Impact of foundation rotations on seismic design of steel braced frames

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ABSTRACT

Seismic requirements for foundation design changed significantly in 2015 edition of National Building code of Canada and the current concrete design standard A23.3-14. Foundation movements under seismic loads are now explicitly acknowledged along with the possibility that the foundation uplifts or rocks. Foundations non-restrained against rotation can be designed as: (i) capacity-protected (CP) and (ii) not capacity-protected (NCP). Foundation rotations increase displacement demand on the superstructure and must be included to determine the deflections of seismic force resisting system. This requirement may significantly influence the design of steel-braced frames' foundations leading to very large footings and high associated cost which can even exceed that of the steel-braced bay itself.

In this paper, the requirements of NBCC 2015 and A23.3-14 for seismic foundation design are reviewed and applied to the 3-storey building with moderately ductile concentrically braced frames with X bracing. The buildings are located in Vancouver and Montreal on Class C and Class E sites. Foundation design included capacity-protected and non-capacity protected options. The impact of drift requirements on foundation design is tracked and discussed. The seismic response of building-foundation-soil system is studied using nonlinear time history analysis. The numerical model developed in OpenSees includes inelastic frame behaviour and nonlinear soil response.

CP foundations behaved according to the capacity design principle and maintained the energy dissipation in the superstructure. For NCP foundations, depending on the site Class and location, three energy dissipating mechanisms were mobilised: rocking, inelastic soil behaviour and inelastic frame response, either alone or in combination. Even though the drift criterion was critical for most designs, the results show that the increase of inter-story drifts in the superstructure caused by footing rotations was small. The simplified equation proposed in A23.3-14 to estimate additional drifts seems overly conservative.

Keywords: steel concentrically braced frames, foundation design, nonlinear time history analysis, soil-structure interaction

INTRODUCTION

Seismic analysis of building structures is most commonly done assuming fixed-base support conditions. However, under earthquake loading, foundations supporting the seismic-force-resisting systems move and the resulting displacements and rotations can increase displacement demand in the superstructure. For some types of foundations such as, for example, anchored foundations or foundations supported on piles, rotations are restrained and the impact of foundation movements on seismic force resisting system (SFRS) can be ignored. For others, such as, for example, isolated shallow foundations, the impact may be large and must be considered in design.

2015 edition of National Building code of Canada [1] incorporates several changes in provisions for seismic design of foundations [2]. One important change is that for the foundations unrestrained against rotation, it is now required to determine the additional SFRS displacements that originate from the foundation rotations and consider them explicitly in SFRS seismic design. According to CSA A23.3-14 [3] foundations with unrestrained rotations can be designed as capacity-protected (CP) or not capacity-protected (NCP). CP foundation must have a sufficient overturning moment resistance to develop the capacity of the SFRS so that the dissipation of seismic energy takes place exclusively in the superstructure. The capacity of SFRS is determined in function of the level of ductility anticipated in design. On the other hand, NCP foundations cannot develop the full capacity of SFRS. Such foundations uplift and rock, thereby limiting the forces imposed to the superstructure.

Methods to estimate foundation rotations and related SFRS displacements vary in function of the type of foundation [3]. For CP foundations, for which small foundation rotations are anticipated, inter-storey drifts can be obtained by combining the inter-storey drifts obtained from fixed-base model and the inter-storey drift equal to the actual footing rotations (in radians). Following equation can be used to estimate foundation rotation if the applied overturning moment is sufficiently large to cause the rotation uplift:

$$\theta = 0.3 \left(\frac{q_s}{G_0}\right)\left(\frac{l_f}{a_s}\right)\left\{1 + 2\left(\frac{a_s}{b_f}\right)^{1.5}\right\} \tag{1}$$

where θ is footing rotation in radians, a_s is the length of the foundation under the uniform bearing stress in soil required to resist the applied loads, b_f is the width of the footing, q_s is the amplitude of the uniform bearing stressed engaged by the applied loads, G_0 is the initial shear modulus of soil and l_f is the length of the footing. If the applied moment is smaller than the moment required to cause the uplift, the linear variation between zero and the value given by Eq. (1) can be applied.

