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SEISMIC INDUCED FLOOR ACCELERATIONS AND DIAPHRAGM FORCES FOR BUCKLING RESTRAINED BRACED FRAMES

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Abstract
Floor and roof diaphragms are essential part of seismic force resisting systems in building structures. They must be designed to withstand and transfer forces that are induced by floor acceleration in an event of strong earthquakes. Reconnaissance reports of major earthquakes have shown that diaphragms structural integrity is one of the key factors to obtain satisfactory seismic performance of structural systems. This article presents a comparison between Canadian and U.S. building code provisions regarding diaphragm seismic design forces and a case of 10-storey buckling-restrained braced frame is elaborated as an example. Despite the general similarities, building codes in the two countries prescribed significantly different design values for the case studied. The results of nonlinear time history analysis conducted on 3 to 15-storey code-conforming ductile steel braced frames are then compared to the Canadian code (NBCC) specified design values. These analyses show that diaphragm forces exceed the design values by a significant margin and diaphragms overload can be repeated many times during a typical design level earthquake. Current peak floor accelerations defined in NBCC for the design of non-structural elements and building components is also shown to be overlay conservative, especially at the roof level. Large diaphragm forces are generated as a result of time delays between storey shear forces in adjacent storeys with maximum values occurring during elastic phases of the response. This delay is strongly related to the ground motion intensity and frequency content. Based on the observations made in this study, possible avenues are proposed to improve design provisions for peak floor accelerations and diaphragm inertia forces.

Keywords: Diaphragm; Braced frame; Floor acceleration; Storey resistance; Non-structural elements.

1 Introduction

Roof and floor diaphragms in building structures serve to collect and transfer lateral loads to the vertical elements of the lateral load resisting systems. They also horizontally connect together the structural elements of the gravity load and lateral load framing systems so that lateral bracing is provided to the gravity system and all lateral load resisting elements contribute to the structure in-plane torsional stiffness and strength. Diaphragms are also used to transfer in-plane horizontal forces at levels where there is an offset discontinuity in the vertical elements resisting lateral loads. Building codes include design provisions to ensure that roof and floor diaphragms can safely achieve these functions without failure.

Lateral loads acting on buildings subjected to ground motions from earthquakes consist of horizontal inertial forces resulting from the building dynamic response to ground motions. At every level, these forces are proportional to horizontal accelerations and design forces should therefore be related to peak floor accelerations that are expected under the design level earthquake. Peak horizontal accelerations are also used to determine design seismic forces for non-structural elements and components of buildings. Examination of code design provisions reveals that differences exist between the acceleration levels that are considered for the design of diaphragms and those assumed for non-structural elements and components. Significant differences in design provisions are also found when comparing different national codes. Recent numerical and experimental studies have indicated that peak floor accelerations assumed in design may deviate considerably from those considered in design, especially when the seismic resistance system responds in the nonlinear range [1-5].

Past studies on diaphragm inertia forces and floor accelerations focused on shear wall structures in which inelastic seismic deformations concentrate in flexure at the structure base. This paper examines floor acceleration demands and resulting horizontal inertia loads in seismic force resisting systems (SFRSs) that deform essentially in shear upon yielding. Prototype steel building structures in which earthquake lateral resistance is provided by buckling restrained braced frames (BRBFs) are selected for this purposes. BRB members are specially designed and constructed to exhibit stable hysteretic response through yielding in compression and tension. BRBFs can therefore be designed for reduced seismic loads and are therefore expected to undergo significant inelastic cyclic deformations during design level earthquakes, which may impact the diaphragm force demands. Design provisions for diaphragms and non-structural elements in the 2010 National Building Code of Canada (NBCC) [7] and the ASCE 7-10 standard in the United States [8-10] are first introduced and compared for a 10-storey BRBF structure. Peak floor accelerations obtained from nonlinear time history analysis (NTHA) performed on prototype BRBF buildings designed in accordance with the NBCC are presented and compared. The height of the buildings is varied and different seismic regions and local site conditions are considered to assess possible effects of these parameters on floor acceleration demands. Floor acceleration and storey shear time histories are examined to determine the origin of the observed floor inertia loads and propose potential avenues for design provisions that more closely reflect the observed structure response.

2 Code Design Provisions for Diaphragms and Elements and Components

2.1 NBCC Provisions

In the 2010 NBCC, floor and roof diaphragms must be designed to remain essentially elastic under forces induced by seismic ground motions. As a minimum, diaphragms must resist the larger of: 1) the forces due to the seismic design loads amplified to reflect the lateral load capacity of the seismic force resisting systems (SFRS), and 2) a force corresponding to the design base shear V divided by the number of storeys. For the former, the seismic design loads are those obtained from the equivalent static force procedure (ESFP) or dynamic response spectrum analysis (RSA), as used in design, and the SFRS lateral load capacity can be determined using the probable resistance of the yielding components of the SFRS, as specified in the applicable design standards. In the NBCC, the design base shear V is determined using a design spectrum $S(T_a)$ obtained from uniform hazard spectral ordinates specified for a probability of exceedance of 2% in 50 years and modified for local site conditions:

