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THE EFFECTS OF SOIL-STRUCTURE INTERACTION ON THE SEISMIC DESIGN OF SHALLOW FOUNDATIONS FOR EBFs

Sanda Koboevic (1), Bardia Ramin Afshar (2)

(1) *Associate professor, Department of Civil Engineering, Polytechnique Montreal, Montreal, QC, Canada, email: sanda.koboevic@polymtl.ca*
(2) *Former M.Sc. student, Department of Civil Engineering, Polytechnique Montreal, Montreal, QC, Canada, email: ushanthan.murugananthan@gmail.com*

Abstract

Past research has demonstrated that foundation rocking can reduce demands on the superstructure by dissipating a portion of earthquake energy through soil-structure interaction (SSI). Canadian seismic provisions allow two approaches for foundation design: capacity-protected (CP) and not-capacity-protected (NCP). CP foundations are designed to fully develop the resistance of the seismic force-resisting system (SFRS), thereby concentrating inelastic activity within the superstructure. In contrast, NCP foundations are designed to develop only a partial capacity of the SFRS, leading to foundation uplift and rotation, which limit the forces transmitted to the superstructure. In practice, Canadian design often results in large foundations for steel frame buildings, significantly increasing construction costs and potentially favouring alternative structural materials. Previous studies on concentrically braced frames have shown that, on competent soils, such large foundations may not be necessary, and satisfactory performance can be achieved even without considering frame overstrength in design.

To further validate the applicability of Canadian seismic provisions for foundation design of steel frames, this study investigates 3- and 8-storey steel buildings with eccentrically braced frames located in Vancouver, Canada. Two site classes (C and E) were considered. Foundation designs followed Canadian seismic design provisions (NBCC 2020 and CSA A23.3-19), with an additional comparison to a design approach without capacity considerations, similar to US practice. Non-linear time history analyses were conducted using the STKO/OpenSees platform, incorporating both inelastic frame behavior and nonlinear soil response. The results indicate that for foundations on competent soil (site class C), even without applying capacity design principles, the inter-storey drift and soil capacity limits were satisfied. This design approach resulted in smaller foundations and reduced ductility demands on the superstructure.

Keywords: Eccentrically braced frames, Seismic foundation design, Soil-structure interaction, Nonlinear time-history analysis.

1. Introduction

The foundation of a structure plays a key role in seismic performance, acting as the interface between the superstructure and the ground to transfer earthquake forces. Traditional foundation design aims to keep the foundation elastic and undamaged, directing energy dissipation into the superstructure. The required foundation strength varies depending on the seismic force resisting system (SFRS) and design philosophy. In Canada, foundations with unrestrained rotations can be designed as capacity-protected (CP) or not-capacity-protected (NCP). CP foundations must resist an overturning moment corresponding to the full capacity of the SFRS, ensuring energy dissipation occurs only in the superstructure. The SFRS capacity depends on the ductility level of the system. In contrast, NCP foundations do not need to fully develop the SFRS capacity. Instead, they allow uplift and rocking, and can limit force demands on the structure but introduce more prominent foundation rotations and increased inter-storey drifts, requiring careful structural design.

Eccentrically braced frames (EBFs) provide an effective balance of stiffness, strength, and ductility by dissipating seismic energy through inelastic deformations in a beam segment called the link. All links are expected to yield simultaneously, creating a shear yielding mechanism that influences force demands on all protected elements, including foundations. Previous studies on concentrically braced

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frames (CBFs) [1,2] suggest that current Canadian foundation design provisions, originally developed for reinforced concrete shear walls, may not fully account for distributed yielding, sometimes resulting in overly conservative foundation designs. Given that EBFs exhibit the same yielding mechanism, albeit differently achieved, it is crucial to assess whether existing foundation design requirements remain appropriate or require modification.

This study evaluates the applicability of Canadian seismic foundation design provisions, provided in National Building Code of Canada [3], and reinforced concrete design standard CSA A23.3-19 [4], for EBFs through nonlinear time history analyses of 3- and 8-storey shear-link EBF buildings designed for Vancouver, Canada. Two soil categories were considered. For comparison, an alternative foundation design omitting capacity requirements, similar ti U.S. practice, was examined. The soil-foundation-structure response was analysed using STKO/OpenSees, incorporating inelastic frame behaviour and nonlinear soil response. A fixed-base scenario was also analysed for comparison. The effects of soil-structure interaction (SSI) on the frame-foundation-soil system were evaluated by examining overturning moments, foundation uplift, permanent soil settlements, maximum spring forces, foundation rotations, shear forces in yielding links, and inter-story drifts.