To predict rotations of NCP foundation A23.3-14 recommends carrying out dynamic analysis with a model which permits to represent the reduced rotational stiffness of the foundation caused by uplift and deformations of underlaying soil. If such analysis is not possible, design drifts can be estimated by increasing the inter-storey drifts obtained from fixed-base model by the largest between: (i) 50% of top floor displacement divided by the height above the footing, (ii) rotation of foundation calculated from Eq. (1) under the overturning moment equal to the nominal overturning capacity of SFRS without exceeding the rotation resulting from the applied overturning moment equal to the factored overturning footing resistance and (iii) inter-storey drift ratio of 0.005. This amplification must be done for every level, including the foundation.

New provisions for seismic design of foundations in A23.3-14 have been proposed based on studies conducted on reinforced concrete buildings with shear walls [2]. In this SFRS, seismic energy is dissipated through the inelastic deformations of a single flexural plastic hinge at the base, the exact location where the force demand is the largest. Consequently, structural overstrength becomes easier to control, which has a positive impact on the foundation overturning moment demand. For steel concentrically-braced frames, seismic energy is dissipated in braces through tensile yielding and flexural yielding that occurs after the brace buckles in compression. The design aims to distribute inelastic demand to all diagonals. When foundation design forces are calculated for such system, the effect of overstrength cumulates and may cause unrealistically large overturning moments. This cumulation may lead to excessive foundation dimensions even if the NCP design option is considered. In addition, Canadian steel design standard (CAN/CSA S16-14) [4] does not provide guidance regarding the level of capacity to consider in seismic design of foundation supporting steel SFRS. The SFRS nominal capacity, which is used in A23.3-14 to determine foundation design moments, is not defined in S16-14. Probable resistances of primary ductile elements are therefore considered in these calculations, which further increases design forces and foundation dimensions. Examples from practice show that the cost of foundations in certain cases may be so high to even surpass that of the bracing system, thereby putting in question the feasibility to use a steel SFRS.

In this paper, the requirements of NBCC 2015 and A23.3-14 for seismic foundation design are applied to the 3-storey buildings with moderately ductile (MD-type) concentrically braced frames with X tension-compression bracing. The buildings are located in Vancouver and Montreal on Class C and Class E sites. For each studied case, the frames are designed according to NBCC 2015 and S16-14 requirements and capacity-protected and not capacity-protected footings are designed. Critical design parameters are identified, with attention focused on the impact of drift requirements on frame and foundation design. The seismic response of building-foundation-soil system is then studied through nonlinear time history analysis using OpenSees software platform. The numerical model includes inelastic frame behaviour and nonlinear soil response. The latter is modeled using the Beam on Nonlinear Winkler Foundation substructure method. The fixed-base case is also analysed for comparison. The response of the soil-foundation-wall system was observed by tracking the overturning moment on foundations, foundation uplift and the settlement of the soil and drift profiles and compared to design predictions.

BUILDING DESIGN

Frame design

The plan view of the 3-storey prototype building is shown in Figure 1. The building is assumed to be located in Montreal (MTL), QC and Vancouver (VCR), on Class C and Class E sites. X-type tension-compression concentrically braced frames, that are the object of this study, provide lateral resistance in the N-S direction. In the E-W direction, lateral loads are resisted by perimeter moment-resisting frames. The position of the two braced frames in the Montreal building is indicated in Figure 1. In Vancouver, two additional frames were placed symmetrically in the central bay on the perimeter of the building in the short direction. The braced bay width is 8m. The building is 13 m high, with typical story height of 4 m and 4.5m in the first storey.