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

where T_a is the structure fundamental period used for design, M_V is a factor that accounts for higher mode effect on base shear, I_E is the importance factor, W is the seismic weight, and R_d and R_o are respectively ductility- and overstrength-related force modification factors. For tall moment frames and braced frames, V for strength design must not be less than the value computed at a period of 2.0 s. If the static equivalent force procedure is used, the seismic forces F_x at every levels are determined from:

$$F_x = (V - F_t) \frac{w_x h_x}{\sum_{i=1}^{n} w_i h_i} \qquad \text{where: } F_t = 0 \;\text{ if } T_a \le 0.7 \text{ s;} \tag{2}$$

$$F_t = 0.07 T_a V \le 0.25 V \;\text{ if } T_a > 0.7 \text{ s}$$

In this equation, F_t is a concentrated load applied at the roof level, and w and h are the seismic weights and elevations of the storey x. Response spectrum analysis is performed using the design spectrum S and the results are multiplied by I_E/R_dR_o to obtain design values. The analysis results must then be further adjusted such that the resulting base shear is not less than V from Eq. (1). For regular buildings, that adjustment may be performed with respect to 0.8 V instead of V. Forces F_x from response spectrum analysis can be determined using differences between storey shears in consecutive floors; however, it is recommended that they be taken equal to the floor absolute accelerations from spectral analysis times the seismic floor weights [9-10].

Diaphragm design forces are examined for the regular 10-storey steel buckling restrained braced frame building shown in Fig. 1a. In the NBCC, R_d and R_o are respectively equal to 4.0 and 1.2 for this SFRS. The building is assumed to be located on a site class C (soft rock or firm ground) in Victoria, British Columbia. Victoria is located along the Pacific west coast and is among the most seismically active and populated areas in Canada. The design spectrum S for this site is plotted in Fig. 1b. Other spectra shown in the figure will be described later. For this structure, $M_v = I_E = 1.0$. For braced frames, $T_a = 0.025\ h_n$ in the NBCC, where h_n is the building height (h_n = 40 m). Alternatively, the period T_1 from dynamic analysis can be used for T_a except that it cannot exceed 0.05 h_n = 2.0 s. In this example, the BRBFs were designed using both the ESFP and RSA methods and frame members were proportioned using the Canadian steel design standard CSA S16-09 [12]. The resulting two designs had periods T_1 = 2.5 and 2.8 s, respectively, and T_a = 2.0 s was used to determine $V = 0.0375\ W$ for both cases. The RSA results were adjusted such the base shear from analysis was equal to 0.8 $V = 0.03\ W$.

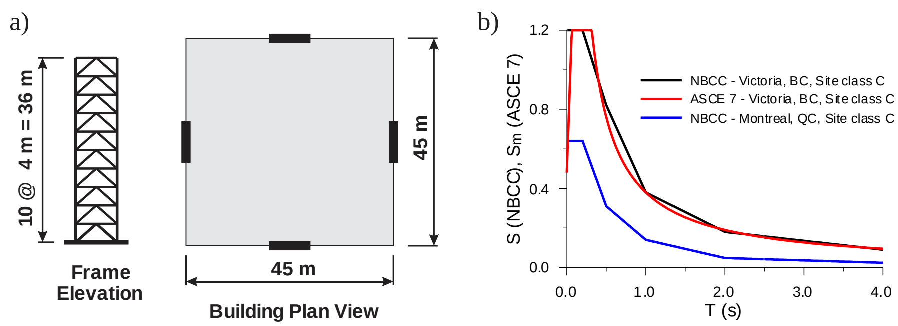

Fig. 1 – BRBF example building; b) Design spectra.

The NBCC diaphragm design forces are given in Fig. 2a for the structures designed using the ESFP and RSA analysis methods. As shown, to ease discussion and comparison, floor accelerations a_f are plotted instead of diaphragm forces; these accelerations being simply the diaphragm forces divided by the corresponding floor seismic weights w_x. For reference, note that the 2% in 50 years peak ground acceleration (PGA) for the Victoria site is 0.61 g. In the figure, forces $F_{x,u}$ correspond to the design seismic loads F_x amplified to reflect the actual frame lateral capacity V_u. For this example, storey shears V_u were determined at every level using the brace maximum probable resistances in compression and tension, as specified in CSA S16 standard. In this example, it was assumed that the brace cores were sized to exactly match the design seismic force demand, the core yield strength from coupon tests were used in brace design (R_y = 1.0), and the braces have tension and compression force adjustment factors ω = 1.4 and β = 1.1, respectively. These two factors account for strain hardening, friction and Poisson's effects that develop at large inelastic brace deformations. A resistance factor ϕ = 0.9 was also considered in brace core design. The forces F_x were then amplified by the ratio V_u/V_x = 1.63, where V_x is the storey shear corresponding to the forces F_x. For RSA, forces obtained from differences in amplified storey shears between consecutive levels ("from Shears") and those from floor absolute accelerations ("from Accel.") are plotted. For consistency, the second are also amplified by the ratio V_u/V_x. The second set of diaphragm design forces are obtained by dividing the design storey shears, V and 0.8V for the ESFP and RSA, by the number of storeys N = 10.

