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STRATEGIES FOR SEISMIC DESIGN OF SHALLOW FOUNDATIONS FOR STEEL BUILDING STRUCTURES

Sanda Koboevic [(1)], Angel Reyes-Fernandez [(2)], Usthanthan Murugananthan [(3)], Lydell Wiebe [(4)]

[(1)] *Associate professor, Department of Civil Engineering, Polytechnique Montreal, Montreal, QC, Canada, email: sanda.koboevic@polymtl.ca*
[(2)] *M.Sc. student, Department of Civil Engineering, Polytechnique Montreal, Montreal, QC, Canada, email: angel.reyes-fernandez@polymtl.ca*
[(3)] *Former M.Sc. student, Department of Civil Engineering, Polytechnique Montreal, Montreal, QC, Canada, email: ushanthan.murugananthan@gmail.com*
[(4)] *Associate Professor and Chair in Effective Design of Structures, Department of Civil Engineering, McMaster University, Hamilton, ON, Canada, email: wiebel@mcmaster.ca*

Abstract

Canadian provisions allow two alternatives for the seismic design of foundations: capacity-protected (CP) and not capacity-protected (NCP). CP foundations should develop the full resistance of seismic force resisting system (SFRS) and are favoured whenever possible. With such foundations the inelastic activity occurs predominantly in the superstructure, unexpectedly high seismic demands are better managed and the global system deformations are not increased significantly by foundation rotations. NCP foundations develop a partial capacity of the SFRS. Being weaker than the SFRS, such foundations uplift and rotate thus limiting the forces transmitted to the superstructure. Conversely, the foundation rotations increase displacements of the superstructure which must be considered in design.

Canadian design practice shows that foundations of steel frame buildings are often large causing a significant increase in construction cost, which may lead to selection of alternative structural solutions built in different materials. Knowing that the design requirements were developed mainly considering the seismic behaviour of concrete shear walls, characterized by the development of a single plastic hinge at the base, it appears necessary to validate their applicability to steel braced frames that exhibit a distributed yielding mechanism, associated with much larger overstrength and higher capacity design forces on foundations.

In this study, 3-storey steel buildings with X-type tension-compression bracing were designed for Vancouver, Canada, to examine different design strategies for foundation design. Two soil types were considered. The foundation design followed Canadian and US seismic design approaches. Non-linear time history analyses were then performed using the OpenSees program. The model included the inelastic frame behaviour and the nonlinear soil response. The forces imposed on foundations obtained from nonlinear time history analysis are compared with design predictions. The foundation displacements and stresses in the soil are also examined to assess the consequences of foundation flexibility on the global structural seismic response.

Keywords: Braced frames, Foundations, Soil-structure interaction, Design, Nonlinear time-history analysis.

1. Introduction

Even though seismic design requirements for shallow foundations have much advanced over time, foundation design for seismic loads is often given a lower priority by practicing engineers in comparison to that accorded to seismic design of superstructures. Foundation design considerations under seismic loads are complex in nature because of the variability of soil conditions and diverse building configurations and structural systems. Limiting the inelastic activity to the superstructure and avoiding foundation rocking and nonlinear soil response is justifiable in view of the difficulty to detect post-earthquake damage and the high cost of possible foundation repairs. On the other hand, the foundation rocking can in fact be beneficial by protecting the superstructure and reducing the need for intricate ductile detailing, provided that the demands on the soil are acceptable.

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In Canada, foundations are generally treated as protected elements and designed to develop the full resistance of seismic force resisting system (SFRS). For unanchored foundations, the alternative design approach is also possible: the foundations can be sized to develop partial resistance of SFRS assuming that at a certain load level the foundation will "rock", and thereby limit the seismic demand on the superstructure. In the United States, practices in individual offices vary significantly as a function of experience and requirements from the client [1]. Because consideration of the SFRS overstrength is not mandatory for foundation seismic design, most often the foundations are designed for the same level of seismic loads as the ductile elements of the superstructure. This implies that some inelastic foundation response will occur at the level of design earthquake. Such approach is justified by the very limited observations of building failures caused by inadequate foundation behaviour during past earthquakes. In some instances, capacity design principles are applied to a certain extent with the objective to control some brittle failure modes related to shear and axial compression. On rare occasion, design offices would opt for a "strong foundation" approach which resembles to Canadian methodology for CP foundation design.

