $R_{\mathrm{d}}R_{\mathrm{o}} = 1.3$. This upper limit implies that non-yielding SFRS components possess a minimum overstrength of 1.3 to resist elastic seismic loads corresponding to $R_{\mathrm{d}}R_{\mathrm{o}} = 1.0$. All other components along the lateral load path, including brace connections, beams, columns, anchor rods, and foundations must also be designed for forces corresponding to C_{u} and T_{u}. The brace post-buckling resistance, $C'_{\mathrm{u}} = 0.2AR_{\mathrm{y}}F_{\mathrm{y}}$ ($< C_{\mathrm{u}}$), must also be considered if it creates higher force demands on the SFRS components.

2.3 Steel deck diaphragm

The method proposed by the Steel Deck Institute [13] has now been adopted in the AISI S310-13 North American Standard for the design of steel deck diaphragms [14]. In that method, the diaphragm nominal shear strength S_{n} depends on the characteristics of the deck panels (deck profile, steel thickness, and steel grade) and the type and spacing of the connections of the deck panels. Strength can be governed by failure of the connections (S_{nf}) or panel shear buckling (S_{nb}). For the former, values of S_{nf} are determined for connection failure in the interior and edge panels. Failure in the corner connections is also considered. Factored shear resistances are then obtained by multiplying nominal strength values S_{n} by the appropriate resistance factors ϕ_{d}. Connection limit states generally control diaphragm design and ϕ_{d} in Canada is equal to 0.50 if welded connections are used or 0.60 if screws or other mechanical fasteners are selected for both the side-lap and frame connections.

3. Building Examples

3.1 Building design

Application of the Canadian seismic design provisions is illustrated for the steel structure shown in Fig. 2. The building has an aspect ratio of 1.5 in plan and is 8.4 m tall. Lateral loads are resisted by the roof diaphragm and two-bay X-braced frames of the Type MD category placed along each exterior wall. The roof steel deck panels are 914 mm wide x 38 mm deep and have a trapezoidal profile consisting of 6 flutes spaced at 152 mm o/c. The panels are made from ASTM A653 steel with $F_{\mathrm{y}} = 230$ MPa and $F_{\mathrm{u}} = 310$ MPa. Columns and braces are square hollow structural shapes (HSS) conforming to ASTM A1085 with a specified yield strength of 345 MPa. The roof framing consists of 12.4 m long open web steel joists supported on steel I-beams. The total roof dead load is 1.0 kPa. The building walls are composed of stiff precast concrete panels with a dead load w_p of 3.62 kPa. The panels are supported on the foundations and extend 500 mm above the roof.

Seismic design and response in the N-S direction is examined as seismic loading in that direction are expected to induce maximum diaphragm deformation effects. Both the T/C and T/O bracing design approaches are studied to investigate possible effects on the SFRS design and response. The structure is located on a class C site in two populated cities of Canada: Vancouver, BC, and Montreal, QC. Vancouver is located in South-West British Columbia, along the Pacific west coast. This is an active seismic region where earthquakes from three different tectonic sources contribute to the hazard: shallow crustal, subduction intraslab, and subduction interface earthquakes. These seismic conditions are similar to those prevailing in major cities such as Seattle and Portland in the Northwestern U.S. The city of Montreal is located in eastern Canada, a region of moderate seismicity where crustal earthquakes having their energy concentrated in high frequencies are expected. The seismicity in Montreal is representative of other populated area of eastern North America such as Boston and New York. UHS accelerations for Vancouver and Montreal are given in Fig. 3a.