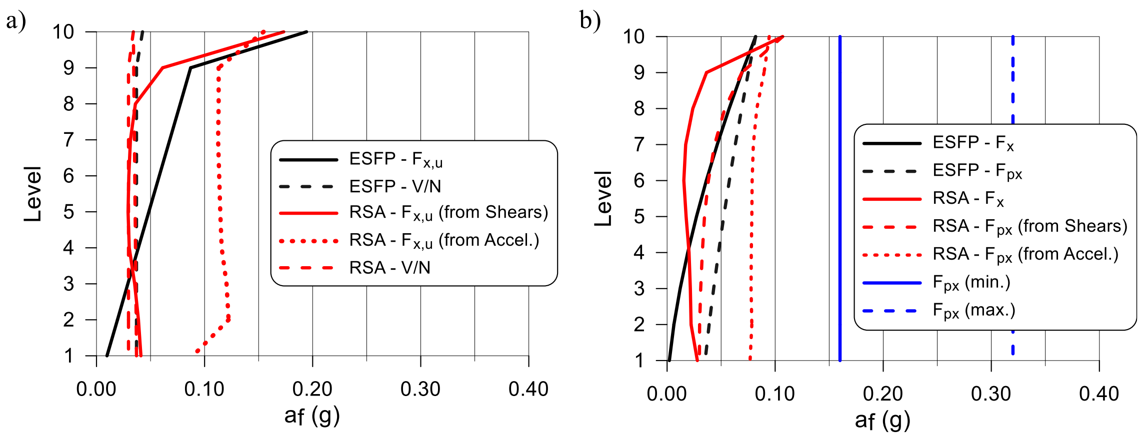

Fig. 2 – Design diaphragm forces in: a) NBCC 2010; b) ASCE 7-10

As shown, the V/N forces govern in the bottom two storeys when the ESFP is used. If RSA is adopted, the V/N forces no longer are necessary. When using RSA, inertia forces determined from the floor accelerations are significantly larger than those calculated from storey shears. It is noted that all methods but V/N approach give comparable high accelerations at the top (roof) level.

In NBCC, design horizontal forces for elements of structures, non-structural components and equipment are given by:

$$V_p = 0.3 F_a\, S_a(0.2)\, I_E\, S_p\, W_p \tag{3}$$

where $F_aS_a(0.2)$ is the design spectral acceleration at a period of 0.2 s including site effects (F_a = acceleration-based site coefficient), I_E is the importance factor, S_p is the horizontal force factor for the element or component, and W_p is the weight of the element or component. The term approximately corresponds to the peak ground acceleration at the structure base and the factor S_p is given by:

$$S_p = \frac{C_p\, A_r\, A_x}{R_p} \qquad \text{with: } A_x = 1 + 2 h_x / h_n \tag{4}$$

In this expression, C_p is the element or component factor, A_r is the element or component force amplification factor, A_x is the height factor, h_x is the height of level x, h_n is the building height, and R_p is the element or component response modification factor. Floor accelerations considered in this calculation can be taken as $0.3F_aS_a(0.2)\, A_x$, where $0.3F_aS_a(0.2)$ approximately corresponds to 60% of PGA. Hence, NBCC assumes floor accelerations linearly varying from 0.6 PGA at the structure base to 1.8 PGA at the roof level. For the 10-storey building example, $0.3F_aS_a(0.2)$ = 0.36 g, which gives floor accelerations varying from 0.432 to 1.08 g. This is significantly higher than the acceleration levels corresponding to the design diaphragm inertia forces in Fig. 2a.

2.2 ASCE 7-10 Provisions

In ASCE 7-10, diaphragms in multi-storey buildings must resist the inertial seismic forces that are taken equal to seismic loads F_x obtained from structural analysis. Forces F_x are typically determined using the equivalent static force procedure (ESFP):

$$F_x = \left(\frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \right) V \qquad \text{where: } k = 1 \qquad \text{for } T < 0.5 \text{ s;} \tag{5}$$

$$k = 1 + (T - 0.5)/2 \;\; \text{for } 0.5 \le T \le 0.5$$

$$k = 2 \text{ for } T > 2.5 \text{ s} \qquad \text{for } T > 2.5$$

, with the design base shear V given by:

$$V = C_s W \qquad \text{where: } \begin{aligned} C_s &= S_{DS}/(R/I_e) \\ &\le S_{D1}/T(R/I_e) \quad \text{if } T \le T_L \\ &\le S_{D1}\, T_L/T^2(R/I_e) \quad \text{if } T > T_L \\ &\ge 0.044 S_{DS}\, I_e \ge 0.01 \\ &\ge 0.5 S_1/(R/I_e) \quad \text{if } S_1 > 0.6 \end{aligned} \tag{6}$$

In these equations, S_{DS} and S_{D1} are the design spectral acceleration at short (0.2 s) and one second period. These values are equal to 2/3 of the risk targeted maximum credible earthquake (MCE_R) spectral values S_{MS} and S_{M1} that are based on a probability of exceedance of 2% in 50 years and adjusted for site effects. T_L is the long-period transition period, S_1 is the MCE_R spectral acceleration at 1 s for site class B, R is the SFRS force modification factor, I_e is the importance factor, and T is the structure fundamental period. Similar to the Canadian code, F_x can be determined using response spectrum analysis. In that case, the spectrum S_a corresponds to C_s in Eq. (6), without the last two lower limits on C_S, and the results must be adjusted such that the resulting base shear is not less than 0.85 the base shear V from Eq. (6). Also as in Canada, F_x from RSA should is preferably obtained from the computed floor absolute acceleration values rather than storey shears.

