Seismic force and ductility demand on the braced bents of single-storey buildings with flexible roof deck diaphragms

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SUMMARY
This paper investigates the influence of the flexibility of the roof diaphragm on the dynamic seismic response of single-storey buildings with metal roof deck diaphragms. Elastic and inelastic responses are investigated for buildings with different lateral stiffness and strength properties. Building lateral deflections as well as shear forces and bending moments in the roof diaphragm are compared to predictions from equivalent static force methods. The ductility demand on the vertical bracing system is also compared to the assumptions made in design. The study shows that the flexibility of the roof diaphragm impacts the inelastic seismic response of these structures. In particular, the ductility demand on the vertical bracing system, the diaphragm shears and bending moments all increase with the roof flexibility. The demand on the vertical bracing is also more pronounced for short period structures. Conversely, diaphragm shears and moments increase with the period.

Keywords: Buildings, Diaphragm, Ductility, Period, Shear flexibility

1. INTRODUCTION

Single-storey steel buildings are extensively used in Canada for light industrial, recreational and commercial applications. In these structures, the metal roof deck panels are used to form an horizontal diaphragm to resist and transfer lateral loads applied at the roof level to the vertical bracing elements (Fig. 1.). When subjected to lateral loads the diaphragm must resist in-plane shear forces and bending moments, which will cause horizontal deformations in shear and flexure. In Canadian building codes, the equivalent static force procedure is generally adopted for the seismic analysis of such single-storey structures. The force demand is obtained from the seismic response of an equivalent elastic single-degree of freedom system characterized by the fundamental period of vibration of the structure and its total seismic weight. The influence of the roof diaphragm's in-plane flexibility can be accounted for in the calculation of the building period. However, past studies have indicated that the diaphragm flexibility may also affect the amplitude and distribution of in-plane shears and bending moments in the roof as well as the peak seismic lateral deflection of the structure (Tremblay and Stiemer 1996, Medhekar and Kennedy 1999, Kim and White 2004, Tremblay et al. 2000). For structures designed with reduced seismic load, due to the use of seismic force modification factors, the ductility demand on the structure may also be impacted by the flexibility of the roof diaphragm (Tremblay and Stiemer 1996, Adebar et al. 2004).

This paper presents a parametric study that has been initiated to assess the influence of the roof diaphragm's in-plane flexibility on the key seismic design parameters for single-storey steel buildings. Furthermore, the range of applicability of the equivalent static force method is evaluated. The study was performed for concentrically braced frame (CBF) steel structures designed in accordance with Canadian code provisions. The buildings were of regular rectangular footprint with uniform mass and stiffness properties, and were assumed to be located on a site class C (firm ground) in Vancouver, British Columbia. Both the elastic and inelastic seismic responses were investigated. The elastic response was obtained from modal response spectrum analysis and linear time history analysis, whereas nonlinear time history analysis was used for the inelastic response. The maximum lateral deflection including diaphragm deformations, the maximum bending moment demand in the diaphragm, and the shear force distribution in the diaphragm were studied. According to current seismic design standards, inelastic seismic response is constrained to the vertical bracing system. For inelastic response, the ductility demand on the vertical bracing system was therefore examined. The parameters that were varied were the period of the structure and the relative stiffness of the roof diaphragm and the vertical bracing system. For the roof diaphragm, the flexural to shear stiffness ratio was also varied. Impact of the hysteretic response of the vertical bracing system was also studied. The results were compared to the values that are used in seismic design when adopting the equivalent static force procedure.

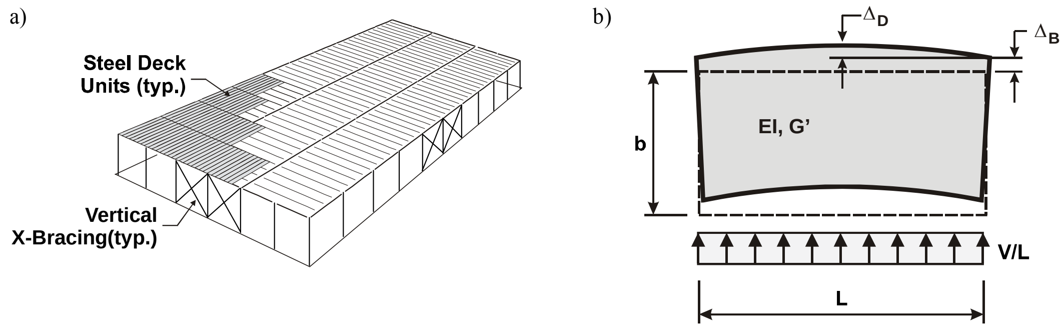

Figure 1. a) Typical single-storey steel building; b) Lateral deformation under uniform lateral loading.

2. PARAMETERS STUDIED

2.1. Building Fundamental Period and Lateral Stiffness Properties

The fundamental period of the structure and the roof diaphragm in-plane flexibility relative to that of the vertical bracing system are the main parameters influencing the seismic dynamic response of single-storey structures with flexible roof diaphragms. The relative magnitude of the in-plane flexural and shear stiffness of the roof diaphragm may also affect the seismic response. For convenience, these stiffness properties are expressed in the form of the respective structural lateral deformations determined under static uniformly distributed lateral loading, as computed when using the seismic static equivalent method. Similarly, for inelastic response, the lateral strength of the vertical bracing system is expressed as a function of the expected elastic force demand based on the fundamental period of the structure, allowing direct comparison with the code equivalent static force procedure.