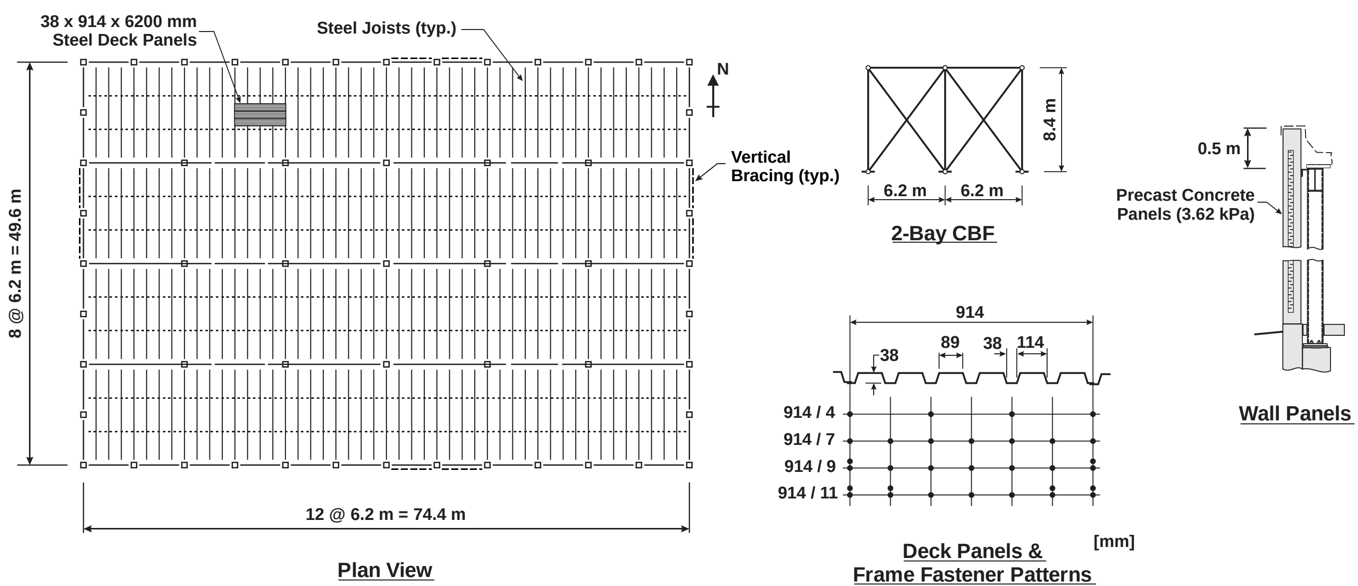

Fig. 2 – Building geometry.

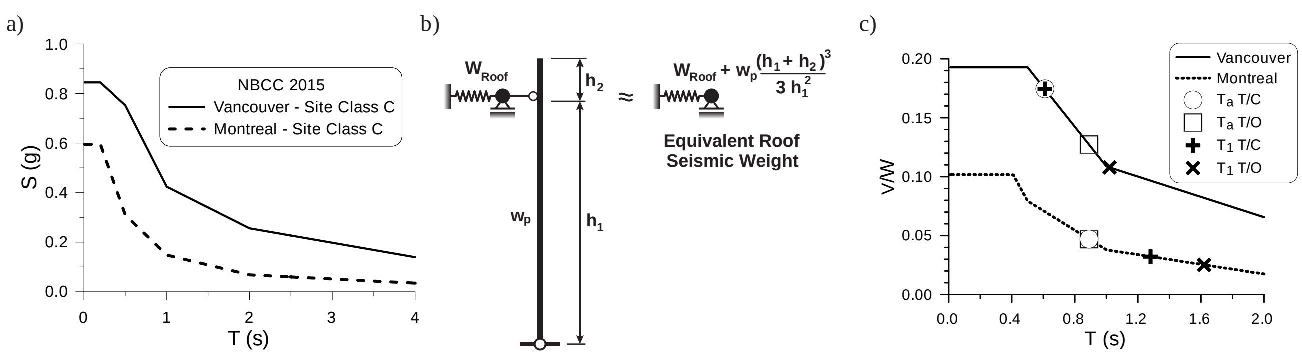

Fig. 3 – a) UHS at the sites; b) Calculation of roof seismic weight including contribution of wall panels; and c) Periods T_a and T_1 and resulting design seismic load.

For the calculation of V in Eq. (1), the building was assumed to be of the normal importance category with I_E = 1.0. As discussed, the factor M_V is taken equal to 1.0 for single-storey buildings. For Type MD CBFs, R_d = 3.0 and R_o = 1.3. Other design values and building properties are given in Table 1 for each site and braced frame configuration. The seismic weight W includes the roof dead load (1.0 kPa) plus 25% of the roof snow load w_s at the site (given in the table). Wall panels of the two E-W walls perpendicular to seismic loading must be laterally braced at the roof level and a portion of their weight is included in W, as described in Fig. 3b. Wall panels parallel to seismic loading are assumed to have sufficient in-plane shear strength to resist the inertia forces they would induce in the N-S direction. Their weight is therefore not considered in the value of W given in Table 1.