Canadian design requirements for the concrete foundations of buildings provided in the 2015 edition of the NBCC [2] and the corresponding concrete design standard (CSA A23.3-14) [3] have been significantly updated compared to previous editions. These developments were primarily driven by analytical studies conducted on reinforced concrete shear wall buildings, considering a variety of shear wall capacities, foundation capacities and soil types [4]. Even though the new provisions are comprehensive, well-structured and provide much clearer guidance to engineers, their application to concrete and steel buildings reveals some inconsistencies. The reasons behind these inconsistencies are twofold: on one hand, CSA A23.3 and CSA S16 [5] define differently the appropriate level of superstructure capacity that should be developed by foundations, and on the other hand, there is an inherent difference between the yielding mechanisms that develop in concrete walls (single plastic hinge at the base) and steel frames (distributed yielding mechanism). For these reasons, the overstrength tends to be more pronounced for steel frames, which in turn leads to very large foundation design forces and thus large foundation sizes. It has been observed in practice that, in some cases, the cost of concrete foundations may even surpass that of the bracing system, thereby calling into question the feasibility to use a steel frame to resist seismic loads. It is therefore of interest to re-evaluate foundation design provisions for steel braced frames, with the objectives of promoting safe and resilient seismic performance while avoiding unnecessary costs.

In this study, 3-storey steel buildings with X-type tension-compression bracing were designed for Vancouver, Canada, to examine different design strategies for foundation design. Two soil types were considered. The foundations design followed Canadian and US seismic design provisions. Non-linear time history analyses were then performed using the OpenSees program. The model included the inelastic frame behaviour and the nonlinear soil response. The forces imposed on foundations obtained from nonlinear time history analysis are compared with design predictions. The foundation displacements and stresses in the soil are also examined to assess the consequences of foundation flexibility on the global structural seismic response.

2. Building design

2.1 Design of the superstructure

The 3-storey building under study is in Vancouver, BC on Class C (very dense soil or soft rock; 360m/s $\leq$ vs $\leq$ 760m/s) and Class E (soft soil; vs $\leq$ 180m/s) sites. The plan view of the building and the design gravity loads are given in Fig. 1. In the N-S direction, examined in this study, lateral resistance is provided by four X-type tension-compression concentrically braced frames of moderate ductility (MD) type with R_d = 3 R_o= 1.6, where R_d and R_o represent force reduction coefficients related to system

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ductility and overstrength, respectively. 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.

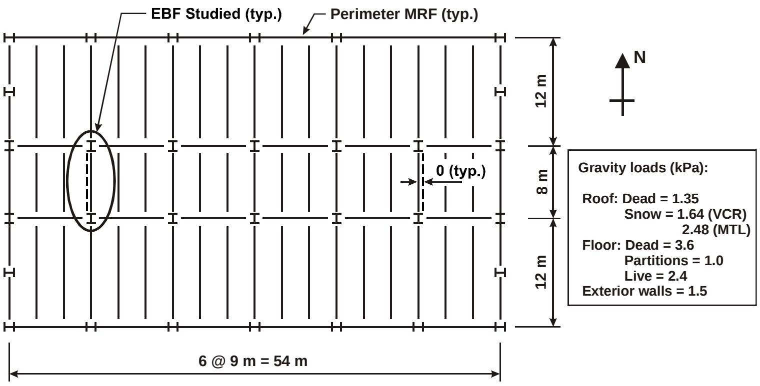

Figure 1. Plan view of the studied building and design gravity loads

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 conditions. The resulting seismic design base shears were 2753 kN and 4162 kN, for the VCR-C and VCR-E frames, respectively. Frame members were first sized to satisfy ductility requirements. Braces were selected from square cold-formed HSS sections while beams and columns were selected from W sections and designed to carry gravity loads and forces corresponding to the development of the brace probable resistances in tension and compression without exceeding forces induced by seismic loads calculated with R_dR_o = 1.3. As per CSA S16-14, in addition to vertical loads, a bending moment equal to 20 percent of the column plastic moment was considered 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. More detailed information on the superstructure design can be found in [6].