Figure 1b shows the plan view of a regular rectangular single-storey building, with vertical bracing bents located in the perimeter walls, subjected to a uniformly distributed lateral load applied at the roof level. At the mid-span of the roof diaphragm, the total lateral deformation of the structure, Δ, is the sum of the lateral deformation of the vertical bracing system, Δ_B, and the in-plane roof diaphragm deformation, Δ_D. The latter includes the deformation of the diaphragm due to flexure, Δ_F, and shear, Δ_S:

$$\Delta = \Delta_B + \Delta_D \quad where\text{:}\ \Delta_D = \Delta_F + \Delta_S \tag{2.1}$$

For a regular structure with uniform stiffness properties subjected to uniformly distributed lateral loading over the span length, L:

$$\Delta_B = \frac{V}{K_B};\quad \Delta_F = \frac{5\,VL^3}{384\,EI};\quad \Delta_S = \frac{VL}{8\,G'b} \tag{2.2}$$

In these expressions, V is the total lateral load, K_B is the total lateral stiffness of the vertical bracing, EI is the flexural stiffness of the roof diaphragm, G' is the unit shear stiffness of the steel deck panels, as can be determined using the SDI method (Lutrell 2004), and b is the plan dimension of the roof diaphragm in the direction parallel to the loading. When considering only the lateral deformations of the vertical bracing, the fundamental period of vibration of the structure, T_B, is given by:

$$T_B = 2\pi \sqrt{\frac{W}{g\,K_B}} = 2\pi \sqrt{\frac{W\,\Delta_B}{g\,V}} \tag{2.3}$$

where W is the total seismic weight, assumed to be uniformly distributed over the diaphragm span, and g is the acceleration due to gravity. If diaphragm deformations are considered, the fundamental period, T, of the building can be approximated from:

$$T = 2\pi \sqrt{\frac{W\,(\Delta_B + 0.76\Delta_D)}{g\,V}} \tag{2.4}$$

In this study the building roof diaphragm and vertical bracing system are modelled using the equivalent beam model of Fig. 2a. The beam has the diaphragm flexural stiffness EI and a shear stiffness GA_s set equal to the overall diaphragm shear stiffness, G'b. The vertical bracing system is represented by springs with total lateral stiffness K_B and lateral yield strength V_y (discussed later). Realistic stiffness properties are obtained from the period and stiffness characteristics of an ensemble of rectangular steel buildings designed according to Canadian seismic provisions. These reference buildings have floor areas varying from 500 to 4500 m^2, plan aspect ratios varying from 1.0 to 2.5, and building heights ranging between 4.2 and 12.6 m. They are located at four Canadian sites having different snow and seismic load conditions. The period values of the reference buildings are given in Fig. 3a. The fundamental period, T, varies from 0.25 s to 1.5 s; it generally increases with the diaphragm span and the building height, h_n. As shown in Fig. 3b, there is no definite trend between a structure's period and the lateral deflection Δ_D/Δ_B and Δ_F/Δ_S ratios, indicating that structures with any period may have relatively stiff ($\Delta_D/\Delta_B = 0$) or flexible diaphragms ($\Delta_D/\Delta_B = 2$), with an average Δ_D/Δ_B ratio of 0.57, or diaphragms that deform mainly in shear with Δ_F/Δ_S ranging between 0 and 0.9 with an average value of 0.19.

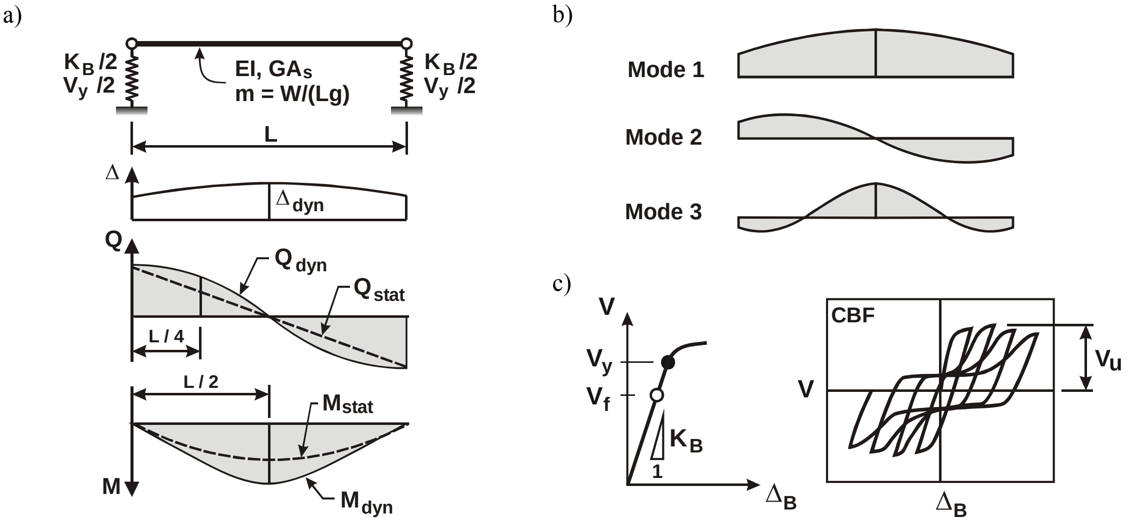

Figure 2. a) Building model and static *vs.* dynamic response; b) Deformed shapes in first three modes of vibration; c) Lateral strength and inelastic response of the vertical bracing system.