In Eq. (2), the period is calculated with h_n = 8.4 m and L = 74.4 m, which gives T_a = 0.59 s. A first design trial was performed with this period value and the period T_1 of that trial structure was determined using Eq. (3). Subsequent design iterations were performed with $T_a = T_1$, without exceeding the upper limit T_a = 1.5 x 0.59 s = 0.89 s, until convergence was reached. The values obtained in the last iteration are reported in Table 1 and the final T_a and T_1 values are indicated on the *V/W vs T* plots of Fig. 3c. At both sites, T/O bracing required smaller braces than T/C bracing, which resulted in more flexible buildings having longer periods T_1. Smaller braces were also needed in Montreal due to the lower design spectrum, which also led to longer T_1 values. Except for the building with stiffer T/C bracing in Vancouver for which $T_1 = T_a$, the upper limit on T_a governed (T_1 > 0.89 s). In previous NBCC editions, the period T_a was limited to $0.05h_n$ = 0.42 s. As can be deduced from Fig. 3c, the new provisions for the design period in NBCC 2015 resulted in significantly lower design seismic loads for all four cases, especially for the structures located in Montreal. The values of V are given in Table 1, together with the notional loads N obtained from the total roof gravity loads (ΣC_f). Details of drift calculations used to evaluate P-delta effects are discussed in the next paragraph. As shown, for all four buildings, the stability coefficients θ are less than 0.1 and P-delta effects were therefore omitted in design.

The selected bracing members are given in Table 1. The members were selected to achieve minimum weight while satisfying limits on slenderness at the global (member) and local (cross-section) levels. As shown, the effective slenderness KL/r of the selected braces range from intermediate (= 114 for T/C bracing in Vancouver) to high (= 168 for T/O bracing in Montreal). These slenderness values were determined with an effective length equal to 0.5 times the o/c brace length minus the length of the end connections. In the table, $V_{u,B}$ is the lateral load when all braces along the two N-S walls reach their probable resistances C_u and T_u and $V_{u,1.3}$ is the upper limit on lateral load as obtained from Eq. (1) with R_dR_o = 1.3. As shown, all CBFs possess substantial overstrength ($V_{u,B} >> V$), which is mainly due to the difference between R_yF_y and F_y for HSS braces (460 *vs* 345 MPa) and, for T/C bracing, the large difference between T_u and C_u for slender braces. For the structure with T/O bracing in Vancouver, $V_{u,B}$ is smaller than $V_{u,1.3}$ and the maximum anticipated seismic load along each N-S bracing line is $V_{u,Wall} = V_{u,B}/2$ = 1035 kN. Beams and columns along the two braced lines were therefore designed for this lateral load in combination with the tributary gravity loads and the resulting lateral stiffness of the structure vertical bracing K_B is given in the table. For the roof diaphragm of this structure, the design maximum factored shear at the diaphragm ends was taken as: $S_{f,max}$ = 1035 kN/49.6 m = 20.9 kN/m. For the Montreal building with T/C bracing, $V_{u,B}$ exceeds $V_{u,1.3}$ and design forces for the SFRS capacity protected elements may be limited to those induced by $V_{u,1.3}$. In-plane accidental torsion and notional loads must be considered when using $V_{u,1.3}$, and $V_{u,Wall}$ is therefore equal to 0.55 $V_{u,1.3}$ + 0.5 N. $V_{u,B}$ also exceeds $V_{u,1.3}$ for the T/C bracing in Vancouver and T/O bracing in Montreal. For those two structures, however, $V_{u,Wall}$ due to $V_{u,1.3}$ plus torsion and notional load effects is larger than the load corresponding to the brace probable resistances ($V_{u,B}/2$) and the latter was therefore used for the design of the roof diaphragm and the CBF beams and columns.