2.2 Design of foundations

Foundations were first designed following the provisions of the Canadian concrete design standard CSA A23.3-14 for two design options: capacity-protected (CP) and not-capacity-protected (NCP). For comparison, an alternative design was also carried out using the US design approach, in which no capacity considerations were taken into account to determine foundation design forces. CSA A23.3-14 requires that the overturning moment resistance of CP foundations be sufficient to withstand the overturning moment introduced by gravity loading and the overturning capacity of SFRS. The latter is determined as a function of realistic estimate of system ductility. CSA A23.3 distinguishes between nominal and probable capacity for concrete systems by defining the specific resistance factors for concrete and steel reinforcement as well as the level of stress developed in tension reinforcement (ϕ_c = ϕ_s = 1, f_y for nominal; and ϕ_c = ϕ_s = 1, 1.25f_y for probable). CSA S16-14, on the other hand, explicitly addresses neither the capacity level of SFRS for seismic design of foundations, nor the nominal capacities of its components. Consequently, in this study, the design overturning capacities of the frames were determined using probable tensile and compressive brace resistances.

NCP foundations must satisfy the following design criteria: (i) they should withstand the overturning moment imposed by 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, or (b) 75% of the nominal overturning capacity of the SFRS; (ii) the soil stress must not exceed the factored soil bearing resistance and (iii) the displacement of the superstructure determined for fixed-base conditions,

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increased to account for the impact of foundation rotation, must not exceed the limit prescribed by NBCC 2015 for selected SFRS. The following criteria were considered for the US design approach: (i) the foundation must resist the overturning moment from the combined factored seismic and vertical loading and (ii) the soil stress must not exceed the factored soil bearing resistance. Note that the foundation design in US is done considering allowable bearing stresses in which case the service level of earthquake loading is used (one third of the full load), and the bearing stresses can be augmented by 30 percent for a combined effect of vertical and lateral loading. For consistency, this relaxation was not considered in the present study and the calculations were done with factored load and factored resistances.

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

Footing dimensions (m)	**VCR-C**	**VCR-E**	**Soil properties***	**Site C**	**Site E**
	Capacity-protected foundations (CP)				
Length (L)	15	17.5	q_{ult} (kPa)	3000	400
Width (B)	4	6	q_f (kPa)	1500	200
Depth (d)	1.3	1.5	G (MPa)	100	20
	Not capacity-protected foundations (NCP)				
Length (L)	14	15			
Width (B)	4	6			
Depth (d)	1.3	1.5			
	US design approach				
Length (L)	10	12			
Width (B)	1.35	4			
Depth (d)	1.0	1.5			

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

A summary of soil properties used for the design is given in Table 1. Factored bearing resistances, 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 from the Canadian foundation manual [7]. 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 on dimensions 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 class site. Table 1 also lists the footing dimensions of the two frames studied. For CP and NCP foundations, the overturning moment demand governed footings dimensions for Class C site. For the frames on Class E site the critical parameter for foundation design was the inter-storey frame drift, augmented to include anticipated foundation rotations. In order to avoid the increase in the frame overstrength the excessive drift of the superstructure for NCP foundations was controlled in this study by increasing the foundation dimensions to minimize its rotations. Foundation designed following the US approach was controlled by the bearing soil stress for Class E site while for the Class C site it was possible to optimize simultaneously the bearing resistance and the overturning capacity.

3. Nonlinear time history analysis: Modelling and ground motions

Nonlinear time history analysis (NLTHA) of the soil-foundation-structure system was done using the OpenSEES program [8]. 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 thus permit to explicitly evaluate deformation and force demands imposed by braces to other frame members and foundations.

As recommended by Aguerro [9], 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

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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, a fictitious gravity column was added. 3% Rayleigh damping was specified [10].

A flexible boundary substructure approach [10] was applied to model the behaviour of the soil-foundation system, including rocking and permanent settlement of the foundation. Nonlinear soil-foundation response was represented using the Beam-on-Nonlinear-Winkler-Foundation concept [11]. The foundation was modelled as an elastic beam with a finite number of vertical (q-z type) nonlinear springs. Nonlinear springs were non-uniformly distributed to simulate the rocking behaviour. A variable spring stiffness was used in order to represent the higher reactions that can develop in the end-zones under the vertical loads. The 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 [12]. More detailed information about the frame and soil-foundation modelling is available in [6].

Ground motion records were selected on the basis of the magnitude-distance scenarios that contribute the most to the seismic hazard for the design cases studied [13]. Two distinct ground motion sets were constituted, one for each class site category. Each set was composed of 15 historical ground motions that were grouped in three subsets of five records for each typical tectonic source (crustal, in-slab and interface). All ground motions records were calibrated to match the NBCC design spectra following the procedure described in [14].