In this study 15 cases were studied, as obtained from the combination of the period T = 0.25, 0.5, 1.0, 1.5 and 2.0 s and the ratio Δ_D/Δ_B = 0.5, 1.0 and 2.0. For all 15 cases Δ_F/Δ_S = 0.2 was used, i.e., close to the mean value. For the intermediate case T = 1.0 s and Δ_D/Δ_B = 1.0, Δ_F/Δ_S = 0.6 and 1.0 was considered in order to examine the effect of this parameter, which led to 17 different buildings. For a given period T and a given Δ_D/Δ_B ratio, the period T_B can be obtained by combining Eqs. 2.3 and 2.4:

$$\frac{T_B}{T} = 2\pi\sqrt{\frac{W}{g\,K_B}}\ \Big/ 2\pi\sqrt{\frac{W\Delta_B}{g\,V}\left(1+0.76\frac{\Delta_D}{\Delta_B}\right)} \xrightarrow{yields} T_B = \frac{T}{\sqrt{1+0.76\frac{\Delta_D}{\Delta_B}}} \tag{2.5}$$

The stiffness K_B can then be determined from Eq. 2.3:

$$K_B = 4\pi^2 \frac{W}{g\ T_B^2} \tag{2.6}$$

For a structure with these T, Δ_D/Δ_B and K_B properties, the flexure and shear stiffness of the roof diaphragm, EI and G'b, can be respectively obtained for a given Δ_F/Δ_S ratio:

$$\frac{\Delta_D}{\Delta_B} = (\Delta_F + \Delta_S)\frac{K_B}{V} = \frac{5\,VL^3}{384\,EI}\left(1+\frac{1}{(\Delta_F/\Delta_S)}\right)\frac{K_B}{V} = \frac{5\,K_B L^3}{384\,EI}\left(1+\frac{1}{(\Delta_F/\Delta_S)}\right) \tag{2.7}$$

$$\xrightarrow{yields} EI = \frac{5\,K_B L^3}{384(\Delta_D/\Delta_B)}\left(1+\frac{1}{(\Delta_F/\Delta_S)}\right)$$

$$\frac{\Delta_F}{\Delta_S} = \frac{5\,VL^3}{384\,EI}\frac{8\,G'b}{VL} = \frac{5\,G'b\,L^2}{48\,EI} \xrightarrow{yields} G'b = \frac{\Delta_F}{\Delta_S}\frac{48\,EI}{5\,L^2} \tag{2.8}$$

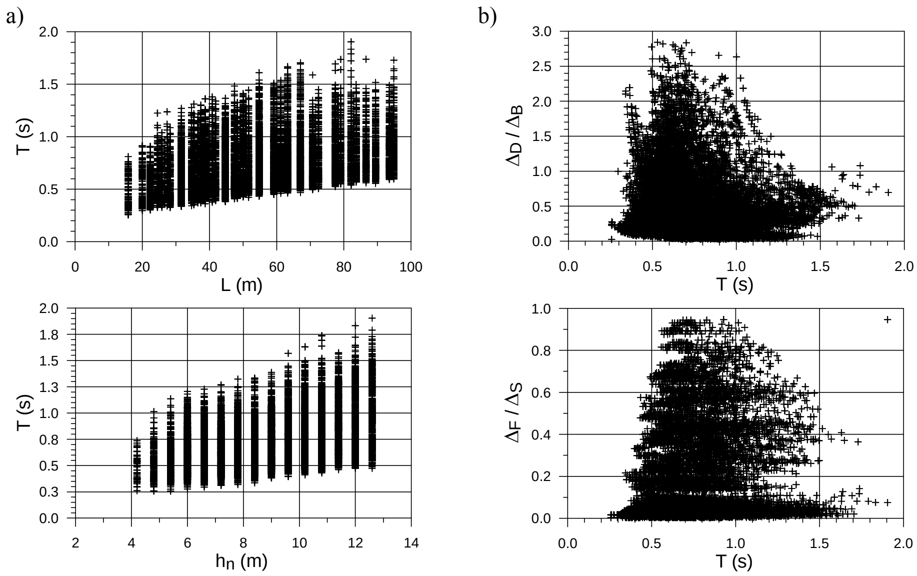

Figure 3. Properties of the reference buildings: a) Period values; b) Relative stiffness properties.