Diaphragms must also be designed to resist the inertia forces F_{px}:

$$0.2 S_{DS} I_e w_{px} \le F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px} \le 0.4 S_{DS} I_e w_{px} \tag{7}$$

where forces F_i correspond to forces F_x from structural analysis and w_{px} is the diaphragm tributary seismic weights (generally same as w_x). Minimum and maximum values of F_{px} are specified in Eq. (7). In ASCE 7, the design peak ground acceleration corresponds to 0.4 S_{DS}. Hence the lower and upper limits in Eq. (7) are forces induced by floor accelerations corresponding to 50% and 100% PGA. The lower limit on F_{px} generally governs for the most ductile systems with high R values that result in smaller values of F_x underestimating higher mode response [10].

ASCE 7-10 also contains minimum force requirements for non-structural elements and building components:

$$0.3 S_{DS} I_p W_p \le F_{px} = \frac{0.4 a\, S_{DS} W_p}{(R_p/I_p)} \left(1 + 2\frac{z}{h}\right) \le 1.6 S_{DS} I_p W_p \tag{8}$$

This force requirement is very similar to the one in NBCC given by Eq. (3), with the component amplification factor a corresponding to $C_p A_r$ and z being same as h_x. In this equation, the reference floor acceleration is 0.4 S_{DS}, i.e. the peak ground acceleration. This appears higher than the 60% PGA implied by

$0.3F_aS_a(0.2)$ in Eq. (3) but the two values are in fact consistent as S_{DS} is 2/3 of MCE_R values established for a 2% in 50 years probability.

Floor accelerations corresponding to minimum diaphragm design forces of ASCE 7-10 are presented in Fig. 2b for the 10-storey BRBF building example. For direct comparison, S_{MS} and S_{M1} of ASCE 7 were set equal to $S(0.2)$ and $S(1.0)$ NBCC values for Victoria and the importance factor $I_e = 1.0$ was considered. In Fig. 1b, the resulting ASCE 7 MCR_E spectrum compares well with the NBCC design spectrum and the design spectral ordinates were $S_{DS} = 0.8$ and $S_{D1} = 0.253$. In ASCE 7, an R factor of 8 is specified for BRBFs. The structure design was performed using both the ESFP and RSA methods. For the ESFP procedure, the design period T permitted in ASCE 7 was equal to 1.68 s and the resulting base shear V was equal to 0.0352 W, as governed by the minimum lateral force requirement ($0.044\ S_{DS}\ I_e$) in Eq. (6). For RSA, the results were adjusted to obtain a base shear equal to $0.85\ V = 0.0299\ W$. Coincidentally, these two values are close to NBCC design base shears, despite the differences in design periods and design spectral ordinates. In Fig. 2b, the diaphragm forces F_x from structural analysis using ESFP and RSA are comparable to those prescribed in Canada. One difference is that the forces in the U.S. are not amplified to account for the difference between factored and probable brace resistances. For this structure, the minimum force $F_{px} = 0.16\ w_x$ specified in Eq. (7) would control diaphragm design over the entire building height. This force exceeds NBCC value obtained from floor accelerations computed with RSA. As was the case for Canada, floor accelerations from Eq. (8) would be much higher, varying from 0.32 to 0.96 g from the structure base to the roof level.

3 Floor Accelerations in Buckling Restrained Braced Frames

3.1 Buildings Studied

A group of prototype steel buildings designed with BRBFs to compare the NBCC diaphragm design forces to the actual inertia force demand from NTHA. The structures have 3, 5, 7, 9, 13, and 15 storeys. The structure plan view and braced frame elevations are illustrated in Fig. 3. Two different locations were considered: Victoria, BC, as previously described, and Montreal, Quebec. The second location is situated in a moderately seismic active region of eastern North America where the anticipated seismic ground motions are expected to have their energy concentrated in the short period range (high frequencies). Site class C was considered at both locations and the design spectra are shown in Fig. 1b. For Victoria, structures on site class E (soft soil) were also studied. The design was performed using the ESFP in accordance with the provisions for NBCC 2010 and CSA S16-09 and the properties of the structures are given in Table 1. As shown, the structures in Montreal are designed for much lower seismic loads (V/W) and are laterally more flexible (longer periods) compared to those situated in Victoria for the same site conditions. Similar trends exist for the two site conditions in Victoria. Additional information on the design of the prototype buildings can be found in [13].

In Table 1, storey shears resistances V_y and V_u at the first level of the buildings are also given. The resistance V_y reflect the probable yield strength of the braces, excluding the strength increase at large deformations associated to the BRB strength adjustment factors ω and β. These effects are included in V_u values. As discussed, storey forces used to determine the diaphragm design forces reflect the as-built lateral capacity of the SFRS specified in the NBCC. In Fig. 4, these design forces are compared to the forces V/N for representative prototype structures. As indicated, the former set of seismic forces amplified for actual brace resistances governed in all cases except at the base of the taller buildings where V/N values are higher.

3.2 Analysis Results

Time history analyses were conducted using ensembles of site representative ground motions. 50th and 84th percentile values of the computed peak floor accelerations are given in Fig. 5 for the 3-, 9-, and 15-storey structures. The controlling NBCC design values are also shown in the figure for comparison.