2. Frame and foundation design

2.1. Design of the superstructure

Chevron-type EBFs with shear-critical links were designed for 3- and 8-storey buildings in Vancouver, BC (VCR3 and VCR8), considering Class C (very dense soil or soft rock; 360 m/s $\leq v_s \leq$ 760 m/s) and Class E (soft soil; $v_s \leq$ 180 m/s) site conditions. In the studied N-S direction, seismic resistance is provided by four EBFs, with a system ductility factor $R_d = 4$ and a system overstrength factor $R_o = 1.5$. In the E-W direction the buildings are braced by perimeter moment-resisting frames.

Fig. 1 illustrates the typical floor plan of the buildings and the design gravity loads. The braced bay width is 8 m, with typical storey heights of 4 m and a first-storey height of 4.5 m, resulting in total building heights of 12.5 m for the 3-storey frames and 32.5 m for the 8-storey frames. A link length of 600 mm was selected. Brace-to-beam connections were designed as moment-resistant, while all other connections were considered pinned.

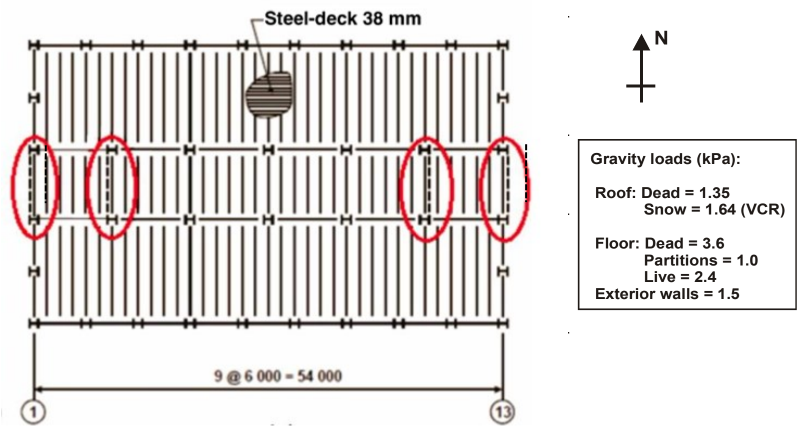

Figure 1. Plan view of the studied buildings and design gravity loads

The frames were designed in accordance with NBCC 2020 [3] and Canadian standard for Steel structures CSA S16-19 [5]. The seismic design base shear was determined using response spectrum analysis under fixed-base conditions, as NBCC 2020 does not require consideration of soil-structure interaction in design. Dynamic base shear was calibrated against $0.8V_d$, as permitted for regular structures. The seismic base shear V_d was determined using the NBCC 2020 equivalent static procedure: $V_d = S(T_a)M_vI_EW / (R_dR_o)$, where $S(T_a)$ is the design spectral acceleration at the fixed-base fundamental

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period T_a, M_v accounts for higher mode effects, I_E is the importance factor, and W is the seismic weight. The fundamental periods from modal analysis (VCR3-C: 0.54 s, VCR3-E: 0.45 s, VCR8-C: 1.33 s, VCR8-E: 0.98 s) were used to calculate V_d, as they exceeded the NBCC 2020 empirical values (VCR3: 0.31 s, VCR8: 0.81 s) but remained within the allowable limit of twice the empirical period. The resulting seismic design base shears were 761 kN, 1057 kN, 1148 kN, and 2455 kN for VCR3-C, VCR3-E, VCR8-C, and VCR8-E, respectively.

Frame members were sized to meet ductility requirements and then checked for all relevent load combination including wind and seismic loading. Links were selected from W-shapes having sufficient inelastic shear resistance for factored seismic loads, including P-Delta effects. Braces and outer beams were designed as beam-columns, resisting forces from $1.3R_y$ times the nominal link shear resistance, V_p. Beams were selected from W-shapes and braces from an HSS section database, with beams assumed fully laterally supported out-of-plane by floor slabs. Bending moments combined with axial tension governed outer beam design, while brace design was controlled by compressive axial loads and bending moments. At link ends, bending moments were initially distributed between the brace and outer beam based on their elastic flexural stiffness. When the moment assigned to the outer beam exceeded its resistance, the remaining moment was transferred to the brace, allowing yielding of the outer beam segment. Columns were considered continuous over the building height. Axial forces from gravity loads were combined with forces from yielding links ($1.3R_yV_p$ for top tiers, $1.17R_yV_p$ for lower tiers). Bending moments due to column continuity and storey drifts were also considered per CSA S16-19.