2.2. Building Modal Properties

Figure 2b shows the typical deformed shapes in the first three modes of vibration for this type of structure. Mode 2 is an unsymmetrical mode which is only excited if different ground motions are applied at the building ends. This study is limited to symmetrical response; mode 2 is therefore ignored. Modal analysis of all 17 structures shows that mass participation in modes 1 and 3 amounts to more than 99% of the total structure mass, indicating that only these two modes are of interest. Mass participation in mode 1 is equal to 99, 96 and 93% for Δ_D/Δ_B = 0.5, 1.0 and 2.0, respectively, revealing that first mode response dominates in all cases, with the third mode contribution increasing slightly with diaphragm flexibility. The computed periods in modes 1 (T) and 3 (T_3) of the structures with Δ_F/Δ_S = 0.3 are plotted in Fig. 4a. For a given Δ_D/Δ_B, the ratio T_3/T does not vary with the period T; it takes values of 0.244, 0.285 and 0.307 for Δ_D/Δ_B = 0.5, 1.0 and 2.0, respectively. The ratio T_3/T decreases slightly when increasing the relative flexural flexibility of the roof diaphragm: for Δ_D/Δ_B = 1.0, T_3/T = 0.285, 0.272 and 0.264 for Δ_F/Δ_S = 0.1, 0.6 and 1.0.

The 2010 National Building Code of Canada (NBCC) design spectrum corresponding to 2% in 50 years probability of exceedance for Vancouver, British Columbia, is given in Fig. 4b. Figure 4c contains the ratio between the spectral ordinates in the third and first modes of the structures. As shown, greater participation from higher modes is expected as the period T is increased. A higher ratio is also observed for Δ_D/Δ_B = 0.5, due to the larger difference between T and T_3 when diaphragm flexibility is limited. However, this effect is expected to be mitigated by the fact that structures with more flexible diaphragms have relatively higher mass participating in their third mode.

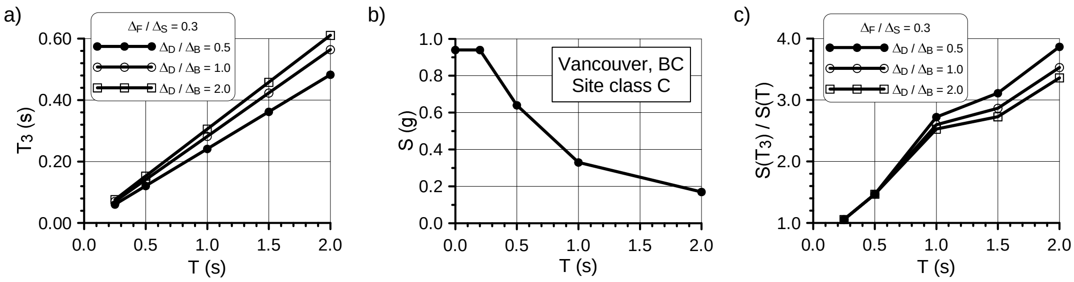

Figure 4. a) Periods in first two modes for the structures studied; b) NBCC design spectrum for Vancouver, BC; c) Spectral acceleration ratios between second and first modes for the structures studied.

2.3. Building Lateral Strength and Inelastic Properties

The lateral strength of the structures, V_y, is established based on current Canadian seismic provisions. For single-storey buildings of the normal importance category the design base shear, V_f, is given by:

$$V_f = \frac{S(T)\,W}{R_d R_o} \tag{2.8}$$

where S(T) is the elastic design spectrum value at the period T, and R_d and R_o are respectively the ductility- and overstrength-related force modification factors. In the NBCC, the building period for design must be obtained from an empirical expression. It is permitted to use the value from dynamic analysis but that value is limited to two times the empirical value. These period requirements are omitted herein so that force demand corresponding to the exact building period is used to establish the lateral strength of the structure. So doing, the deviation from the lateral force method obtained from the dynamic analysis will be solely due to the roof diaphragm flexibility. The R_d and R_o factors depend on the type of vertical bracing system used in the structure. For concentrically braced frame (CBF) steel structures the product of R_d and R_o varies from 1.95 to 3.9, depending on the ductility category. These two extreme values rounded-off to the nearest integer are used in this study: R_dR_o = 2.0 and 4.0.

As shown in Fig. 2c, a pinched hysteresis model is considered to represent CBFs with bracing members buckling in compression and yielding in tension. For this system, the probable yield strength, V_y is the storey shear triggering buckling of the compression braces in the CBFs. In this study, V_y was taken equal to $1.2\times1.3V_f = 1.56V_f$. The 1.2 factor represents the ratio between the probable and nominal brace compressive resistances, as specified in the CSA S16-09 Canadian steel design standard (CSA 2009). The factor 1.3 corresponds to the overstrength-related force modification factor, R_o, representing the overstrength likely to be present in actual structures due to the resistance factor used in design, the difference between probable and nominal yield strengths, and the fact that members are typically oversized due to a selection from a discrete list of available sections (Mitchell et al. 2003). The reduced lateral strength near zero deformation is set equal to 0.3 V_y, which is typical for intermediate brace slenderness.

2.4. Response Parameters Studied

As illustrated in Fig. 2a, the structural response parameters of interest are the shear (Q) and bending moments (M) in the roof diaphragm as well as the building lateral deflections, Δ. The diaphragm shear

Q is examined at the diaphragm ends (Q_{End}) to evaluate possible differences in seismic force amplitudes and at L/4 ($Q_{L/4}$) to determine if the variation of shears along the diaphragm span deviates from the straight line typically assumed in static design. The bending moment is investigated at the diaphragm mid-span only, $M_{L/2}$, using the static bending moments obtained from the end shears (VL/8) as a reference. The following lateral deflections are studied: Δ_B, Δ_D, and the total deflection at the diaphragm mid-span, Δ. For the response spectrum and linear time history analysis methods, the analysis results are compared with those assumed in design when using the equivalent static analysis method. Static deflections of the vertical bracing system and at the diaphragm mid-span are obtained as described above.