The frame designs were done in accordance with the requirements of NBCC 2015 and CSA S16-14. The seismic design base shear was determined from a response spectrum analysis assuming fixed-base support conditions and calibrated against $0.8V_d$, as permitted for regular structures. V_d is the seismic base shear obtained using NBCC 2015 equivalent static procedure: $V_d = S(T_a)M_vI_EW/(R_oR_d)$. In this expression, $S(T_a)$ is the design spectral acceleration at fundamental building period T_a, M_V is a factor accounting for higher mode effects, I_E is the importance factor, and W is the seismic weight. For the MTL buildings, dynamic periods (MTL-C: T_a = 0.79 s, MTL-E: T_a = 0.7 s) exceeded the NBCC limit, thus the spectral acceleration S was determined for T_a = 0.66 s. For VCR buildings, the fundamental periods obtained from dynamic analysis (VCR-C: T_a = 0.49 s and VCR-E: T_a = 0.45 s) were greater than NBCC empirical value (T_{emp} = 0.34 s), but smaller than the NBCC limit for this type of structures ($2T_{emp}$ = 0.66 s) and so dynamic periods were used in the base shear calculations. For all buildings in this study R_o = 1.3, R_d = 3, M_v = 1 and I_E = 1. The resulting seismic design base shears were 1091kN, 1599kN, 2753kN and 4162kN, for MTL-C MTL-E, VCR-C and VCR-E frames, respectively.

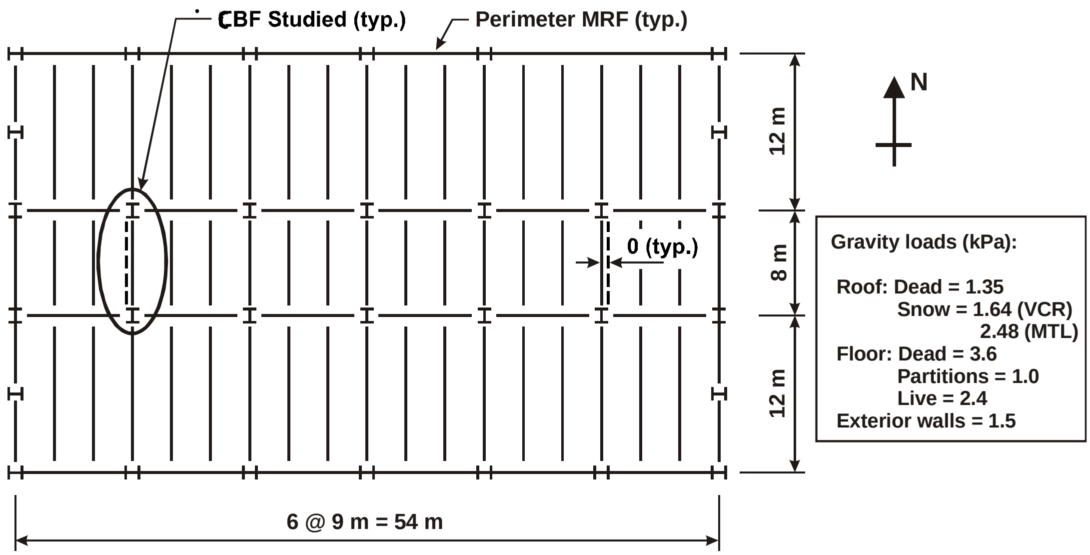

Figure 1: Plan view of the studied building and design gravity loads

Sections of frame members were first selected in compliance with ductility requirements of CSA S16-14 for moderately ductile (Type MD) concentrically braced frames. Braces were selected from square cold-formed HSS sections conforming to CSA G40.20. Their effective slenderness kL/r was limited to 200. Effective length factor, k was taken equal to 0.45 to account for the impact of brace end connections and the mid-support provided by the tensile braces. Beams and columns were selected from W sections compliant with ASTM A992 and designed to carry gravity loads and forces corresponding to the development of brace-probable resistance in tension and in compression without exceeding forces induced by seismic loads calculated with R_oR_d = 1.3. Beams were considered fully laterally supported out of plane by the floor slab. Columns were continuous over the entire height and oriented for weak axis bending under lateral loads. Columns were designed as beam-columns. Capacity axial forces were determined considering that all the braces at the levels above develop their full probable resistance. Vertical components of these forces were added and combined with the gravity induced forces. A bending moment equal to 20 percent of the column plastic moment was applied, to account for the bending moments that arise from non-uniform storey drift demands over the frame height. The frames were then verified for adequate strength and stiffness under all relevant load combinations. No further section modifications were necessary.