2.2. Design of foundation

The overturning capacity of CP foundations is determined based on the ductility of the SFRS. If the ratio R_dR_o/γ_w exceeds 2.5, the foundation must resist the lateral load corresponding to the probable overturning capacity of the SFRS, in addition to the applied factored gravity load. In this expression, γ_w represents the system overstrength. Conversely, NCP foundations are not required to develop the full capacity of the SFRS. Instead, their overturning resistance must be sufficient to withstand factored gravity loads and the greater of: (i) 75% of the nominal overturning capacity of the SFRS, or (ii) the overturning moment resulting from the design load combinations that include earthquake effects, calculated using $R_dR_o = 2$.

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

	Footing dimensions (m)	VCR-C	VCR-E
3-storey EBF		Capacity-protected foundations (CP)	
	Length (L)	11.2	16.8
	Width (B)	4.5	6
	Depth (d)	1.3	1.7
		No capacity design (NCD)	
	Length (L)	11.2	16.8
	Width (B)	2	5
	Depth (d)	1.3	1.7
8-storey EBF		Capacity-protected foundations (CP)	
	Length (L)	16	24
	Width (B)	6	12
	Depth (d)	1.5	2
		No capacity design (NCD)	
	Length (L)	16	24
	Width (B)	3.5	9
	Depth (d)	1.5	2

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

For all EBFs studied, the probable overturning moment ($M_{probable}$) was found to be lower than the overturning moment calculated using $R_dR_o = 2$, so all foundations were designed as CP foundations. The impracticality of NCP foundations can be attributed to the high ductility of the EBF systems. The

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design overturning capacities of the frames were determined based on full probable link resistances, considering direct summation. For comparison, an alternative design (NCD) was conducted similarly to the US design approach, in which capacity considerations were not included to determine foundation design forces.

Table 1 presents a summary of the soil properties used for design, along with the final footing dimensions for the studied cases. The factored bearing resistances (q_f) for site classes C and E were derived from field data, while the ultimate bearing resistance (q_{ult}) and shear modulus (G) were determined using the Canadian Foundation Engineering Manual [6]. Although q_{ult} varies with foundation dimensions, an analysis of the soil friction angles considered in this study indicated that its impact on foundation sizing was negligible. As a result, as shown in Table 1, the same factored bearing resistance values were applied to all foundations within the same site class.

3. Modelling and ground motion selection for non-linear time history analysis

Nonlinear time history analysis (NLTHA) of the soil-foundation-structure system was done using OpenSeeS [7]. Scientific Toolkit for OpenSees (STKO) [8], an advanced GUI, was used to facilitate the preparation of the OpenSees models and the visualisation of the analysis results.

The links were modelled using force-based fiber elements, with a section aggregator included to account for nonlinear shear behaviour. The shear response was represented using the Steel02 material model, where the yield strength was defined as the probable link shear resistance ($V_{pr}=R_yV_p$, where $R_y = 1.1$), and the initial shear stiffness was given by GA_v, with G being the shear modulus and A_v the shear area of the link. The strain hardening ratio was set to 0.003. The isotropic hardening parameters (a1, a2, a3, a4) were assigned values of 0.033, 1, 0.033, and 1.0, respectively, while the elastic-plastic transition parameters (R0, cR1, cR2) were set to 18.5, 0.925, and 0.15 following the recommendations by Yang et al. [9] who verified the model's accuracy by comparison with experimental data from tests conducted by Dusicka and Lewis [10].

The outer beam segments and diagonals were modelled using eight nonlinear beam-column elements with fiber discretization of the cross-section. This approach allowed for the representation of cross-sectional yielding and flexural buckling. Each element contained four integration points, and the cross-section was modeled using 16 fibers, as recommended by Agüero et al. [11]. To account for the end restraint conditions induced by the gusset plates, rotational spring elements were incorporated into the model. In contrast, the columns were modelled using elastic beam-column elements.