For all structures, the roof diaphragm was designed for the shear $S_{f,max}$ at the diaphragm ends. For simplicity in this study, the same design was adopted over the entire roof area. In practice, the design along the diaphragm span is adjusted to match the shear force demand; this aspect is discussed later. In the study, #12 self-drilling screws were assumed for the side-lap connections. Hilti HSN 24 power actuated frame fasteners were considered for 0.76 and 0.91 mm thick deck panels and 19 mm puddle welds were adopted when 1.21 mm thick steel was required. Table 1 gives the required deck steel thickness (e.g., 1.21 mm), the frame fastener pattern (e.g. 11/7 – see Fig. 2) and spacing of the side-lap connectors (e.g., 91 mm). The resulting diaphragm in-plane shear stiffness G' values are given in the table. As shown, a much stronger and stiffer diaphragm is needed for the structure with T/C bracing in Vancouver. Moderate values are obtained for the Vancouver building with T/O bracing whereas much lighter diaphragm designs can be used in Montreal. The perimeter beams along the E-W walls were designed to resist compression axial loads from in-plane diaphragm moments induced by lateral seismic loads corresponding to the lesser of $V_{u,B}$ or $V_{u,1.3}$, as applicable. The diaphragm in-plane moment of inertia I_D contributed by the E-W perimeter beams and the steel deck panels is given in the table.

The seismic load V_Δ, as determined with the actual period T_1 and the resulting deflections Δ_B, Δ_D, and Δ_T are given in Table 1. As shown, V_Δ is less than V for all buildings except for the one with T/C bracing in Vancouver. For all structures, the total drift including inelastic response, Δ_T, satisfies the NBCC limit of 2.5% h_n. Due to the higher diaphragm flexibility of the structures in Montreal, the Δ_D/Δ_B values at this location are higher than in Vancouver. For the T/O bracing in Vancouver, Δ_D/Δ_B is even lower than the 0.5 limit below which diaphragm flexibility effects on vertical bracing ductility demands and diaphragm forces can be ignored.

Table 1: Design values and SFRS properties

Site Bracing design	Vancouver T/C	Vancouver T/O	Montreal T/C	Montreal T/O
w_s (kPa)	1.64	1.64	2.48	2.48
W (kN)	6997	6997	7772	7772
T_1 (s)	0.61	1.02	1.28	1.62
T_a (s)	0.61	0.89	0.89	0.89
V (kN)	1221	893	368	368
ΣC_f (kN)	5203	5203	5978	5978
N (kN)	26	26	30	30
θ	0.022	0.054	0.052	0.074
Bracing members	HSS114x114x6.4	HSS89x89x4.8	HSS89x89x4.8	HSS76x76x3.2
KL/r	114	146	146	168
$V_{u,B}$ (kN)	3866	2070	2070	1157
$V_{u,1.3}$ (kN)	3661	2679	1104	1104
$V_{u,Wall}$ (kN)	1933[1]	1035[1]	622[2]	579[1]
K_B (kN/mm)	122.8	36.6	71.9	22.3
$S_{f,max}$ (kN/m)	39.0	20.9	12.5	11.7
Deck panels[3]	1.21-11/7-91	0.76-9/7-111	0.76-4/7-119	0.76-4/7-141
G' (kN/mm)	35.3	17.3	4.05	4.01
I_D (10^{12}mm^4)	29.9	20.6	19.2	18.0
V_Δ (kN)	1221	754	250	196
Δ_B (mm)	10.0	20.6	3.5	8.8
Δ_D (mm)	7.6	9.2	11.9	9.5
Δ_T (mm) (%h_n)	68 (0.81)	116 (1.38)	60 (0.72)	71 (0.85)
Δ_D/Δ_B	0.76	0.45	3.43	1.07
V_u/V_Δ	3.17	2.75	8.28	5.90
$(V_u/V_\Delta)\Delta_D$ (mm) (%h_n)	24 (0.29)	-	-	-
$\Delta_{B,a}$ (mm) (%h_n)	44 (0.53)	80 (0.96)[4]	14 (0.16)[4]	34 (0.41)[4]