Foundation design

Foundations were designed in accordance with the Canadian concrete design standard A23.3-14. Both capacity-protected (CP) and not-capacity-protected (NCP) options were examined. According to A23.3-14, overturning moment resistance of CP foundations be sufficient to withstand the overturning moment introduced by the gravity loading and the overturning capacity of SLRS. The latter is determined in function of realistic estimate of system ductility. For NCP foundations, following design criteria must be satisfied: (i) they should withstand the overturning moment imposed by the gravity loading and the larger of: (a) the overturning moment resulting from the factored loading that includes the seismic loads, calculated using R_dR_o=2.0, and (b) 75% of the nominal overturning capacity of the SFRS; (ii) the soil stress must not exceed the factored soil bearing resistance; (iii) the displacement of the superstructure determined for fixed-base conditions, increased to account for the impact of foundation rotation, must not exceed the limit prescribed by NBCC 2015 for selected SFRS. The notion of nominal capacity being absent in S16-14, probable tensile and compressive resistances of braces were used to determine the design overturning moment demand for both CP and NCP variants.

A summary of soil properties considered in the design is given in Table 1. Factored bearing resistance, q_f , for site class C and E soils were obtained from field data. Ultimate bearing resistance, q_{ult} , and shear modulus, G, were determined using the

Canadian foundation manual [5]. Note that, even though q_{ult} varies as a function of the foundation dimensions, it was established by inspection that for the soil friction angles considered in this study, the impact was negligible. For that reason, as seen in Table 1, the same values of factored bearing resistance were used for design of all foundations on the same site class. Summary of footing dimensions for the four frames studied is given in Table 1. For both locations, regardless of the type of the foundation (CP or NCP), the overturning moment demand governed dimensions of the footings supporting frames on Class C site. For the frames on Class E site the critical design parameter was the frame drift, augmented to include anticipated foundation rotations. This is not surprising as the Class E soil has a low shear modulus, so large rotations foundation rotations were obtained using Eq.1., which in turn resulted in large inter-storey drifts. For example, for MTL-E frame with NCP foundation, the maximum inter-storey drift caused by foundation rotation was four times larger than the inter-storey drift induced by the seismic loading considering a fixed-base condition.

Table 1. Summary of foundation dimensions and soil properties for four studies frames.

Footing dimensions (m)	**MTL-C**	**MTL-E**	**VCR-C**	**VCR-E**
	Capacity-protected foundations (CP)			
Length (L)	13.5	17	15	17.5
Width (B)	4	6	4	6
Depth (d)	1	1.5	1.3	1.5
	Not capacity-protected foundations (NCP)			
Length (L)	12	13.5	14	15
Width (B)	4	6	4	6
Depth (d)	1	1.5	1.3	1.5

Soil properties*	**Site C**	**Site E**
q_{ult} (kPa)	3000	400
q_f (kPa)	1500	200
G (MPa)	100	20

* q_{ult}: ultimate bearing soil resistance
q_f: factored bearing soil resistance
G: shear modulus

NONLINEAR TIME HISTORY ANALYSIS

Modeling of the frame

Nonlinear time history analysis (NLTHA) was carried out using OpenSEES software platform [6]. A 2D model of the superstructure-foundation-soil system is illustrated in Figure 2. Force-based nonlinear beam-column elements were used for braces whereas elastic beam-column elements were used for the beams and columns. The model can represent tension yielding and in-plane and out-of-plane flexural buckling of braces and nonlinear soil response. Thus, the inelastic deformation demands of the braces can be explicitly determined as well as elastic force demand imposed by braces on other frame members and foundations. As recommended by Aguerro [7], each brace was divided into 16 elements, with 4 integration points per element and fiber discretization of the section to reproduce distributed plasticity. The Giuffré-Menegotto-Pinto (Steel 02) material with kinematic and isotropic hardening properties was assigned to the fibers. Initial out-of-straightness was considered. Zero-length elements with high axial and negligible flexural stiffness were applied to model the beam-to-column connections. Column bases were assumed to be fixed. To include P-Δ effects in the analysis, fictitious gravity column was added. 3% Rayleigh damping was specified [8].