A flexible boundary substructure approach was employed to model the soil–foundation system, capturing rocking and permanent settlement. The nonlinear soil–foundation interaction was represented using the Beam-on-Nonlinear-Winkler-Foundation (BNWF) model. The foundation was idealized as an elastic beam supported by a finite number of vertical nonlinear springs of the q-z type. To account for the increased reactions that develop in the end zones under vertical loads, the springs were assigned variable stiffness. Following the recommendations in [12], the footing end-length ratio (L_{end}/L) was set at 20%, and a spring spacing ratio (Ie/L) of 4% was chosen, ensuring a minimum of 25 springs along the footing length. Further details on the frame and soil–foundation modelling can be found in [13].

Ground motion records were selected based on the magnitude-distance scenarios that have the most significant contribution to the seismic hazard for the studied design cases [14]. To ensure a representative selection, two distinct ground motion sets were established, corresponding to site class categories selected for this study. Each set consisted of 15 historical ground motion records, categorized into three subsets of five records, representing the dominant tectonic sources for Vancouver: crustal, in-slab, and interface earthquakes. The selected ground motions were then scaled and calibrated to match the NBCC 2020 design spectra, following the methodology outlined in [15]. This approach ensured consistency in spectral characteristics while preserving the natural variability of the recorded motions.

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4. Results and discussion

The response of the superstructure–foundation–soil system was evaluated based on link shear force demand, inter-story drifts, foundation overturning moments, foundation uplift, soil settlement, and maximum forces in the nonlinear soil springs. For comparison, results were also obtained for the fixed-base (FB) system. The values presented in this section correspond to the mean of the five largest peak response values from 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)	$M_{f\,FB}$ (kNm)	M_d (kNm)	$M_{f\,SSI}/M_{f\,FB}$	M_d/M_{FB}
VCR3-C	CP	18338	18472	16719	0.99	0.91
VCR3-E		27685	28201	23448	0.98	0.83
VCR8-C		69549	71318	66735	1.04	0.94
VCR8-E		145148	151432	126472	0.96	0.84
VCR3-C	NCD	16055	18472	7672	0.87	0.42
VCR3-E		29110	28201	11189	1.03	0.40
VCR8-C		65634	71318	30108	0.92	0.42
VCR8-E		136023	151432	67273	0.90	0.44

To evaluate the impact of including soil-structure interaction (SSI) in the analysis, Table 2 compares the overturning moment demand on the foundation obtained from the analysis of superstructure-foundation-soil system ($M_{f\,SSI}$) with the foundation overturning moment obtained from nonlinear time-history analysis (NLTHA) for fixed-base conditions ($M_{f\,FB}$). The ratio of design moment M_d to $M_{f\,FB}$ is also provided to validate design predictions. For capacity-protected (CP) foundations, the design overturning moment M_d corresponds to the probable overturning resistance of the frame (M_p), considering that all shear links have reached their probable shear resistance ($1.3R_yV_p$). In contrast, for foundations designed without capacity considerations (NCD), the design overturning moment is determined at the base of the frames using response spectrum analysis.

As shown in Table 2, the ratio of design moment to the fixed-based overturning moment demand ($M_d/M_{f\,FB}$) varies between 0.83 and 0.94, suggesting that the design estimates are somewhat unconservative. This underestimation is likely due to greater strain hardening in yielding links than anticipated in the design. Indeed, the maximum link shear forces reveal that strain-hardening factors recorded in fixed-base analyses exceed the 1.3 assumed in design by approximately 10% for 3-storey frames (1.41) and 20% for 8-storey frames (1.6). The ratio $M_{f\,SSI}/M_{f\,FB}$ remains close to one for CP foundations on Class C site, indicating that the foundation provides sufficient resistance to ensure energy dissipation primarily through inelastic frame response. However, for the 8-storey frame on a Class E site, a lower $M_{f\,SSI}/M_{f\,FB}$ ratio suggests increased energy dissipation through the foundation-soil system. As the foundation size decreases (NCD foundation), $M_{f\,SSI}/M_{f\,FB}$ also decreases, indicating a greater contribution of the soil-foundation system to seismic energy dissipation. This trend is further supported by the analysis of maximum link shear forces, which reveals that smaller foundations result in reduced inelastic demand on the links and smaller strain-hardening factors (1.33 and 1.47 for 3- and 8-storey frames, respectivelly). These reductions confirm that energy dissipation is distributed between the superstructure and the foundation-soil system, rather than being primarily concentrated in the frame.

