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ROCKING RESPONSE OF TALLER REINFORCED CONCRETE WALLS

Marc-André Lemieux[1] and Sanda Koboevic[2]

[1] École Polytechnique de Monteal
Montreal (Quebec) Canada
marc-andre.lemieux @polymtl.ca

[2] École Polytechnique de Montreal
Montreal (Quebec) Canada
sanda.koboevic@polymtl.ca

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Abstract. *The study of seismic response of taller reinforced concrete shear walls with rocking foundations is carried out on the example of a 20-storey building. The building is framed by simple ductile concrete shear and located in Montreal, Canada. The location is representative of Eastern North American seismic conditions characterized by high-frequency ground motions that can exemplify the impact of higher modes on the response. The walls are designed and detailed according to the Canadian design provisions for ductile shear walls. Different options for the foundation design are considered to examine the effect of foundation size on the type of structural response. Two-dimensional nonlinear time history analyses of the wall-foundation system were performed using the OpenSees program for simulated ground motions compatible with the design spectrum. Wall inelastic behaviour and the nonlinear soil response were both represented in the model. Soil-structure interaction and uplift due to overturning are modeled using the beam-on-nonlinear-Winkler-foundation concept. The overturning moments, base shears and roof displacements are compared to those obtained assuming a fixed-base condition to quantify the impact of rocking on the superstructure. The usefulness of rocking to mitigate higher-mode effects is also evaluated. The foundation displacements and stresses in the soil are examined to assess the consequences of the rocking behaviour on the global structural response. The analyses show that the code compliant design exhibited yielding at the wall base and rocking. Reduction of foundation size increased the rocking response and eliminated the inelastic response in the superstructure. Rocking reduced the force demand on the superstructure without significantly increasing the displacements, but some permanent deformations and high bearing pressures developed in the soil under the foundation edges. Rocking did not reduce the impact of higher modes on the response.*

1 INTRODUCTION

Capacity-based approach to seismic design of buildings generally aims to confine energy dissipation in the superstructure and avoid any damage to the foundations. This design philosophy can be justified by the difficulty to conduct inspection of the foundations following an earthquake and the high cost associated with possible repairs. Conversely, such approach neglects the great potential to engage nonlinearity inherent to the foundation-soil system through foundation yielding, the mobilisation of bearing capacity failure mechanisms in supporting soil or the uplifting at foundation-soil interface which can efficiently reduce global seismic demand to the structural system [1,2,3,4].

Incorporating soil-structure interaction effects to reduce seismic design loads has been allowed to certain extent in modern building codes. Some of the related provisions imply for instance that uplift due to rocking will eventually occur during the design earthquake and thus limit seismic forces introduced into the superstructure. However, this is not the primary mechanism of energy dissipation intended in the design. In Canada, design procedures for reinforced concrete shear walls [5] favour the formation of flexural plastic hinges at the base of the structure. Foundations are sized to carry overturning moments corresponding to wall nominal bending moment capacity, the corresponding shear force and the tributary gravity loads. Design forces are limited to 50% of the elastic seismic shear which corresponds to load level at incipient rocking [6]. The latter usually applies to design foundations of overstrong walls for which minimum ductility requirements are critical.

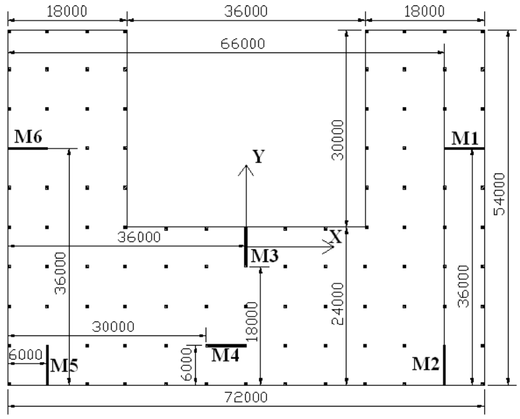
For taller buildings located in moderate seismic zones such approach generally results in excessively large foundations. Modifying design approach by favoring rocking as a principal mechanism of limiting seismic forces in the superstructure could be beneficial for such structures. Negative consequences are also possible: horizontal displacements of the structure may be increased beyond acceptable limits and permanent deformations can develop in the soil. Further studies of global structural response using adequate analytical models and experimental studies are therefore needed before such procedures can be safely implemented in the design.

The study presented in this paper was conducted with the objectives to understand better the seismic response of taller reinforced concrete shear walls with rocking foundations. A studied 20-storey building was located in Montreal and framed by simple ductile reinforced concrete shear walls. This location is representative of Eastern North American seismic conditions with characteristic high-frequency ground motions that can exemplify the impact of higher modes on the response. The walls were designed and detailed following the Canadian design provisions for ductile shear walls. Different options for foundation design were considered to examine the impact of foundation size on the structural response.

Two-dimensional nonlinear time history analyses of the wall-foundation-soil system were performed using the OpenSees program for a set of simulated ground motions compatible with the design spectrum. The model included the wall inelastic behaviour and the nonlinear soil response. Soil-structure interaction and uplift due to overturning were modeled using the beam-on-nonlinear-Winkler-foundation concept. The overturning moments, base shears and roof displacements are determined and compared to those obtained assuming a fixed-base condition to quantify the impact of rocking on the superstructure. The impact of rocking to higher-mode response is also evaluated. The foundation displacements and stresses in the soil are examined to assess the consequences of rocking behaviour on the global structural response.