4. Results and discussion

The response of the soil-foundation-frame system was examined by tracking the overturning moment at the frame base, the foundation uplift and the settlement of the soil, maximal forces in the nonlinear soil springs and inter-storey drifts. The extent of inelastic activity in braces was also monitored and the principal mechanisms of energy dissipation were identified. The results are expressed as the mean value of the five largest peak response values found for individual ground motion records as recommended in [14].

Table 2– Summary of the results for the overturning moments on foundations

Building	Foundation type	$M_{f\ SSI}$ (kNm)	M_p (kNm)	M_d (kNm)	$M_{f\ SSI}/M_p$	$M_{f\ SSI}/M_d$	M_d/M_p
VCR-C	CP	22162	26552	26552	0,83	0,83	1
VCR-E		26806	31464	31464	0,85	0,85	1
VCR-C	NCP	20645	26552	20971	0.78	1.07	0.79
VCR-E		25519	31464	26544	0.81	1.01	0.84
VCR-C	US approach	15609	26552	10324	0.59	1.50	0.39
VCR-E		21500	31464	14069	0.68	1.53	0.45

In Table 2, the overturning moment demand on the foundation obtained from the analysis ($M_{f\ SSI}$) is compared to the foundation design overturning moment M_d to assess the values used in design. The ratio of M_d to M_p is also given. For CP foundations, the design overturning moment M_d is equal to the probable overturning resistance of the frame M_p. As seen in Table 2, the ratio of the overturning moment demand to design moment ($M_{f\ SSI}/M_d$) varies between 0.83 and 0.85. This result suggests that the design estimates are somewhat conservative and implies that the foundation has sufficient resistance to ensure energy dissipation primarily through inelastic frame response. Indeed, results obtained for brace axial forces and deformations confirm that a significant inelastic activity took place in these elements. On the other hand, the results obtained for foundation uplifts (VCR-C 11.6 mm, VCR-E 4.1mm) and permanent soil settlements (VCR-C 1.8 mm, VCR-E 12.4 mm) indicate that some limited inelastic

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response of the foundation-soil system did occur, including primarily rocking for Class C site frame and a combination of predominant inelastic soil response and some rocking for Class E site frame. The maximum forces in the soil springs reached $0.1q_{ult}$ et $0.46q_{ult}$ for Class C and Class E sites, the values that can be easily accommodated by the soil.

For NCP foundation, ratios of $M_{f\,SSI}$ to M_d are close to one for both site classes, showing that the design foundation moments were well predicted. Inspection of inelastic brace response as well as recorded foundation uplifts (VCR-C 21.4 mm, VCR-E 18.3 mm) and permanent soil settlements (VCR-C 2.1 mm, VCR-E 23.8 mm) confirm that, for both sites, the energy was dissipated jointly in the frame and in the soil-foundation system, thereby limiting the inelastic frame demand. For Class C site frame, rocking behaviour dominated the soil-foundation response, while for Class E site frame the energy dissipation through rocking and inelastic soil deformations was comparable.

The foundations designed following the US methodology, which were the smallest two of the six foundations examined, underwent very important rocking and the inelastic response of the soil underneath was extensive. In fact, all energy dissipation happened in the foundation-soil system, eliminating completely the inelastic frame response. In Fig. 2 the normalized maximum brace compression and tensile forces for VCR-E frame are below one in all cases, confirming that no inelastic activity took place in the superstructure. Similar observation was made for the VCR-C frame, for which only two ground motion records induced small inelastic demand in the top storey braces. Note that sliding behaviour of foundation was not represented in the model because the previous studies conducted by the authors revealed that for CP and NCP foundation this failure mode was not critical. However, considering the size of the foundation, it is possible that for the VCR-C frame, and for the two records that induced some yielding in compression braces, the sliding would most probably have happened first thereby eliminating completely the inelastic response of the superstructure.