The results for foundation uplift in 3-storey frames with CP foundations (VCR3-C: 1.5 mm, VCR3-E: 1.8 mm) and permanent soil settlements (VCR3-C: 2.2 mm, VCR3-E: 17 mm) suggest that some inelastic response of the foundation-soil system occurred. On the Class C site, such response was minimal and primarily involved limited rocking. In contrast, on the Class E site, the response was more pronounced, combining inelastic soil deformation with some rocking.

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As the size of the NCD foundation decreased, the impact of soil-structure interaction became more evident. On the Class C site, the foundation experienced increased uplift (21 mm), while on the Class E site, greater permanent settlement was observed (29 mm), slightly exceeding the acceptable limit of 25 mm. For the Class C site, the maximum forces in the soil springs remained within acceptable limits, reaching $0.13q_{ult}$ for the CP foundation and $0.33q_{ult}$ for the NCD foundation. On the Class E site, the recorded values were higher ($0.6q_{ult}$ for CP and $0.65q_{ult}$ for NCD) but still considered acceptable, particularly since these peaks occurred at extreme foundation springs and only for very short durations.

For 8-storey frames, regardless of site class or foundation type, the maximum recorded spring forces were similar to those observed in 3-storey frames. As expected, uplift and permanent soil settlements were greater than those recorded for the 3-storey frames, particularly for the NCD foundation. However, for frames on the Class C site, these values remained well within acceptable limits, with maximum uplift reaching 24 mm and maximum permanent settlement reaching 8 mm for the system with an NCD foundation. For Class E site, on the other hand, for which inelastic soil deformations dominated the response, high values of permanent soil settlement were observed (61mm and 101 mm for CP and NCD foundations, respectively), exceeding the acceptable values by a large margin.

One of the unique aspects of seismic foundation design in Canada is the requirement to account for the increase in inter-story drifts caused by the rotations of unanchored foundations in the superstructure design. Previous studies on concentrically braced frames (CBFs) [1,2] have shown that this increased drift can become a governing factor in frame design, potentially leading to larger foundation sizes to mitigate the effects of foundation rotations.

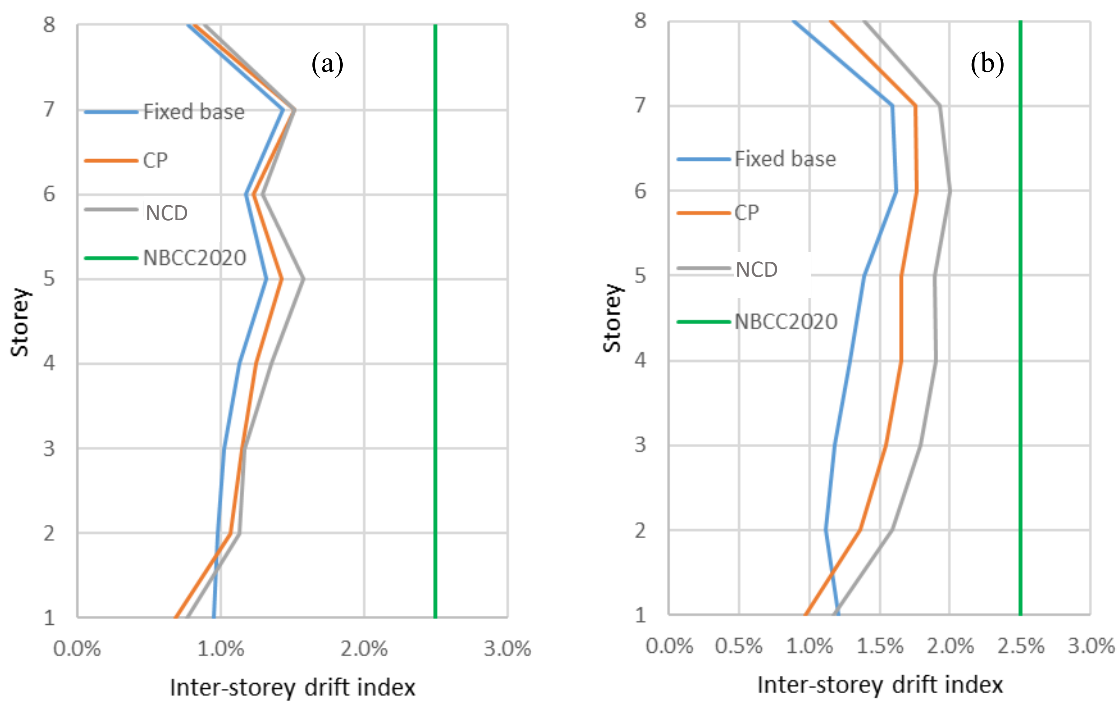

Figure 2. Inter-storey drift index for Vancouver 8-storey frame on (a) Class C and (b) Class E sites

