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Seismic design of steel braced frames' foundations

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ABSTRACT

Seismic design of foundations has undergone important changes in 2015 National Building Code of Canada and the Canadian concrete design standard CAN/CSA-A23.3-14. However, the current Canadian steel design standard (CAN/CSA S16-14) does not detail requirements for foundation design of steel buildings. It is stated that the foundation shall develop the capacity of seismic force resisting system, but no guidance is given regarding which capacity level to use in design, or the combination rules that reflect the vertical distribution of ductility demand. The latter is problematic for steel braced frames for which the ductility demand is not concentrated at the base of the structure, and thus any excessive design overstrength cumulates and affects adversely the foundation design and their cost.

In this study, two 3-storey steel frame buildings with X-type tension-compression bracing were designed for moderate and high Canadian seismic zones, to examine the "not capacity-protected" foundation design. Such foundations do not develop full capacity of the superstructure and can uplift and rock, thereby limiting the force demand. Very dense and soft soil were considered. The numerical model developed in OpenSees includes inelastic frame behaviour and nonlinear soil response. Overturning moment on foundations from nonlinear time history analysis are compared with design predictions. The foundation displacements and soil stresses are also examined to assess the consequences of foundation flexibility on the global structural response. The results show that the foundations demonstrate the rocking behaviour anticipated in design, but the increase of building inter-storey drifts is smaller than predicted.

1 INTRODUCTION

Seismic design provisions for building foundations have changed significantly in the 2015 edition of the National Building Code of Canada (NRCC 2015) and the Canadian concrete design standard A23.3-14 (CSA 2014). Foundations are now explicitly considered as either (i) restrained against rotations (e.g. anchored foundations or isolated shallow foundations) or (ii) free to rotate. In the latter case, the increased horizontal deflections and drifts must be accounted in the design of seismic force resisting systems (SFRS) and accommodated by gravity resisting systems. Two options are possible to design such foundations: capacity-protected (CP) and not capacity-protected (NCP). CP foundations must be able to resist the factored gravity loading and the lateral load overturning capacity of SFRS. This is the preferred type of foundation as it is anticipated that the dissipation of seismic energy will take place primarily in the SFRS, thereby avoiding problems of foundation damage and difficult post-earthquake repairs. In addition, when CP foundations are present, SFRS has a more robust response to any unforeseen severe seismic demand, and additional deformation imposed to the superstructure due to foundation rotations are less important.

NCP foundations, previously called in Canadian codes "rocking" foundations, need not to develop the full capacity of SFRS. Such foundations can uplift and rock, thereby limiting the forces imposed to the superstructure. The superstructure is still the intended source of energy dissipation and must be designed to develop the level of ductility anticipated for SFRS employed. According to A23.3-14, an NCP foundation must have an overturning resistance sufficient to resist factored gravity loads and the larger of: (i) 75% of nominal overturning capacity of SFRS and (ii) overturning moment resulting from the design load combinations including earthquake effects calculated using $R_oR_d = 2$. Nominal capacity is calculated considering resistance factors for concrete (ϕ_c) and steel reinforcement (ϕ_s) equal to 1, while R_o and R_d represent the overstrength and ductility factors, respectively, defined in NBCC for a chosen SFRS. Foundation rotations being unrestrained can cause a significant increase deformations and displacements of SRFS and need to be properly estimated. In all cases, the maximum overturning capacity of foundations should not exceed the overturning moment caused by seismic loads calculated considering $R_oR_d = 1$.

New requirements for seismic design of foundations have been proposed based on studies conducted on reinforced concrete shear wall buildings (Adebar et al. 2014). The inelastic mechanism allowed for this SFRS in a single flexural plastic hinge that develops at the base of the structure. Considering that the maximum force demand happens at the same location, it is easier to control the overstrength for such system, and consequently the forces that are used for foundation design. 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, contrary to A23.3-14 which clearly defines the nominal capacity of concrete structural elements, CAN/CSA S16-14 does not provide guidance regarding the level of capacity to consider for foundation design. This absence of guidance further contributes to increase design forces and foundation dimensions. It has been observed in practice that the cost of foundations design in certain cases may even surpass that of the bracing system, thereby putting in question the feasibility to use a steel SFRS.