[1]Governed by the probable resistance of the bracing members ($V_{u,B}$)
[2]Governed by the upper limit on seismic loads corresponding to R_dR_o = 1.3 ($V_{u,1.3}$)
[3]Steel thickness (mm) - frame fastener pattern - side-lap fastener spacing (mm)
[4]$\Delta_{B,a} = R_dR_o\,\Delta_B$ (limited by elastic force demand at period T_1)

In Table 1, values of V_u/V_Δ were calculated using the lateral load $V_{u,B}$. For the building with T/C bracing in Vancouver, Δ_{Ba} is obtained from Eq. (4) using: $\Delta_{Ba} = \Delta_T - (V_u/V_\Delta)\Delta_D = 68 - 24 = 44$ mm. This is only 13% more than $R_dR_o\Delta_B$ = 39 mm due to the large system overstrength (V_u/V_Δ = 3.17) and the limited diaphragm flexibility (Δ_D/Δ_B = 0.75). For the Vancouver building with T/O bracing, Δ_D/Δ_B is less than 0.5 and amplification of vertical bracing deformations due to diaphragm response may be ignored. In that case, total deformations of the bracing bents and roof diaphragms can be taken equal to $R_d R_o \Delta_B$ = 80 mm (0.95% h_n) and $R_d R_o \Delta_D$ = 36 mm (0.43% h_n). If the approach used for T/C bracing was used, $(V_u/V_\Delta)\Delta_D$ would be equal to 2.75(9.2) = 25 mm (0.30% h_n) and Δ_{Ba} would be Δ_T - $(V_u/V_\Delta)\Delta_D$ = 116 – 25 = 91 mm (1.08% h_n). In Montreal, due to the large differences between the periods T_a used for strength design and the actual periods T_1, the ratios V_u/V_Δ exceed $R_d R_o$ = 3.9 for both bracing designs, indicating that the bracing members will unlikely develop their full probable resistance T_u and C_u under the design earthquake level. For these structures, Δ_{Ba} is therefore expected to remain close to $R_d R_o \Delta_B$, i.e. the deflection under the elastic force $R_d R_o V_\Delta$ at the period T_1. For all structures, the value of Δ_{Ba} is much less than drifts that can cause failure in type MD CBFs and there was no need to use a reduced R_d value in design

3.2 Building seismic response

Nonlinear response history analysis (NLRHA) was performed for each structure. For each site, representative earthquake ground motions were selected and scaled as described in [15]. For Vancouver, a suite of 11 ground motions was used for each of the three contributing earthquake sources, i.e. crustal, intraplate, interface earthquakes. For Montreal, the ground motion ensemble contained a suite of 5 simulated ground motions from smaller (M6.0) earthquakes at close distance and 6 simulated ground motions from larger (M7.0) events at larger distances. The analyses were performed using the OpenSees platform. The braces were modeled using force-based beam-column elements and the Giuffre-Menegotto-Pinto steel (Steel02) material to reproduce brace inelastic buckling and tension yielding [16]. Beams and columns were modelled using elastic beam elements. Similarly, elastic beam elements deforming in flexural and shear were used for the diaphragm. Geometric non-linearity was considered in the analysis and damping corresponding to 3% of critical in first mode was specified. Values of peak response parameters are given in Table 2. For each location, the largest mean values among the mean values of each motion suite were retained. Hysteretic and time history responses under selected ground motions inducing demands close to mean values are presented in Fig. 4.

Table 2: Mean value of peak response parameters

Site Bracing design	Vancouver T/C	Vancouver T/O	Montreal T/C	Montreal T/O
Δ_B (% h_n)	0.61 (1.15)[1]	0.95 (0.99)	0.26 (1.63)	0.45 (1.10)
Δ_D (% h_n)	0.35 (1.21)	0.08 (0.19)	0.92 (1.67)	0.74 (1.68)
Δ_T (%h_n)	0.90 (1.11)	1.01 (0.73)	1.08 (1.50)	1.06 (1.25)
$S_0/S_{f,max}$	0.96 (0.96)	1.00 (1.00)	1.11 (1.11)[2]	0.90 (0.90)
$S_{L/4}/S_{f,max}$	0.66 (1.03)	0.61 (0.95)	0.87 (1.23)[2]	0.69 (0.95)
M_D (MN-m)	41400 (1.01)	21800 (0.96)	14900 (1.18)[2]	11900 (0.93)

[1] Values in brackets are the ratios between NRHA results and predicted values.
[2] Values in brackets for $S_0/S_{f,max}$, $S_{L/4}/S_{f,max}$, and M_D respectively reduce to 0.85, 0.97, and 0.89 if $V_{u,Wall}$ = 809 kN is used.