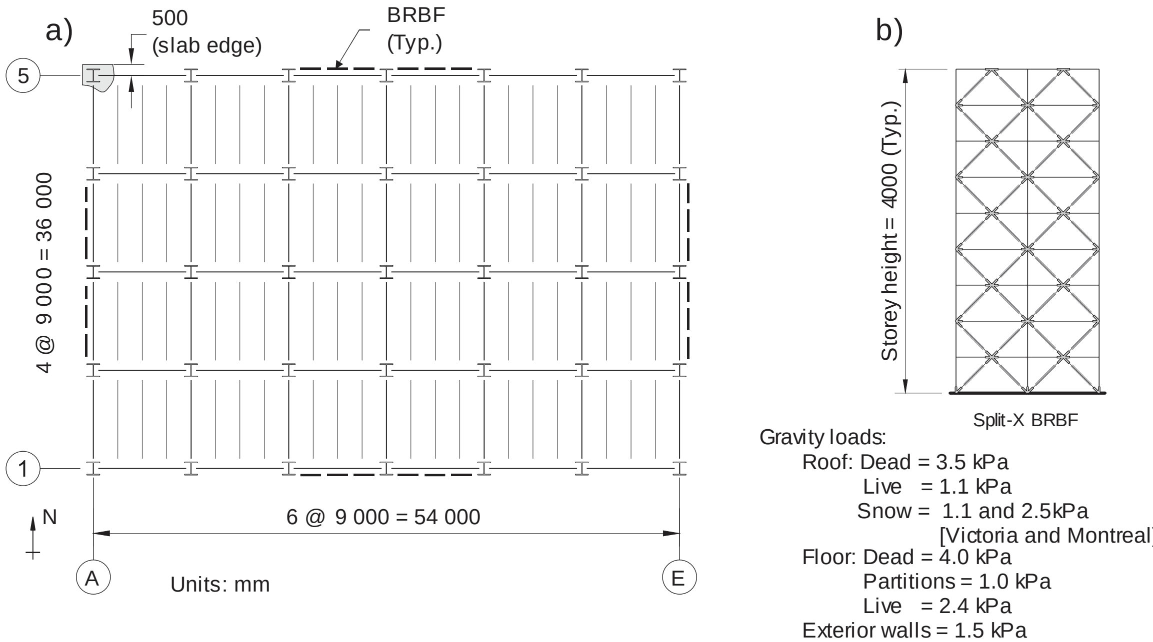

Fig. 3 – Buildings studied: a) Plan view; b) BRBF elevation and design gravity loads.

Table 1 – Properties of the buildings studied

Item	Site class	n Storeys	T_a (s)	T_1 (s)	T_2 † (s)	V/W (–)	$V_{y,1}/W$ (–)	$V_{u,1}/W$ (–)
Victoria	C	3	0.6	0.63	0.26	0.153	0.230	0.338
		5	1.0	1.07	0.43	0.079	0.131	0.192
		7	1.4	1.40	0.54	0.063	0.108	0.158
		9	1.8	1.82	0.68	0.046	0.086	0.127
		13	2.46	2.46	0.87	0.038	0.075	0.111
		15	2.7	2.70	0.94	0.038	0.075	0.111
Victoria	E	3	0.64	0.64	0.26	0.150	0.221	0.325
		5	0.82	0.82	0.32	0.150	0.225	0.330
		7	1.03	1.03	0.39	0.136	0.204	0.300
		9	1.25	1.25	0.47	0.121	0.184	0.270
		13	1.94	1.94	0.68	0.070	0.123	0.181
		15	2.24	2.24	0.78	0.065	0.110	0.161
Montreal	C	3	0.6	1.03	0.44	0.058	0.095	0.139
		5	1.0	1.68	0.69	0.029	0.057	0.083
		7	1.4	2.15	0.85	0.024	0.048	0.070
		9	1.8	2.68	1.00	0.018	0.040	0.059

† T_2 is the period of the second mode of structure

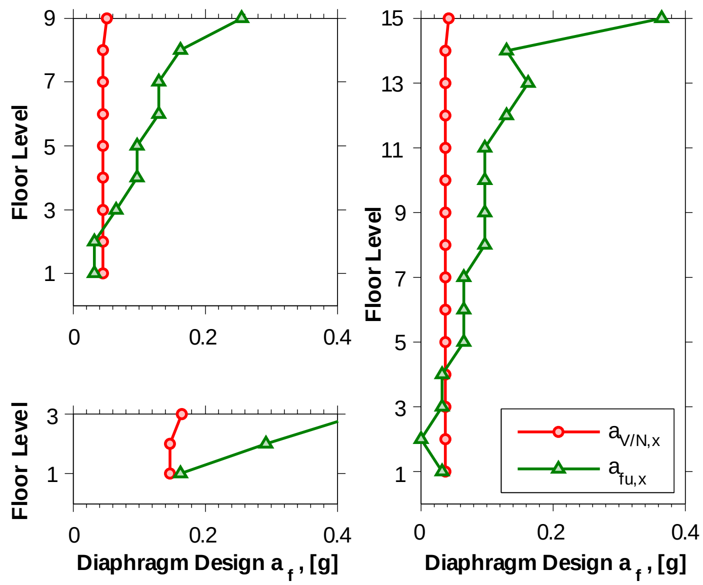

Fig. 4 – NBCC diaphragm design forces (expressed in terms of floor accelerations) for prototype buildings on site class C in Victoria