In this study, two 3-storey steel frame buildings with X-type tension-compression bracing were designed for moderate and high Canadian seismic zones, with the objective to examine "non-capacity protected" foundation design. Very dense soil and soft soil were considered. Nonlinear time history analysis is carried out for ground motion records selected based on predominant moment-distance scenarios for selected design location and scaled to 2015 NBCC design spectra. The numerical model is developed in the OpenSees program and 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. Overturning moment on foundations obtained from nonlinear time history analysis are compared with design predictions. The foundation displacements and soil stresses are also examined to assess the consequences of foundation flexibility on the global structural response.

2 BUILDING DESIGN

2.1 Design of the superstructure

Two Canadian locations, Montreal (MTL), QC, and Vancouver (VCR), BC, were selected for this study, assuming site Class C (very dense soil or soft rock; 360m/s $\leq v_s \leq$ 760m/s) and Class E (soft soil; $v_s \leq$ 760m/s) conditions at the foundation level. The plan view of the 3-storey building under study and the design gravity loads are given in Figure 1. In the E-W direction, the lateral resistance is provided by perimeter moment-resisting frames while in the N-S direction X-type tension-compression concentrically braced frames are used. The latter are examined in this study. The location of the two braced frames in the Montreal building is shown in Figure 1. For the Vancouver building, two additional frames are symmetrically placed on the perimeter in the central bay. The braced bay width is 8m. The typical storey height is 4m, and the first storey height is 4.5m, giving a total building height of 13m.

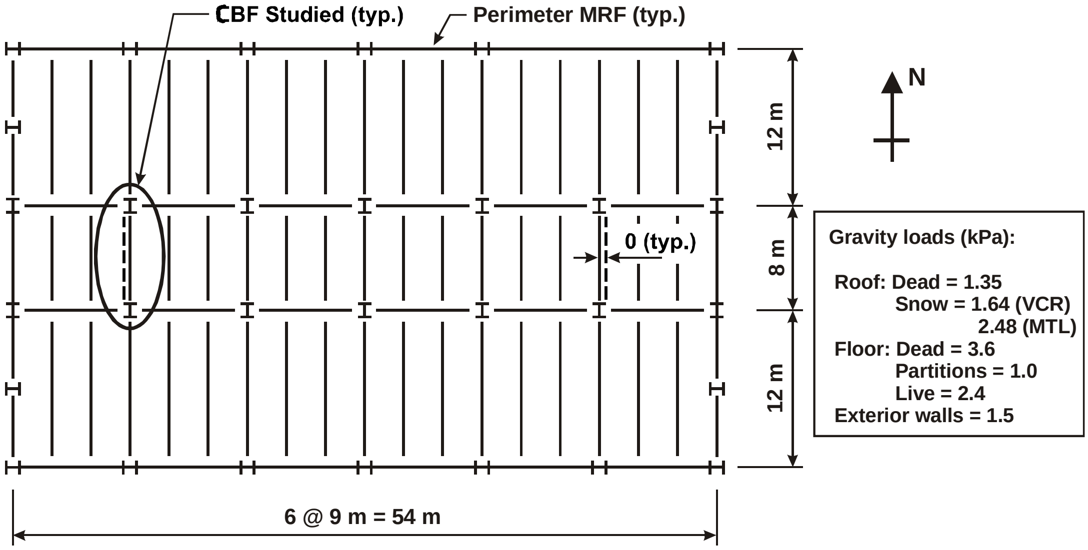

Figure 1: Plan view of the studied building and design gravity loads

The frames were designed following 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 as NBCC 2015 does not require the inclusion of soil-structure interaction in the design. Dynamic base shear was calibrated against $0.8V_d$, as permitted for regular structures. The seismic base shear, V_d, is determined according to 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 for the design location determined at the fixed-base 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 VCR buildings, the fundamental periods obtained from modal analysis (VCR-C: T_a = 0.49 s and VCR-E: T_a = 0.45 s) were used in the calculations as they exceeded the code NBCC 2015 empirical value (T_{emp} = 0.34 s) but were smaller than the maximum period permitted by NBCC for this type of structures ($2T_{emp}$ = 0.66 s). 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 calculation was done using T_a = 0.66 s. 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.