For the nonlinear time history analyses, the diaphragm shear and bending moment as well as the diaphragm deflection, Δ_D, used for comparison are those determined under a uniform lateral load corresponding to the probable maximum base shear, V_u, that develops when the tension acting braces reach their probable yield tensile resistances (see Fig. 2c). This base shear is taken equal to $V_u = 1.25\ V_y$, which is expected for braces with intermediate slenderness. The diaphragm deflection under V_u is referred to as $\Delta_{D,u}$. For the lateral displacements at the bracing bents, the ratio of the nonlinear time history analysis deflections to the Δ_B value computed under the load V_y, $\Delta_{B,y}$, gives the ductility demand on the bracing bents. The total deflections at the diaphragm mid-span are compared to the one used in design, i.e. the static deflection under the load V_f multiplied by R_dR_o to account for inelastic effects, $R_dR_o\Delta_f$, as prescribed in the NBCC.

3. MODAL RESPONSE SPECTRUM ANALYSIS

Modal response spectrum analysis (RSA) was performed for all 17 structures assuming 5% damping in all modes. In Fig. 5, the results are compared to those obtained from a static analysis performed under a uniformly distributed lateral load V/L, where V = S(T)×W, i.e. assuming a base shear force determined using the computed first mode period only. Contrary to code practice, the RSA results are not scaled to the static values so that the effect of dynamic response can be examined for all parameters, including base shear force.

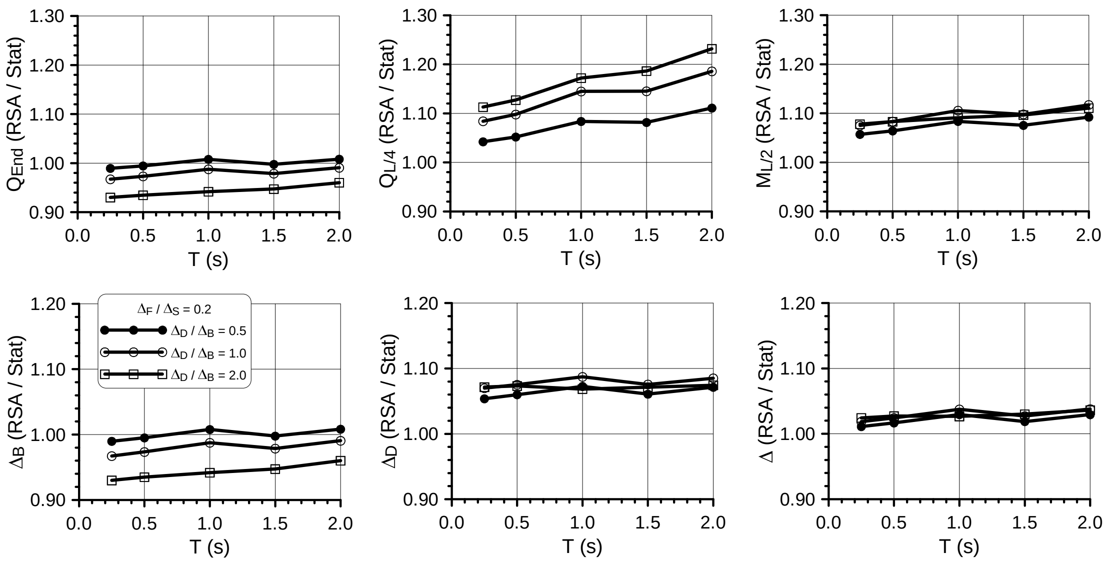

Figure 5. Comparison of design parameters from response spectrum and static analysis methods for the structures with $\Delta_F/\Delta_S = 0.3$.

As shown, the dynamic shear force at the diaphragm end, Q_{End}, which corresponds to half the total seismic load applied to the structure, is equal to the one obtained from static analysis for more rigid diaphragms ($\Delta_D/\Delta_B = 0.5$), regardless of the period. This was expected as rigid diaphragm structures behave more like single-degree-of-freedom systems, i.e. with base shear essentially governed by the first mode period. For more flexible diaphragms, Q_{End} is smaller than the static value; the reduction being more pronounced for shorter period structures. The shear force along the diaphragm span, $Q_{L/4}$, is always larger than predicted by the straight line variation adopted in static analysis, with more pronounced deviations when the diaphragm flexibility and the period are increased. Dynamic bending moments at the diaphragm mid-length are also always higher than the static values, but there is nearly no variation with the diaphragm flexibility and only a slight increase with the period. Dynamic lateral deformations for the vertical bracing system follow the same trend as Q_{End} since the structural response is elastic. Diaphragm deformations, Δ_D, are consistently 6 to 8% larger than the static values, regardless of T and Δ_D/Δ_B.