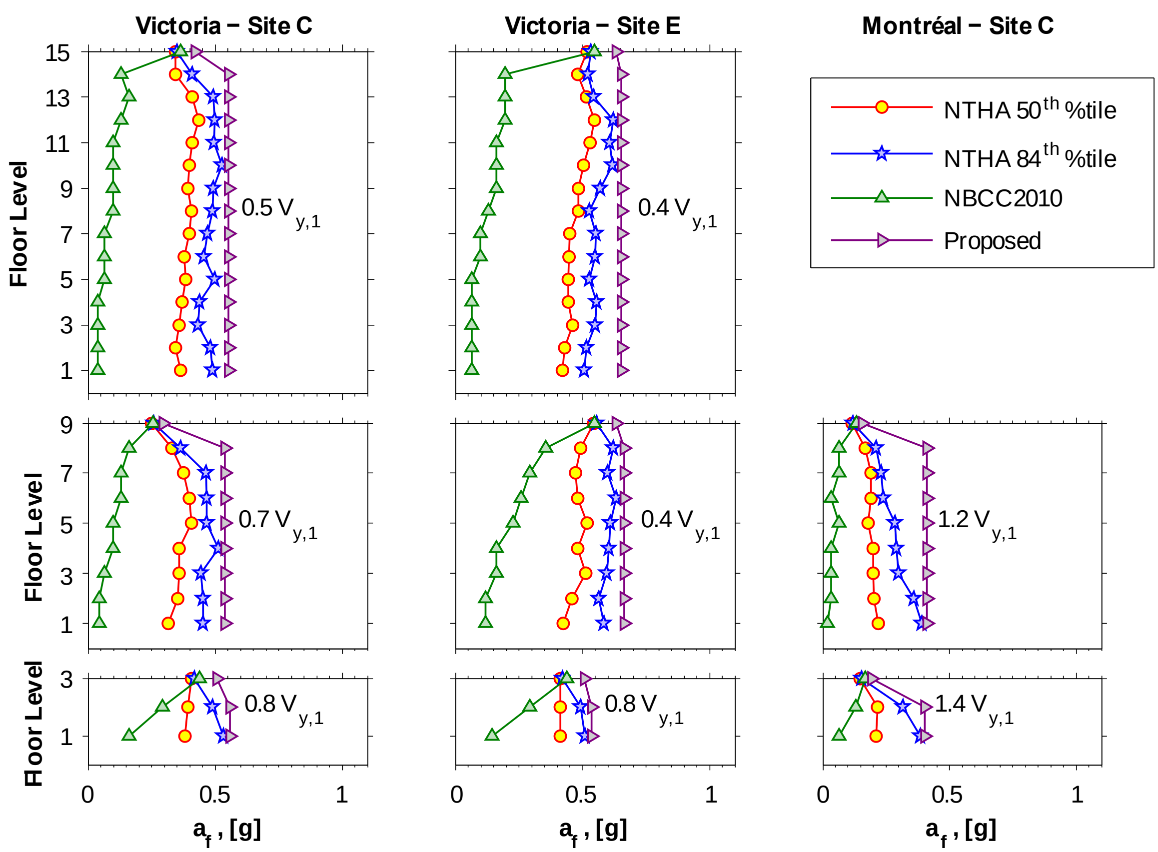

Fig. 5 – Design and NTHA peak floor accelerations.

In all structures, the diaphragm force demands induced by the earthquakes significantly exceed the NBCC design forces by a large margin at every structure level except the roof. Good correlation at the top level is due to the fact that roof inertia loads are bounded by the yield resistance of the structure last level, a condition that is properly accounted for in current NBCC provisions.

Examination of the storey time history responses at the other levels showed that the higher diaphragm forces result from differences in storey shear forces acting in adjacent storeys. An example of this behaviour is illustrated in Fig. 6 for the diaphragm at the first-storey of a 5-storey building designed for generic class C site in Victoria. As shown, diaphragm forces are caused by differences between storey shear forces acting below and above the floor diaphragm as storey shear unbalance must be equilibrated by inertia forces at floor levels. In the NBCC, it is assumed that the seismic forces act in the same direction at all levels and diaphragm forces therefore correspond to the differences in lateral resistance between consecutive floors. As shown in Figs. 6b-d, in reality, time delays between the responses in successive floors can induce much larger inertia loads at floor levels during the elastic phases of the structure response, when small differences in storey drifts can result in large differences in brace forces. In the example shown, the difference in storey shears between levels 1 and 2 nearly reaches the total yield capacities of the two storeys. In Fig. 6a, it is evident that NBCC design forces can be exceeded a large number of times during a single earthquake event and such repeated overloading conditions may lead to severe damage to diaphragms and their components. The study showed that time delays between consecutive floor responses vary with the structure characteristics. In Fig. 7, it is shown that the delay can also be related to ground motion intensity parameters such as PGA, the dominant ground motion period, and strong motion duration, suggesting that ground motion characteristics should be considered for diaphragm design. Data presented in plots of Fig. 7 are from the NTHA of the 15-storey building that was designed for site class C in Victoria.