Initial sizes of frame members were determined based on ductility requirements in compliance with seismic design provisions of CSA S16-14 for moderately ductile (Type MD) concentrically braced frames. Braces were sized to carry seismic loads and selected from square cold-formed HSS sections conforming to CSA G40.20. The fixity of their end connections and the mid support provided by the braces in tension were accounted by taking an effective length factor, k equal to 0.45. Brace effective slenderness kL/r was limited to 200. An effort was made to minimize the brace overstrength. 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. Three values are defined in S16-14: probable tensile resistance, $T_u = A_gR_yF_y$, probable compressive resistance, C_u = min ($A_gR_yF_y$ and $1.2C_r$ calculated with ϕ = 1 and R_yF_y), and probable post-buckling compressive resistance, C_u' = min (C_u and 0.20.2 $A_gR_yF_y$). In these expressions, Ag is brace gross cross-sectional area, and R_yF_y is the probable yield stress. 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. To determine column design forces, it was assumed that all the braces at the levels above develop their full probable resistance, and the vertical components of these forces were added and combined with the gravity induced forces. Columns were designed as beam-columns. As per S16-14, 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 stiffness and the strength under all relevant load combinations including gravity loads, notional loads, wind and seismic loads and drift requirements of NBCC. All initially selected sections proved satisfactory.

2.2 Foundation design

Foundations were designed in accordance with the Canadian concrete design standard A23.3-14. While in this study both capacity-protected (CP) and not-capacity-protected (NCP) options were examined, in the present paper, attention was directed to NCP foundations. Three main criteria have to be met in the design of such foundations: (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.

Unlike A23.3-14, S16-14 does not explicitly address the capacity level of SFRS that should be considered in seismic design of foundations. The notion of nominal capacity being absent in this standard, probable tensile and compressive resistances of braces were used to determine the design overturning moment demand.

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

Footing dimensions (m)	MTL-C	MTL-E	VCR-C	VCR-E	Soil properties*	Site C	Site E
Length (L)	12	13.5	14	15	q_{ult} (kPa)	3000	400
Width (B)	4	6	4	6	q_f (kPa)	1500	200
Depth (d)	1	1.5	1.3	1.5	G (MPa)	100	20

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

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 (CGS 2013). 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. For the frames located on site Class C, the stability criterion governed footing dimensions, while for the frames located on Class E site increased frame displacements resulting from anticipated foundation rotations were critical. Summary of footing dimensions for the four frames studied is given in Table 1.

3 MODELING FOR NONLINEAR TIME HISTORY ANALYSIS

3.1 Selection and scaling of ground motion records

Seismic response of the soil-foundation-structure system was studied using 2-D nonlinear time history analysis (NHTA) for sets of ground motion records compatible with design NBCC spectra. Ground motion records are first selected on the basis of magnitude-distance scenarios that contribute the most to the seismic hazard for the design cases studied as suggested in Commentary J of NBCC 2015 (Tremblay et al. 2015). To compensate for the lack of historical earthquake records representative of seismic hazard in Eastern Canada, simulated records were also employed. 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 (2009) 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 according to 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 NBCC design spectrum for Class E site are illustrated in Figures 2 (a) and 2 (b) for Montreal and Vancouver, respectively.