4. TIME HISTORY ANALYSIS

4.1 Linear Time History Analyses

Time history dynamic analysis was performed using an ensemble of 10 site specific ground motions recorded in past earthquakes. The 5% damped spectra of the scaled ground motions are shown in Fig. 6a. The analysis was performed using the OpenSees platform. Rayleigh damping with 5% of critical damping in modes 1 and 3 was used to isolate the effect of using ground motions instead of a 5% damped response spectrum ordinates on the seismic demand. By taking advantage of the structural symmetry it was possible to model only half the building length; the diaphragm half-span was discretized using 20 beam elements and discrete masses at nodes.

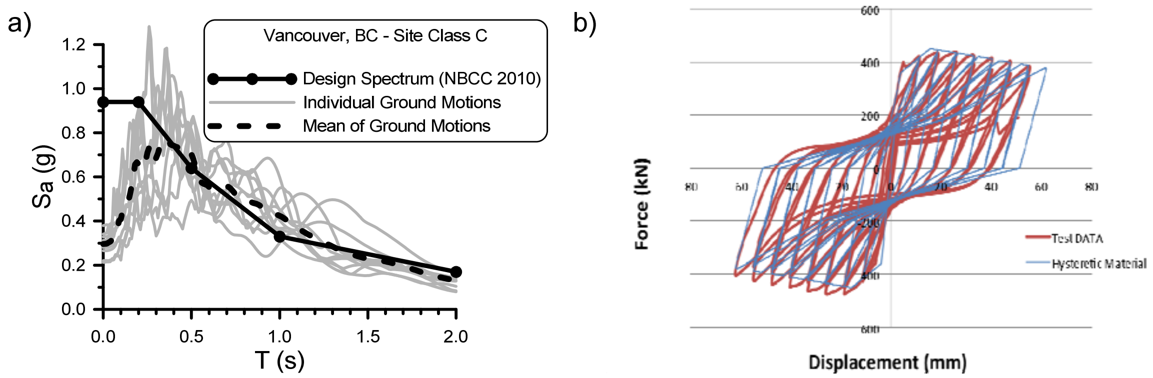

Figure 6. a) Design and ground motion acceleration response spectra for 5% damping; b) Comparison between X-braced frame measured and predicted (OpenSees) hysteretic responses.

The mean results for the ground motion ensemble are compared to static analysis predictions in Fig. 7. Except for the shear $Q_{L/4}$, the shear to flexure diaphragm deformation ratio has no significant influence on the results. Contrary to the response spectrum analysis, the period has a profound effect on the force and drift demands. In Fig. 6a, the mean response spectrum of the ground motion is lower than the design spectrum at T = 0.25 and 2.0 s, close to the design spectrum at T = 0.5 and 1.5 s and exceeds the design spectrum at T = 1.0 s. The ratios in Fig. 7 exhibit the same trends with comparable amplitudes, except for Q_{L4} and $M_{L/2}$. For instance, at T = 1.0 s, the ratio between the ground motion and design spectra is approximately 1.25, which corresponds to the ratio for all drift components and total storey shears. Hence, the observed variations with the period can be generally attributed to the differences between the ground motion and design spectrum demands. Compared to the other design parameters, the shear $Q_{L/4}$ and bending moment $M_{L/2}$ both exhibit relatively greater deviations from static values, however, suggesting that additional dynamic amplification takes place under seismic ground motions. For $Q_{L/4}$, this amplification is more pronounced for long period structures with larger Δ_D/Δ_B ratios, i.e., structures with relatively more flexible roof diaphragms.

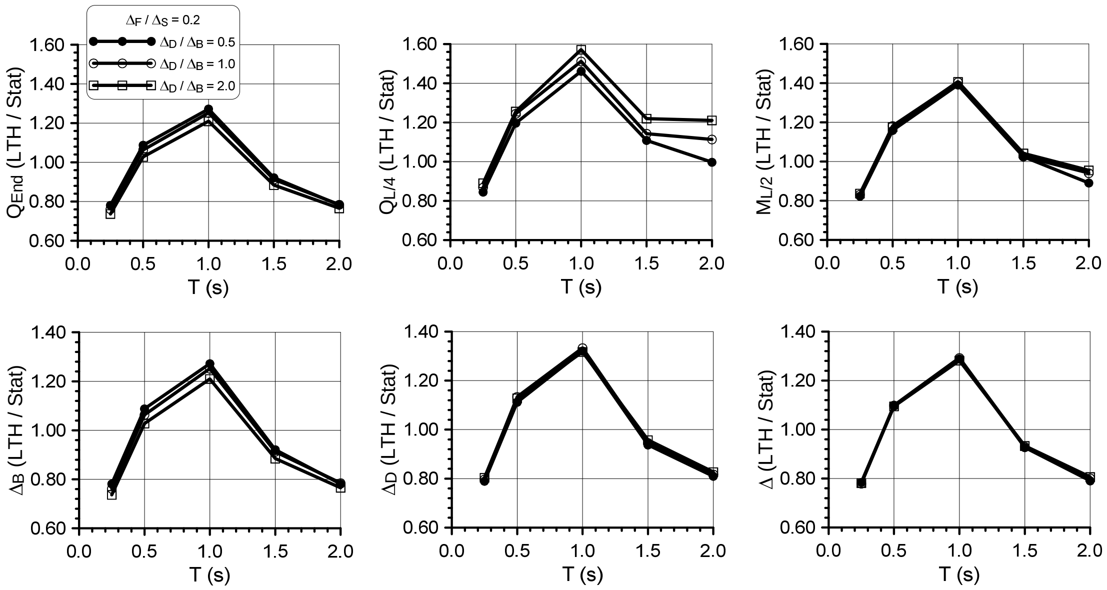

Figure 7. Comparison of design parameters from linear time history (mean of peak values) and static analysis methods for the structures with $\Delta_F/\Delta_S = 0.2$.