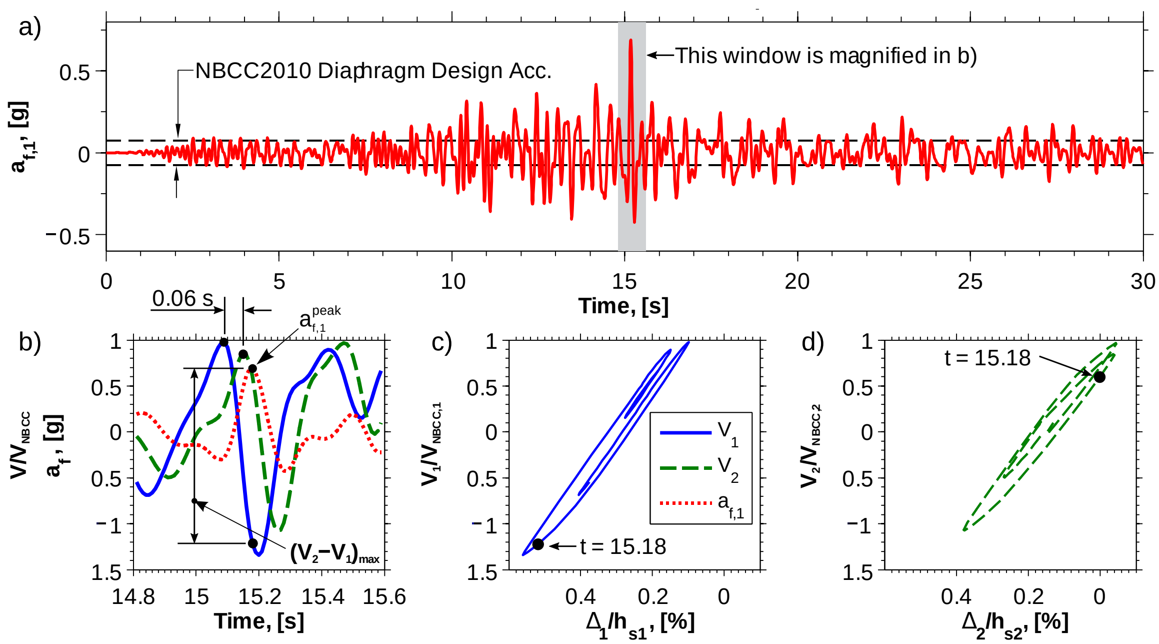

Fig. 6 – Floor acceleration at the first level of the 5-storey building on site class C in Victoria: a) Floor acceleration time history; b) Time window of storey shears and floor acceleration responses; c) and d) Normalized storey shear-storey drift hysteresis at first and second storey.

In Fig. 8, the computed peak floor accelerations are normalized with respect to the peak ground accelerations of the individual scaled ground motions. The results show that floor accelerations generally increase with the vertical position along the building height, as currently considered in the NBCC for non-structural components and building components. However, the amplification is less than the value assumed in the code with peak values not exceeding 2.0 times the peak ground accelerations. In Montreal, de-amplification is even observed along the structure height. This can be attributed to the large differences between ground motion dominant periods and structure periods at this location.

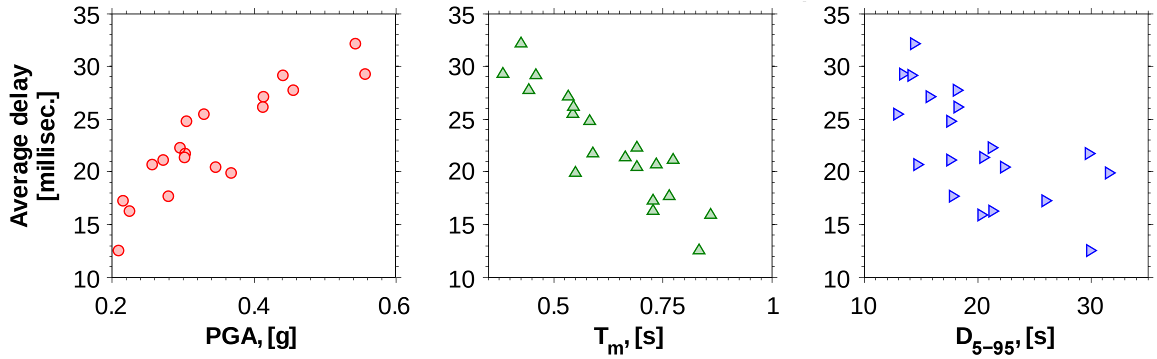

Fig. 7 – Relationship between ground motion parameters and time delays causing inertia forces in floor diaphragms. Notes: T_m is the mean period of ground acceleration; $D_{5\text{-}95}$ is the significant duration.