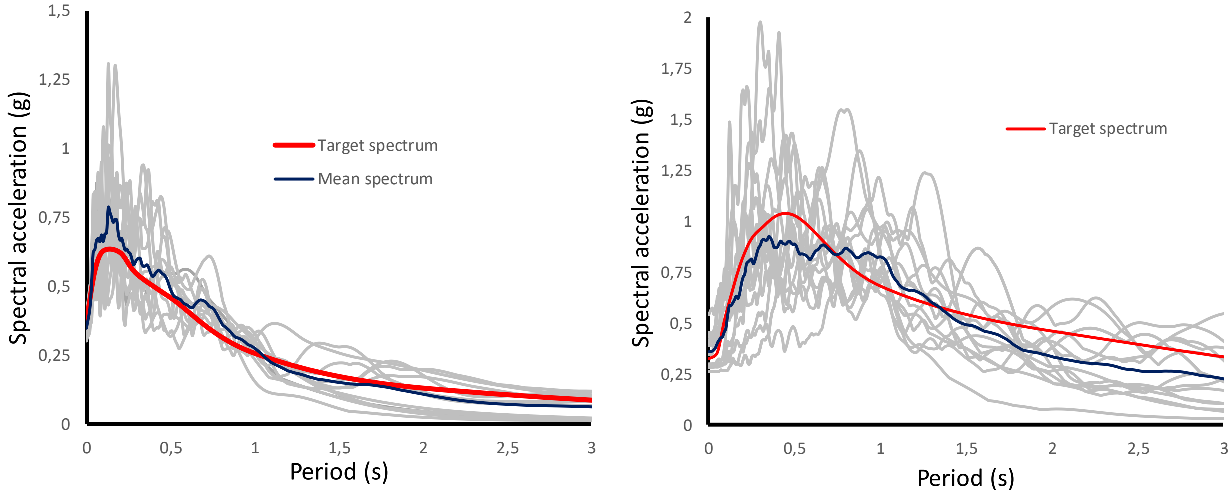

Figure 2: NBCC 2015 design acceleration spectrum, individual record spectra and mean 5% damped elastic acceleration spectrum for calibrated ground motions: (a) MTL-E and (b) VCR-E

3.2 Modelling of the superstructure

A 2D model was developed in the OpenSEES software platform (McKenna et al. 2004) to carry out nonlinear time history analysis of the superstructure-foundation-soil system seismic response (Figure 3). Inelastic frame behaviour and nonlinear soil response were represented. Force-based nonlinear beam-column elements were used for braces whereas elastic beam-column elements were used for the beams and columns. The model allows to explicitly evaluate the inelastic deformation demands on the braces from frame members and foundations. As recommended by Aguerro et al. (2006), 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 columns were added. 3% Rayleigh damping was specified (Deierlein et al. 2010).

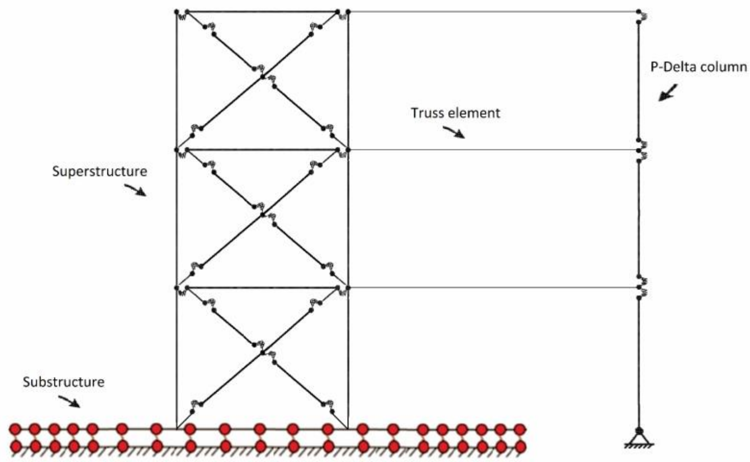

Figure 3: OpenSEES model of structure-foundation-soil system (adapted from Aguero (2006) and Prishati (2008))

3.3 Foundation-soil modelling

The modelling of soil-foundation system was implemented using a flexible boundary substructure approach (Deierlein et al. 2010). This model enables the representation of rocking, sliding and permanent settlement of the foundation. Kinematic effects were neglected as discussed in Kramer and Stewart (2004). Nonlinear soil-foundation response was represented using the Beam-on-Nonlinear-Winkler-Foundation concept (Gajan et al. 2008). The foundation is modelled as an elastic beam with a finite number of vertical (q-z type) nonlinear springs. Each spring is represented by one-dimensional zero-length element, and their nonlinear inelastic behaviour is modeled using QzSimple2 material. Radiation damping was calculated as specified by Gazetas (1991). 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. As suggested by Gajan et al. (2008), a 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 of 25 springs along the footing length.

4 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 with respect to the position under gravity loads and maximal forces in the nonlinear soil springs. The impact of footing rotations on frame inter-storey drifts was assessed. The results represent the mean of the five largest peak response values found for individual ground motion records (Tremblay et al. 2015). The fundamental vibrational periods were also examined.

As expected, the inclusion of soil-structure interaction effects resulted in the lengthening of the fundamental periods. The period increase was independent of the design location and was more pronounced for the softer soil (15% and 5% for site Class E and C, respectively). The recorded drifts were smaller than design predictions, reaching a maximum of 1% for VCR-E frame. This value is well below the NBCC limit of 2.5%.