The two buildings in Vancouver generally behaved as expected in design. In Fig. 4a, for the T/C bracing, the braces buckled in compression and nearly developed their yield tensile strength T_u, and the CBF probable resistance $V_{u,B}$ was therefore reached. For the T/O bracing in Fig. 4b, more pronounced brace inelastic response is observed, as was anticipated due the lower system overstrength, and the force $V_{u,B}$ was attained several times during the earthquakes. In Table 2, NLRHA drifts for the T/C bracing CBF in Vancouver were generally well predicted, although consistently underestimated. The deformations of the T/O bracing bents were correctly estimated when neglecting the amplification due to diaphragm response, as allowed in view of the low Δ_D/Δ_B ratio, but Δ_T was over-estimated compared to NLRHA (1.38% *vs* 1.01 h_n), likely because the roof diaphragm experienced smaller than anticipated deformations in NLRHA (0.08 *vs* 0.43% h_n – also see Fig. 4b). A better match is obtained when considering $(V_u/V_\Delta)\Delta_D$ = 0.30% h_n for the diaphragm but the error on Δ_T is still present and will require further investigation. In Table 2, the shear demand from NLRHA reaches $S_{f,max}$ along the end walls, i.e. the value based on $V_{u,B}$ that was considered in design. As shown in the $S/S_{f,max}$ time histories of Figs. 4a and 4b, diaphragm shears at the ends (S_0) and at the quarter of the diaphragm span ($S_{L/4}$) are generally in phase and vary in accordance with first mode response. NLRHA values of ($S_{L/4}$) in Table 2 are however higher than $0.5S_0$, suggesting dynamic amplification at $L/4$ compared to static shear distribution.