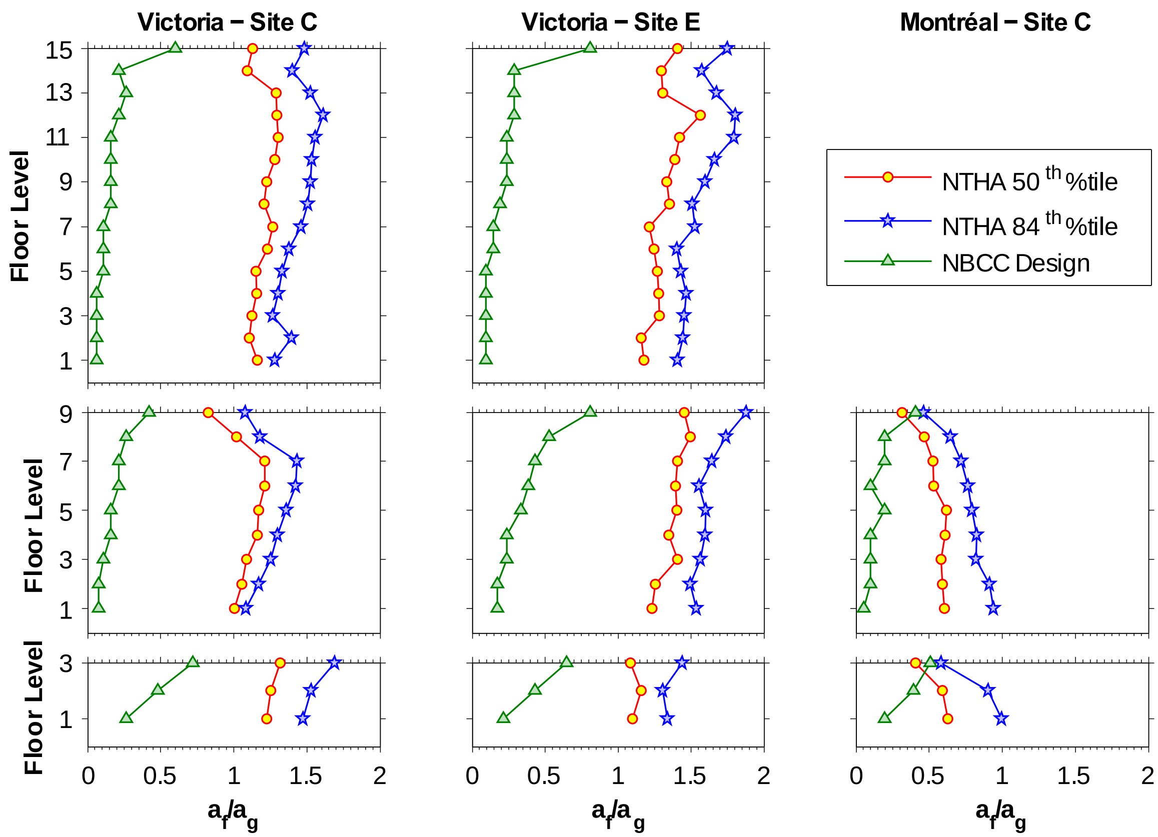

Fig. 8 – Design and NTHA peak floor accelerations normalized to peak ground accelerations.

3.3 Possible avenues for improved design provisions

The results and observations from this study clearly demonstrate that current code provisions for diaphragm designs and peak floor accelerations must be revisited for SFRSs that essentially deform in shear upon yielding. In Fig. 5, peak floor accelerations are nearly constant over the entire frame heights except at roof level. This suggest that a single diaphragm force value could be used for a given structure. In the plots of Fig. 5, a vertical line has been introduced that envelops the 84th percentile acceleration values and the diaphragm force corresponding to that line is given as a function of the V_y value at the structure base. This value is selected as the yield lateral capacity in the first level likely plays a major role in limiting the response in the building levels above. As shown, that force level however varies with the building height and the location of the structure and further studies will be needed to identify and assess relations between floor accelerations and these parameters.

The strong correlation observed between floor and ground motion accelerations could also be exploited when establishing inertia forces for diaphragm design.

4 Conclusions

A study was conducted to assess the provisions of NBCC 2010 and ASCE 7-10 for the seismically induced inertia diaphragm forces and floor accelerations to be used in seismic design. Although both codes have similarities, several differences exist that can lead to considerable variations in diaphragm design forces. The study focused on steel buckling restrained braced frames. For these structures, the lower limit on diaphragm forces prescribed in ASCE 7 is more critical than the inertia forces obtained from structural analysis. It is also significantly higher than the diaphragm forces prescribed in the NBCC.

Nonlinear time history analysis of 16 prototype buildings showed that diaphragm forces exceed the NBCC specified design values by a large margin. Diaphragm overloading typically occurs numerous times during each individual ground motion, suggesting that it would likely produce severe damage. These large forces are caused by delays between storey shear forces acting in adjacent storeys during elastic phases of the responses. Strong correlation was observed between that delay and ground motion intensity characteristics. The results indicate that inertia induced forces at the roof level are bounded by the probable lateral capacity of the last storey and are nearly constant for all remaining levels. The amplitude of the forces can be expressed as a function of the structure base shear yield resistance but that fraction was found to vary with the building height and the site conditions. Additional studies are needed to better characterize these relationships and propose design values. For the buildings studied, current peak floor accelerations assumed in NBCC for the design of non-structural elements and building components are overly conservative. However, the peak floor accelerations were found to correlate well with peak ground accelerations, which means that the current approach only need to be adjusted to reflect actual acceleration amplifications. Future studies should also explore the possibility of exploiting the correlation between peak ground and floor acceleration for diaphragm design.

Acknowledgements

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