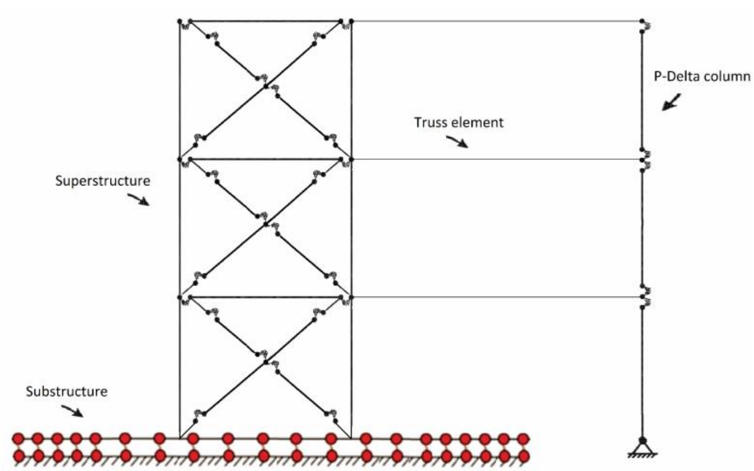

Figure 3. OpenSEES model of structure-foundation-soil system (adapted from Aguero (2006) and Prishati (2008)).

Modeling of the soil-foundation system

The modelling of soil-foundation system was implemented using a flexible boundary substructure approach [8]. This model enables the representation of rocking, sliding and permanent settlement of the foundation. Kinematic effects were neglected as discussed in [9]. Beam-on-Nonlinear-Winkler-Foundation concept was applied to represent nonlinear soil-foundation response. [10]. The foundation is modelled as an elastic beam with a finite number of vertical (q-z type) and horizontal (p-x and t-x) nonlinear springs. For each spring one-dimensional zero-length element is used, and their nonlinear inelastic behaviour is modeled using modified versions of QzSimple1 material [11]. Nonlinear springs were non-uniformly distributed to simulate the rocking behaviour. The use of the variable spring stiffness permitted to represent the higher reactions that can develop in the end-zones under the vertical loads. Footing end-length ratio (L_{end}/L) was set at 20%, and a spring spacing ratio (I_e/L) of 4% was selected considering a minimum number of 25 springs along the footing length [12].

Selection and scaling of ground motion records

Ground motion records for nonlinear time were selected on the basis of magnitude-distance scenarios that contribute the most to the seismic hazard for the design cases studied [13]. Four distinct ground motion sets were constituted for two design locations and two site categories. For Eastern Canada, a set consisted of 11 simulated ground motions from Atkinson database [14] for Western Canada, a set of 15 historical ground motions, 5 for each typical tectonic source (crustal, in-slab and interface) have been used. All ground motions records were calibrated following the procedure described in Tremblay et al. (2015). The response spectra of the individual scaled records and the mean spectrum of the set as well as target modified NBCC design spectrum for Class E site are illustrated in Figures 2 (a) and 2 (b) for Montreal and Vancouver, respectively.