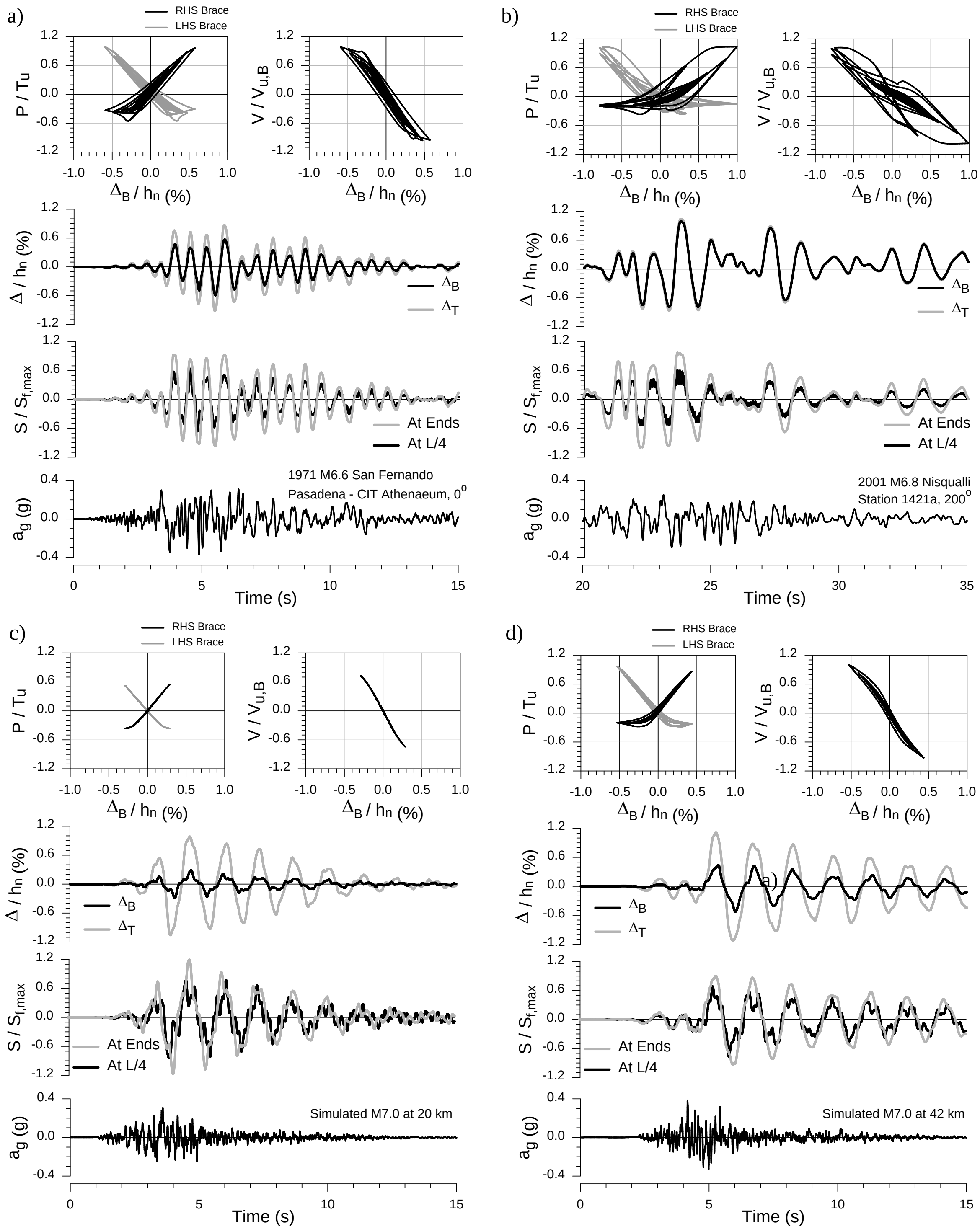

Fig. 4 Hysteretic responses of the braces and bracing bents and time history response of drifts and diaphragm shears in the: a) & b) T/C & T/O bracing in Vancouver; c) & d) T/C & T/O b in Montreal.

As mentioned, dynamic magnification of diaphragm response can be predicted using a modified response spectrum analysis. This analysis can be performed using the simple beam model with flexible supports shown in Fig. 5a. For symmetrical buildings as discussed herein, even numbered modes do not contribute to the response and only odd numbered modes need to be considered. Diaphragm in-plane shears contributed by the first mode, $V^1(x)$, and by all relevant subsequent modes, $V^{3+}(x)$, as determined from response spectrum analysis, are then combined using:

$$V(x) = \alpha V^1(x) + V^{3+}(x) \qquad \text{,where: } \alpha = \frac{V_{\text{u,Wall}} - V^{3+}(0)}{V^1(0)} \tag{8}$$

An absolute sum is adopted assuming that both modal values can reach their peak simultaneously and the first mode contribution is multiplied by a scaling factor α determined such that the total shear at the diaphragm ends ($\alpha V^1 + V^{3+}$) is equal to $V_{\text{u,Wall}}$. The approach is illustrated in Fig. 5b for the diaphragm of the Vancouver building with T/C bracing. For this structure, although diaphragm response is limited (Δ_D/Δ_B = 0.76), NLRHA shears deviate from the linear variation from static analysis and the mean demand is well predicted by the proposed approach. The method is also used for moments in the diaphragm, with the first mode moments being multiplied by the same α factor determined with end shears. In Table 2, the method gives good results for diaphragm shears at L/4 and diaphragm moments at the building mid-length, M_D, for both Vancouver buildings. Moments M_D from NLRHA for these two structures are equal to 1.16 and 1.18 times the static values.