In Fig. 2 inter-story drift ratios for 8-storey frames with CP and NCD foundations, obtained from nonlinear time-history analysis (NLTHA) and normalized by the story height, are compared to the 2.5% limit specified in the National Building Code of Canada (NBCC). For reference, results for the fixed-base scenario are also included. As the foundation size decreases, inter-story drift increases. For the 8-storey building on a Class C site, the highest inter-story drift occurs at the 5th level, reaching

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approximately 1.6%. On the Class E site, the maximum drift index of 2% is recorded at the 6th level. Despite these variations, all drift values remain within the NBCC limit. For the 3-storey frames, in all cases studied, drift values remain well below the design limit, with a maximum of approximately 1%.

5. Conclusions

In this study, Canadian seismic foundation design requirements are applied to 3-storey and 8-storey steel buildings with eccentrically braced frames to assess their applicability to steel structures. The frames are designed for Vancouver, considering both Class C (competent soil) and Class E (soft soil) site conditions. Only capacity-protected (CP) foundations, sized to develop the full capacity of the frame, could be designed under Canadian provisions. For comparison, foundations were also designed without capacity considerations (NCD), following an approach similar to U.S. practice. The seismic response of the building-foundation-soil system was investigated through nonlinear time-history analysis using the OpenSees software platform. The numerical model incorporated inelastic frame behaviour and nonlinear soil response. A fixed-base scenario was also analysed for comparison. The effects of soil-structure interaction (SSI) on the frame-foundation-soil system were evaluated by examining overturning moments, foundation uplift, permanent soil settlements, maximum spring forces, foundation rotations, shear forces in yielding links, and inter-story drifts.

The following conclusions can be drawn from this study:

- Contrary to previous observations for concentrically braced frames (CBFs), non-capacity-protected (NCP) foundations were not a viable design option for the eccentrically braced frames (EBFs) studied. This limitation stems from the higher ductility coefficients assigned to EBFs compared to CBFs, which lead to lower design moments for fully developing the seismic force-resisting system's capacity. As a result, these moments fall below the minimum required for NCP foundation design, making the approach impractical.
- For both CP and NCD foundations, energy dissipation occurred through a combination of superstructure response and foundation-soil interaction. In CP foundations, the overturning moment demand closely matched fixed-base predictions, particularly on Class C sites, indicating that the foundation provided sufficient resistance to concentrate energy dissipation primarily within the inelastic response of the frame. In contrast, for NCD foundations, the reduction in overturning moment demand led to a greater share of seismic energy being dissipated through foundation rocking and inelastic soil deformation. While inelastic demand on the frame also decreased, the reduction was less significant than previously observed for CBFs on Class C sites when foundation dimensions were reduced.
- Inter-story drift values remained below the NBCC limit in all cases, though drift increased with greater foundation flexibility. For 8-storey frames, the highest drift occurred at mid-height levels, with Class E sites exhibiting slightly larger values than Class C. However, even with NCD foundations, drift remained within the 2.5% NBCC limit. This suggests that strict CP foundation design may not always be necessary for drift control and raises questions about the accuracy of current methods for estimating the impact of foundation rotations on inter-story drifts.
- The results suggest that for steel frames on competent soils, applying CP foundation principles may lead to over-conservative foundation sizing, increasing construction costs without significant performance benefits. Allowing some level of foundation flexibility through NCD design could lead to cost-effective solutions while maintaining adequate seismic performance. However, on softer soils, careful consideration is needed, as excessive soil deformations could compromise overall structural performance.

While these findings provide valuable insights into the seismic response of eccentrically braced frames with different foundation designs, certain limitations should be acknowledged. This study focuses on two specific site classes (Class C and Class E), which may not fully represent the wide range of soil conditions encountered in practice. The findings are based on ground motions calibrated for Vancouver,

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meaning the results may not be directly applicable to other seismic regions with different soil characteristics and tectonic settings. Ongoing research aims to extend these analyses to a broader range of soil conditions and seismic environments to further generalize the conclusions.

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