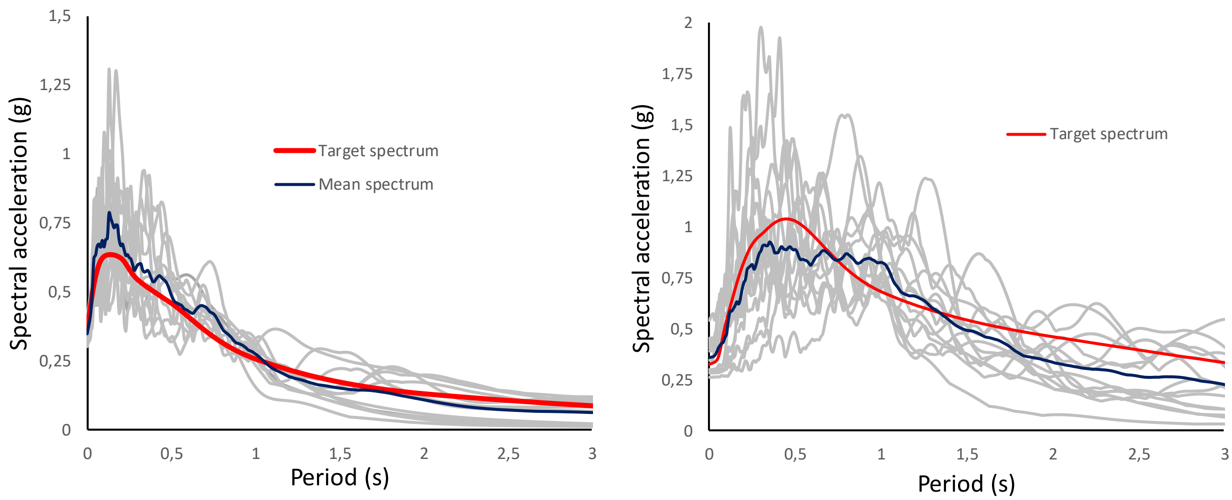

Figure 3. Target acceleration spectrum, individual record spectra and mean 5% damped elastic acceleration spectrum for calibrated ground motions: (a) MTL-E and (b) VCR-E.

RESULTS AND DISCUSSION

The response of the soil-foundation-wall system was examined by tracking the overturning moment at the frame base, the foundation uplift and the settlement of the soil and inter-storey drifts. Maximal forces in the nonlinear soil springs were also assessed. The comparison with design predictions is done using the mean of the five largest peak response values from NLTHA found for individual ground motion records [13].

In Table 2, the overturning moment demand on foundation, M_f, obtained from NLTHA using the full model ($M_{f\ SSI}$) and the fixed-base model ($M_{f\ FB}$) is shown for two foundation options. These results are compared in the same table with the probable overturning resistance of the frame, M_p, and the foundation design overturning moments, M_d, to evaluate the validity of the design predictions. Regardless of the location and the site class, for CP foundations, the ratios of $M_{f\ SSI}/M_d$ slightly exceeded 0.8 except for the MTL-C frame, for which the overturning moment demand on foundation from NLTHA was equal to 0.75 M_d. Considering that for CP foundations design overturning moment is equal to the probable overturning resistance of the frame M_p, this result indicates conservative design estimates and implies that the foundation has an adequate resistance to ensure dissipation of earthquake energy through inelastic frame action. This observation is further confirmed by a very small difference between the overturning moments obtained from fixed-base and full models and significant force demand in braces.

Table 2: Summary of the results for the overturning moments on foundations.

		M_f NLTH (kNm)		**Overturning capacity (kNm)**		**Ratios**			
Structure	Foundation type	Full model $M_{f\ SSI}$	Fixed-base $M_{f\ FB}$	Probable M_p	Design M_d	$M_{f\ SSI}/M_p$	$M_{f\ FB}/M_{f\ SSI}$	$M_{f\ SSI}/M_d$	M_d/M_p
MTL-C	CP	14606	15090	19846	19846	0.74	1,03	0.74	1
MTL-E		23093	23534	27727	27727	0.83	1.02	0.83	1
VCR-C		22162	22523	26552	26552	0,83	1.02	0,83	1
VCR-E		25519	26830	31464	31464	0,81	1.05	0,81	1
						$M_{f\ SSI}/M_p$	$M_{f\ FB}/M_p$	$M_{f\ SSI}/M_d$	M_d/M_p
MTL-C	NCP	14337	15090	19846	15003	0.72	0.76	1.01	0.76
MTL-E		21043	23534	27727	21421	0.76	0.85	1.09	0.77
VCR-C		20645	22523	26552	20971	0.78	0.85	1.07	0.79
VCR-E		25519	26830	31464	26544	0.81	0.85	1.01	0.84