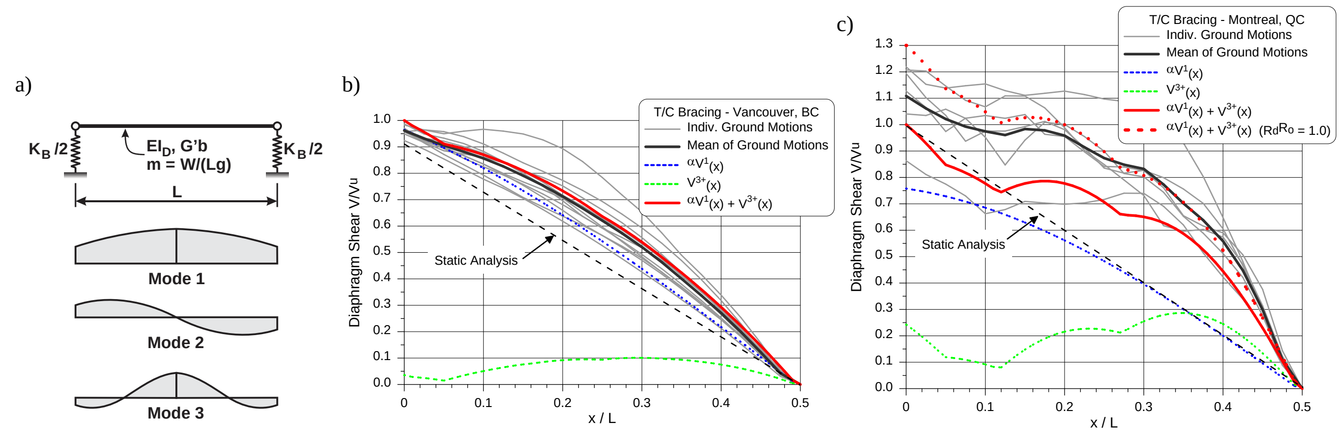

Fig. 5 – Prediction of diaphragm shears using modal response spectrum analysis: a) Mode shapes; b) T/C bracing in Vancouver; and c) T/C bracing in Montreal.

As shown in Figs. 4c and 4d, because of their high overstrength, both structures in Montreal exhibit nearly linear seismic response as only brace buckling was observed. In both cases, a large portion of the total storey drift is contributed by the roof diaphragms, especially for the structure with stiffer and stronger T/C bracing, as was predicted in design. However, in Table 2, deformations of the bracing bents and diaphragms were generally underestimated at the design stage. The fact that the structures remain almost linear and only 3% damping was specified in NLRHA, compared to 5% assumed in design, may have contributed to this difference. For T/C bracing, diaphragm shears at the end walls exceed the value corresponding to $V_{\text{u,Wall}}$. This is possible because $V_{\text{u,Wall}}$ used to design the diaphragm was limited by the seismic force obtained with R_dR_o = 1.3, as permitted in codes in anticipation that roof diaphragms and other non-yielding SFRS elements possess sufficient overstrength to resist the expected higher force demand corresponding to R_dR_o = 1.0. The structure with T/O bracing attracted lower seismic forces due to its longer period and more pronounced nonlinear response. The time histories of diaphragm shear in Figs. 4c and 4d reveal that shears along the diaphragm span are highly influenced by third and higher mode response. The modified response spectrum approach (Eq. 8) can account for this behaviour, as demonstrated in Fig. 5b for the T/C bracing. In the figure, the method was first applied using α computed with $V_{\text{u,Wall}}$ = 622 kN, i.e. the seismic force obtained with R_dR_o = 1.3. As shown, although codes allow using this design force level in case of overstrong CBFs, higher shears can develop in the structure under design earthquake levels. When re-applying the method with $V_{u,Wall}$ = 809 kN corresponding to R_dR_o = 1.0, shears from response spectrum analysis envelope well the shear demand from NLRHA. In Table 2, the method can provide good estimates of $S_{L/4}/S_{f,max}$ and M_D for the two structures in Montreal.

4. Conclusions

Seismic design of single-storey steel buildings with flexible roof diaphragms according to Canadian codes has been presented with focus on the new provisions that have been implemented in 2015 for the design period, increased ductility demand in the vertical bracing and dynamic magnification of diaphragm forces. The design procedure was applied to four structures designed with two different bracing systems and located at two different sites in Canada. In all cases, the new design period resulted in reduced design seismic loads. Nonlinear response history analysis of the prototype structures showed that the structures performed as intended in design. Drift estimates were underestimated in some of the cases, which will require further investigation. Diaphragm in-plane shears and moments could be reliably predicted using a modified response spectrum analysis method.

5. Acknowledgements

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