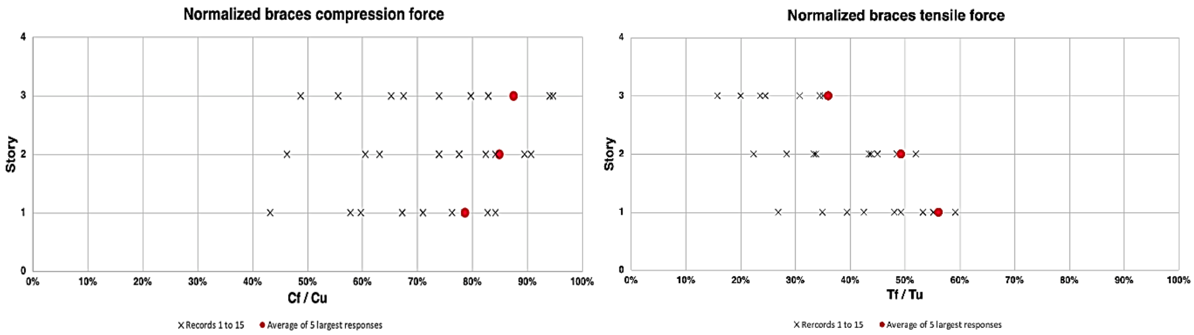

Figure 2. VCR-E frame: Normalized compressive and tensile forces in braces

Such behaviour was anticipated because of a relatively small size of the foundations that resulted from the US design approach; their volume (13.5 m^3 and 72 m^3) was approximately 5.5 and 2 times smaller compared to the volume of CP (78 m^3 and 157.5 m^3) and NCP (72.8 m^3 and 135 m^3) foundations for the VCR-C frame and VCR-E frame, respectively. To draw a picture of foundation-soil behaviour and to assess whether the imposed demand was acceptable or not, the permanent soil settlements and the maximum forces in the soil springs were examined. For the VCR-C frame the energy was principally dissipated through foundation rocking with maximum uplift reaching 38 mm, while the permanent soil settlements remained below 5 mm. However, the springs at the ends of foundation attained the ultimate bearing soil resistance q_{ult}, which level is of great concern even though there is uncertainty about the bearing soil resistance being fully mobilised in view of the small permanent settlement. For the VCR-E frame, the maximum uplift of 53 mm was recorded, while the maximum observed permanent soil settlement was extremely high (143 mm), exceeding by almost 6 times the limit of 25 mm considered in the literature as acceptable [15]. Even though the superstructure remained completely elastic in this case, the foundation was far too small to guarantee a satisfactory seismic response.

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It is interesting to note that in all cases, the maximum inter-storey drifts remained well below the NBCC limit of 2.5%. The largest values were recorded for the frames with foundations designed following the US approach and were equal to 0.6% and 1.2% for the VCR-C and VCR-E frames respectively.

5. Conclusions

In this study, different strategies for seismic design of shallow foundations were examined on the example of a 3-storey steel building with tension-compression X-bracing of medium ductility (MD) type, located in Vancouver, BC, Canada. The study included two soil conditions: very dense soil or soft rock (Class C site) and soft soil (Class E site). Design of the frames was carried out in accordance with Canadian design requirements for steel structures. Foundations were designed in compliance with the Canadian requirements for both capacity-protected and not capacity-protected footing options. For comparison, the US design methodology was also applied whereby no overstrength of the superstructure was considered to determine the design loads for foundations.

The results showed that inclusion of soil-structure interaction reduced the overturning moment demand on the foundation in all cases. For CP foundations, design estimates were on the conservative side and the energy dissipation occurred mainly but not exclusively in the superstructure, contrary to what was foreseen in the design. For NCP foundations, the design estimates of the overturning moment demand on the foundation were accurate and the energy dissipation involved both inelastic frame response and nonlinear response of foundation-soil system. The latter involved rocking that was more prominent for Class C site and the inelastic soil response with permanent settlement that was more noticeable for Class E site. The US methodology resulted in the smallest foundations for sites considered and design moments were underestimated by about 35%. For both sites, the energy dissipation was observed in foundation-soil system and the superstructure remains elastic. For Class C site, most of the seismic energy was converted to kinematic energy developed by foundation rocking and very small permanent settlement were observed. However, the soil springs at foundation ends reached the ultimate bearing resistance, which is of great concern as the seismic performance of the superstructure can be jeopardized. For Class E site, excessive permanent settlement that were observed would fully compromise the integrity of the superstructure. The results of this study suggest that some overstrength needs to be considered in seismic design of shallow foundations for steel braced frames, but smaller than what is suggested in Canadian design procedures. Determining the most appropriate value is the subject of an ongoing study.

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