For NCP foundations, the ratios of $M_{f\ SSI}/M_d$ are all close to 1, with the maximum difference of 10 % observed for MTL-E frame, showing that the design foundation moments were well predicted. A closer look at the $M_{f\ SSI}/M_p$, $M_{f\ FB}/M_p$ and M_d/M_p ratios provides insight into the mechanisms engaged to dissipate the seismic energy. For the MTL-C frame, there is 5 % difference between the normalised overturning moment demand at the frame base for fixed-base and SSI model, implying that the foundation-soil system did not contribute to energy dissipation. Indeed, considering that this frame is founded on stiff Class C soil, the normalised design overturning moment, M_d/M_p being the same as the normalised overturning moment, M_{fFB}/M_p, confirms this conclusion. The maximum permanent soil settlement below 1.5 mm and the maximum foundation uplift of only 6 mm further prove the limited energy dissipation in soil-foundation system. For VCR-C, $M_{f\ FB}/M_p$ exceeded M_d/M_p and $M_{f\ SSI}/M_p$ by about 8%. The result suggests that the energy dissipation occurred in the frame and in the soil-foundation system, the latter being mostly by rocking thereby limiting the inelastic frame demand. This behavior is further attested by the small permanent soil settlements and larger foundation uplifts that reached maximum values of 2 mm and 19 mm, respectively.

For MTL-E frame, inclusion of SSI effects resulted in 9% reduction in overturning moment demand. $M_{f\ FB}/M_p$ ratio (0.85) exceeded M_d/M_p ratio (0.77) suggesting that the superstructure and the soil-foundation system participated in energy dissipation. Both rocking and inelastic soil deformations were observed. The peak foundation uplift was 18 mm, while the permanent soil settlement slightly surpassed 21 mm. This is below the limit of 25 mm considered in the literature as acceptable [15]. For VCR-E frame, peak foundation uplift (18 mm) and permanent soil settlement (19 mm) are almost equal suggesting that the energy dissipation through rocking and inelastic soil deformations was comparable. $M_{f\ FB}/M_p$ and M_d/M_p ratios are very similar thus indicating that the superstructure was engaged in energy dissipation. Indeed, the inspection of brace forces confirms significant inelastic response of these elements, nevertheless of lesser extent compared to the one observed for the frames with CP footings.

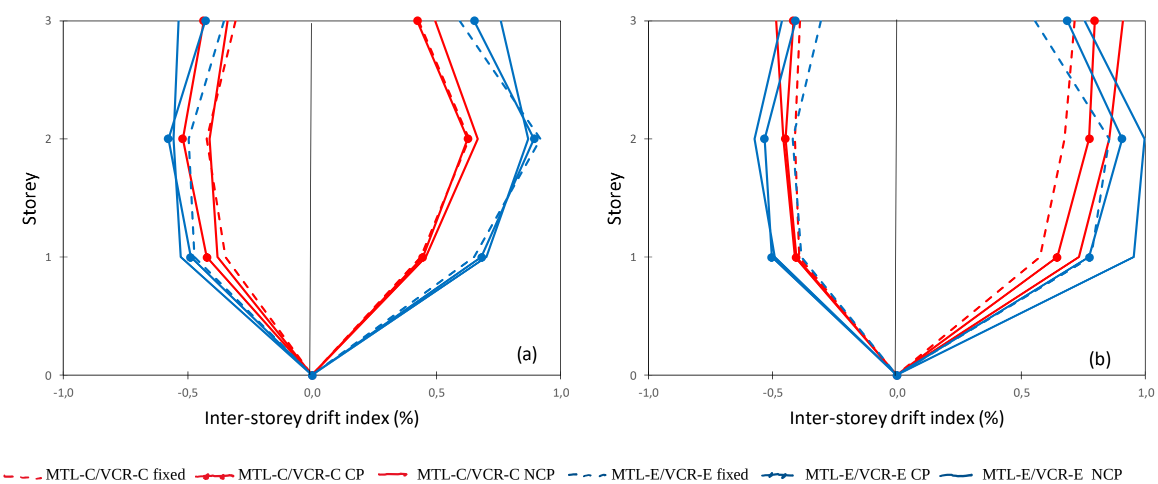

Figure 4. Inter-storey drifts from NLTHA: (a) Montreal and (b) Vancouver.

In Figure 4, the NLTHA inter-storey drifts, obtained for the frames with CP and NCP foundations applying the full SSI and the fixed-base models are compared for Montreal (Fig. 4 (a)) and Vancouver (Fig.4 (b)). The results are expressed as a percentage of the storey height (drift index). Overall, the recorded drifts were small, with the maximum of one percent reached for VCR-E frame with NCP foundation. This value is well below the NBCC limit of 2.5%. As anticipated, for both locations and both site classes, the smallest inter-storey drifts were recorded for the fixed-base frames and the largest for the frames with NCP foundation, but the differences were rather small. In general, the impact of the site class on drifts was more significant compared to the impact of the type of foundation.