4.2 Nonlinear Time History Analyses.

Nonlinear time history dynamic analysis was performed using the same ensemble of ground motions. The same OpenSees model was also used except that hysteretic response was assigned to the vertical bracing system, as discussed in Section 2.3. The *Hysteretic* material in OpenSees was selected to reproduce the pinched hysteretic CBF response; a comparison of the model response with X-bracing test data by Tremblay et al. (2003) is presented in Fig. 6b. A test specimen X6-C with brace effective slenderness, KL/r, of 90 was used to calibrate the model, as this brace slenderness is representative of single-storey steel buildings. In the model, the maximum probable storey shear resistance, V_u was set equal to 1.25 V_y, as discussed previously, and was reached at a ductility, $\Delta_B/\Delta_{B,y}$, of 4.0. Strength degradation due to brace buckling and stretching was then specified. In the analysis, mass proportional Rayleigh damping corresponding to 4% of critical damping in first mode was used. That damping level was observed in in-situ forced vibration testing performed on a similar building type (Proulx et al. 2012). The results are presented in Figs. 8 and 9 for $R_dR_o = 2.0$ and 4.0, respectively.

For R_dR_o = 2.0, the ductility demand in the vertical bracing exhibited the same variation with the period as observed between the ground motion and design spectra, which is expected in view of the limited inelastic response. The ductility varies between 1.0 and 2.0, which is consistent with the global force reduction factor used in design, $R_dR_o = 2.0$, and the overstrength assigned in the model, $R_o = 1.3$, giving $R_d \approx 1.5$. The ductility increases when increasing Δ_D/Δ_B. The maximum storey shear is approximately equal to 0.8 V_u, i.e. close to V_y, which is consistent with the ductility experienced by the vertical bracing. In all structures, the shear in the diaphragm at quarter roof span exceeds the static shear computed with the maximum base shear reached in the analysis and the amplification increases with the period and the relative flexibility of the diaphragm, with a maximum value of approximately 1.5 for T = 2.0 s and $\Delta_D/\Delta_B = 2.0$. Dynamic amplification of the moment at the diaphragm mid-span is also observed, but it is not as significant as for $Q_{L/4}$. In all structures, diaphragm deflections consistent with the maximum base shears are obtained. These deformations seem to be relatively greater when Δ_D/Δ_B is increased, suggesting dynamic amplification when diaphragm flexibility is higher. For all buildings, the method proposed in NBCC to predict total building deflections including inelastic effects appears to give realistic estimates, regardless of Δ_D/Δ_B.

For higher force modification factors (Fig. 9), the ductility demand on the bracing bent increases significantly, varying between approximately 2.5 to 6, and a different variation with the period is observed with the short period structures experiencing the highest ductility levels. As was the case with R_dR_o = 2.0, higher inelastic demand is consistently observed in the CBFs of buildings with relatively more flexible diaphragms (higher Δ_D/Δ_B ratios), but the effect is much more pronounced than with R_dR_o = 2.0 and for the structures with shorter periods. The maximum ductility of 6 was reached for periods of 1.0 s and less, and Δ_D/Δ_B = 2.0. This ductility is approximately twice the R_d value expected when using R_dR_o = 4.0. For all structures, the base shear reached V_u. The amplification of the shear $Q_{L/4}$ follows the same trend as with R_dR_o = 2.0, but is more significant with a maximum of 2.0 for T = 2.0 s and Δ_D/Δ_B = 2.0. Dynamic amplification of $M_{L/2}$ is also more pronounced and follows the same trend as $Q_{L/4}$. The amplification is less, however, with a maximum of 1.4. Diaphragm deflection and total deflection results are similar to those observed for the lower ductility (R_dR_o = 2.0) structures. The NBCC approach for total deflections appears, however, to give more conservative predictions for longer periods (T = 1.5 and 2.0 s).