The overturning moment demand on foundation, M_f, obtained from NLTHA considering the full models and the fixed-base case is shown in Table 2. These results are compared in the same table with the probable overturning resistance of the frame, M_p, and the foundation design overturning moments to assess the validity of the design predictions. The total soil settlements and the foundation uplifts are shown in Figure 4 (a). The ratios of $M_{f\,SSI}/M_d$ are all close to 1, with the maximum difference of 10 % observed for MTL-E frame, which indicates 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 was dissipated 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.

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

	M_f NLTH (kNm)		Overturning capacity (kNm)		Ratios			
Structure	Full model $M_{f\,SSI}$	Fixed-base $M_{f\,FB}$	Probable M_p	Design M_d	$M_{f\,SSI}/M_p$	$M_{f\,FB}/M_p$	$M_{f\,SSI}/M_d$	M_d/M_p
MTL-C	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 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 for this case. 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 (Lindeburg 2015). For VCR-E frame, peak foundation uplift (18 mm) and permanent soil settlement (19 mm) imply 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.

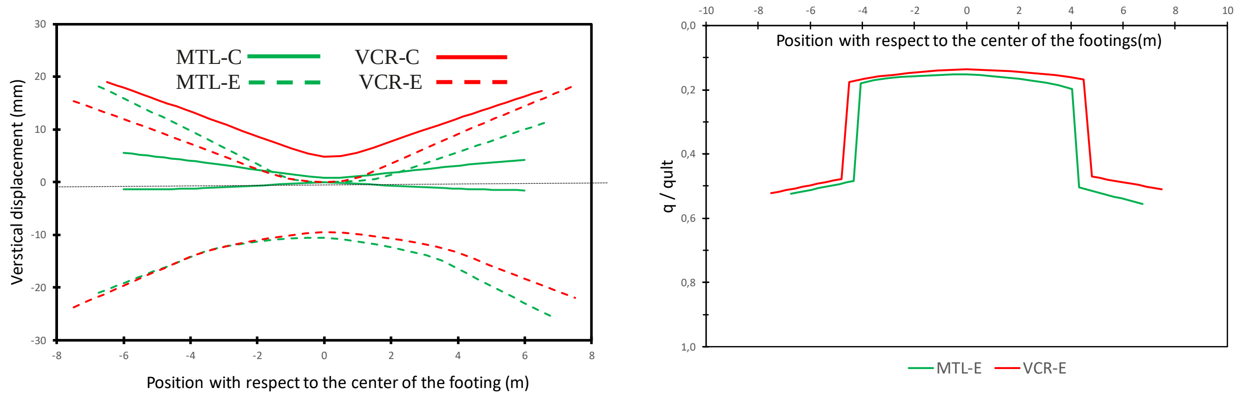

Figure 4: (a) Permanent soil settlements, foundation uplifts and (b) normalised forces in the springs

In Figure 4 (b) the forces in the soil nonlinear springs for MTL-E and VCR-E frames are normalized by the ultimate soil bearing resistance. The peak values occur in the edge springs reaching 0.46 q_{ult} and 0.5 q_{ult} for MTL-E and VCR-E, respectively thus indicating an inelastic soil response ($q_f > 0.3q_{ult}$). This level of bearing pressure can be easily accommodated by the soil. For MTL-C and VCR-C structures, the maximum normalized force recorded in the edge spring is only $0.1q_{ult}$, implying that the soil response is elastic.

5 CONCLUSIONS

The following conclusions can be drawn from this study:

- The foundation design was governed by different criteria as a function of 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 most critical.
- As anticipated, the inclusions of SSI lengthened the structural period. The effects were independent of the design location, and more pronounced for the soft soil (15% for class E site). For all frames, the foundation rotations augmented the inter-storey drifts, with the maximum drift index of 1% recorded for VCR-E frame. This is well below the NBCC limit of 2.5% and smaller than the design prediction. Considering that for the frames on Class E site drifts were critical parameters in the foundation design, the method to estimate increase in displacements due to the rotations of NCP foundations should be further investigated.
- In all cases, the inclusion of SSI lessened the overturning moment demand but to a different extent. Design estimates 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 dense soil in the moderate seismic zone the energy was dissipated in the superstructure, while in high seismic zone, rocking also took place. For the softer soil, for both seismic zones rocking and inelastic response of frame and soil were observed. The impact of such behaviour on design procedures is currently under study.

6 ACKNOWLEDEGEMENTS

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