Table 3: Comparison of NLTHA inter-storey drifts with design predictions.

	MTL-C				**MTL-E**				**VCR-C**				**VCR-E**			
	CP		NCP		CP		NCP		CP		NCP		CP		NCP	
Storey	Δ_{SSI}	Δ_{design}	Δ_{SSI}	Δ_{design}	Δ_{SSI}	Δ_{design}	Δ_{SSI}	Δ_{design}	Δ_{SSI}	Δ_{design}	Δ_{SSI}	Δ_{design}	Δ_{SSI}	Δ_{design}	Δ_{SSI}	Δ_{design}
3	0.4	1.5	0.5	1.1	0.7	2.3	0.8	2.2	0.8	1.8	0.9	1.2	0.7	2.4	0.8	2.1
2	0.6	1.6	0.7	1.2	0.9	2.5	0.9	2.4	0.8	1.9	0.8	1.3	0.9	2.5	1.0	2.3
1	0.5	1.6	0.5	1.2	0.7	2.5	0.7	2.4	0.6	1.9	0.7	1.3	0.8	2.5	1.0	2.2

Table 3 compares the maximum absolute value of inter-storey drift indexes from NLTHA to the design predictions. For all cases studied, the design estimates were on conservative side, and exceeded the values obtained by analysis by a large margin; 2 to 3 times, for frames with NCP and CP foundation, respectively. For both locations and both site classes, the drift predictions were less accurate for the frames with CP foundations. The best match was achieved for VCR-C NCP frame for which, on average, design estimates were about 1.3 times larger than the drift observed in the analysis. Considering that the dimensions of the foundations for frames on Class E sites were governed by the inter-storey drift requirements, it is of interest to refine the procedure to estimate additional frame displacements induced by foundation rotations.

CONCLUSIONS

In this study, the requirements of NBCC 2015 and A23.3-14 for seismic foundation design are applied to the 3-storey buildings with moderately ductile (MD-type) concentrically braced frames with X tension-compression bracing. The objective was to to validate their use for the seismic design of foundations supporting steel buildings. The frames are designed for Vancouver and Montreal consdering Class C and Class E sites according to NBCC 2015 and S16-14 requirements. Capacity-protected and not capacity-protected footings were considered. The seismic response of building-foundation-soil system was studied using OpenSees software platform. The numerical model includes inelastic frame behaviour and nonlinear soil response. The fixed-base case is also analysed for comparison.

The following conclusions can be drawn from this study:

- In all cases, the inclusion of SSI reduced the overturning moment demand but to a different extent. For CP foundations, design estimates of overturning moments were on conservative side. It was found that for frames with this type of foundations, the energy dissipation was restricted to the superstructure, as intended in design.
- For NCP foundations, design estimates of overturning moments appear to be adequate for the cases studied. However, participation of the foundation-soil system and the superstructure in energy dissipation varied as a function of the location and soil type. For the frames in Montreal, founded on more dense soil the energy was dissipated in the superstructure, while in Vancouver, rocking also took place. For the softer soil, both n Vancouver and in Montreal, three mechanism were engaged: frame inelastic response, rocking and inelastic soil response. The impact of such behaviour on design procedures is a subject of an ongoing study.
- Different criteria governed the foundation design depending on the soil quality. For both locations, the overturning moment demand was critical for the foundations on dense soil or soft rock (Class C), while for the soft soil (Class E), satisfying the design requirement of frame drift limits was the most critical.
- Foundation rotations resulted in larger inter-storey displacements for all cases studied, but, overall, the increase was not very significant. As expected, frames on softer soil developed larger inter-storey drifts. The maximum inter-storey drift index equal to 1% was recorded for VCR-E frame. This is well below the NBCC limit of 2.5% and significantly smaller than the design prediction for both foundation types. Considering that for the frames on Class E site drifts were the critical parameters in the foundation design, the method to estimate increase in displacements due to the rotations of NCP foundations should be further investigated.

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