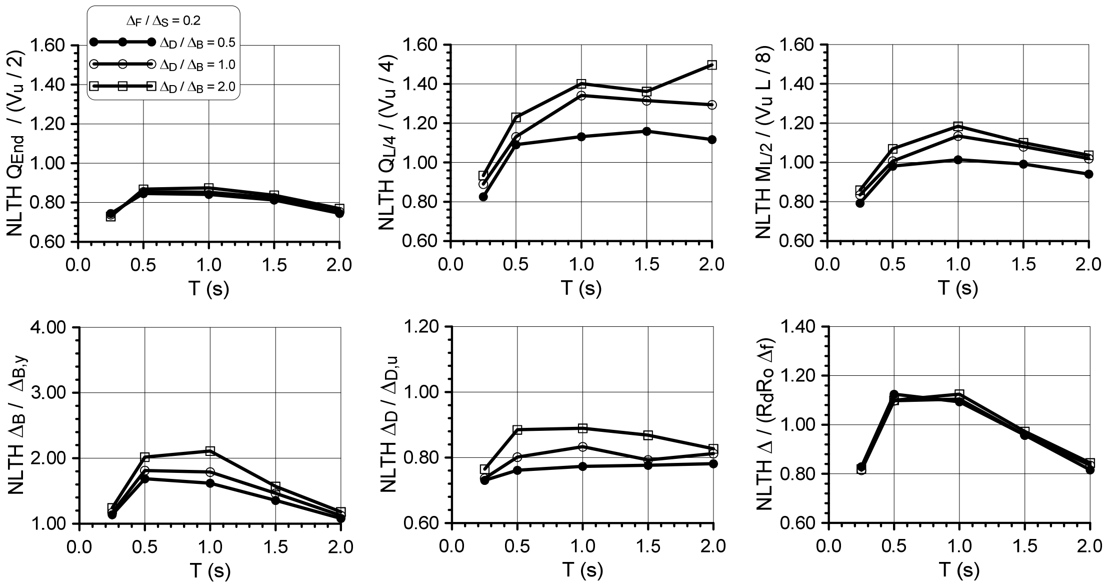

Figure 8. Comparison of design parameters from nonlinear time history (mean of peak values) and static analysis methods for the structures with Δ_F/Δ_S = 0.2 and R_dR_o= 2.0.

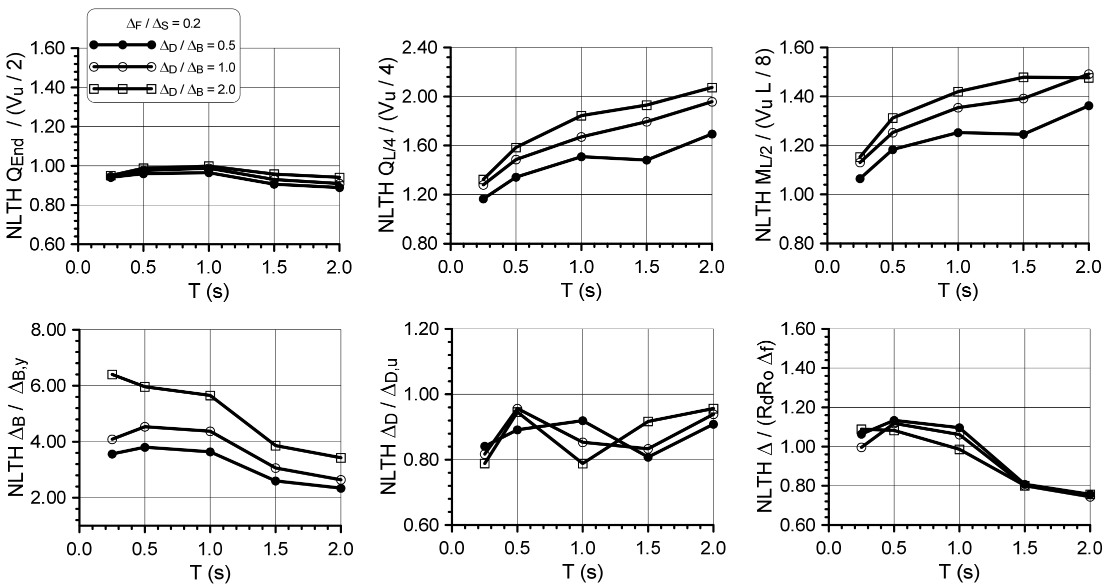

Figure 9. Comparison of design parameters from nonlinear time history (mean of peak values) and static analysis methods for the structures with Δ_F/Δ_S = 0.2 and R_dR_o= 4.0.

5. CONCLUSIONS

A parametric study was performed to assess the influence of the in-plane flexibility of metal roof deck diaphragms on key seismic design parameters for single-storey steel building structures. The study showed that:

- Except for the base shear value, compared to the static equivalent force procedure, the modal response spectrum analysis consistently gives higher values of diaphragm shears at quarter roof length, diaphragm bending moments at the roof mid-length, and diaphragm and total deflections at the roof mid-length. Shears at quarter roof length deviate relatively more from static values as the building period and the flexibility of the roof diaphragm are increased.
- An increase in diaphragm shears with diaphragm flexibility was also observed in the results of the linear time history analyses.
- Nonlinear time history analysis showed that diaphragm shears at the roof quarter length and bending moments at the roof mid-length increase with the building period and the flexibility of the diaphragm and as the design seismic loads are reduced (i.e., higher R_dR_o factors are used). The ductility demand on the vertical bracing system increases as the design seismic loads are reduced, the flexibility of the diaphragm is increased, and the building period is reduced. In all cases, however, realistic estimates of the roof diaphragm deflections and total building deflections can be obtained by static analysis.

These results indicate that the flexibility of roof diaphragms must be explicitly accounted for in seismic analysis and design. In particular, the variations of diaphragm shears, diaphragm bending moments and ductility demand on vertical bracing with the building period, the diaphragm flexibility and the R_dR_o factors cannot be predicted with current elastic analysis methods. Further study is needed to obtain similar response for other locations and site conditions in Canada, as well as for other vertical bracing systems, such as eccentrically braced steel frames or concrete tilt-up wall panels.

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