2 BUILDING DESIGN

The study was conducted for 20-storey building designed for Montreal, Quebec, on a reference class C site (360m/s ≤ v_s ≤ 760m/s). Montreal is located in moderate seismic zone with spectral accelerations for reference class site varying between 0.64g and 0.048 g for 0.2 s and 2 s periods, respectively. The building floor plan is shown in Figure 1. Three ductile rectangular reinforced concrete walls, 9 m long, provide lateral resistance in each orthogonal direction. Wall thickness varies between 400mm for walls M1, M3 and M6 and 500 mm thickness for walls M2, M4 and M5. Typical storey height is 2.95 m with 3.45 m at the first storey giving a total building height, h_n = 59.5 m.



Gravity Loads

Roof:	Dead	= 6.20 kPa
	Toiture	= 1.00 kPa
	Snow	= 2.48 kPa
Floor:	Dead	= 5.20 kPa
	Partitions	= 1.00 kPa
	Live	= 1.90 kPa
Exterior walls		= 1.20 kPa

Figure 1. Layout of a typical floor (dimensions in mm) and design gravity loads

Design was performed in accordance with the provisions of NBCC 2010 [7]. Gravity loads are given in Figure 1. The 3D response spectrum analysis was employed to determine seismic load effects. NBCC 2010 does not require the inclusion of soil-structure interaction in the design, thus in the analysis the fixed-base support conditions were represented. Accidental in-plane torsion was included. To account for concrete cracking, gross section properties of the wall were reduced by 30% reflecting the axial load present at the wall base.

In order to engage 90% of the seismic mass of the building, as required by the NBCC, it was necessary to consider 9 modes. The first two modes (T_1 = 5.08 s, T_2 = 4.81 s) are predominantly flexural and mobilize 54% and 64% of the mass, respectively, while the participating mass of the third torsional mode (T_3 = 4.55 s) is 54%. This confirms that the impact of higher modes on the building response can be anticipated. In spite of the characteristic U-shape, the building was not irregular in torsion (B < 1.7), and no other irregularities were present. Thus, as permitted by NBCC 2010 for regular buildings, the seismic base shear from dynamic analysis was increased to 80% of the design base shear, V, calculated by the equivalent static method. The design base shear, V, is obtained by dividing the elastic base shear, $V_e = S(T_a)\ M_v\ I_E\ W$, by the product of ductility and the overstrengh-related force modification factors, R_d and R_o. In this expression, $S(T_a)$ is the design spectral acceleration for the given site 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. The fundamental periods obtained from modal analysis exceeded the code suggested empirical value by a large margin (T_{emp} = 1.07 s), and thus the maximum period permitted by NBCC for this type of structures, $T_a = 2T_{emp}$ was used to determine V. The calculation of seismic base shear is summarized in Table 1. Because of the limitation imposed on the design fundamental period, the minimum design shear obtained for periods larger than 4.0 s did not control the design.

Seismic loads induced the largest bending moments in the shear walls M2, M5 and M4. Because of the symmetric arrangement, forces induced in walls M2 and M5 were identical. To maintain consistency with the previous study that investigated the rocking behavior of a 10-storey wall [8], further design and nonlinear time history results were carried out for these two walls. Forces considered for the design of walls M2 and M5 are shown in Table 1.

Fundamental period (s)	5.08
Design period (s) (T_a=2*T_{emp})	2.14
Spectral acceleration at the design period (g), S(T_a)	0,046 g
Higher mode effects amplification factor (M_v)	3
R_o	1.6
R_d	3.5
Seismic weight (kN)	486530
Building design base shear (kN) *	9652
Design base shear for M2 and M5 (kN)*	4355
Design bending moment for M2 and M7 (kNm)*	85319
Axial load in the wall at ground floor level (1.0D+0.5L+0.25S) (kN)	14090

* includes accidental torsion

Table 1. Seismic load calculations and design seismic forces at the wall base

The design was carried out in accordance with the seismic requirements of CAN/CSA A23.3 standard [5] for ductile shear walls. The design procedure is capacity-based and aims to achieve the formation of flexural plastic hinge only at the base of the structure and prevent brittle shear failure as well as any damage to the foundations. The compressive concrete strength, f'_c, was 30 MPa, and the yield strength of the reinforcement, F_y, was 400 MPa. Plastic hinge zone extended to the sixth floor. The design shear force in the zone of plastic hinge was set equal to the value that corresponded to the development of the probable moment capacity at the base of the wall (ϕ_c = 1.0, ϕ_s = 1.0 and 1.25f_y) because this value was smaller than the elastic shear force. Outside of this zone, the bending moments and shear forces were amplified by the ratio of the factored bending resistance of the wall at the top of the plastic hinge to the factored seismic bending moment at the same location. The required flexural reinforcement was determined using the program RESPONSE2000 [9], taking into account the tributary gravity loads. Minimum reinforcement requirements did not govern the design. Note that the selected design did not fully comply with rotational ductility requirements prescribed by [5]. This condition was not problematic in view of the seismic response observed in subsequent nonlinear time history analysis.

The size of the foundation was determined using the capacity design. According to the Canadian code, the foundations are designed for the lesser of the shear force corresponding to the development of nominal bending moment capacity at the wall base (ϕ_c = 1.0, ϕ_s = 1.0 and f_y) and 50% of the elastic seismic base shear. The latter corresponds to the force level at which the rocking movement is anticipated. Such approach recognises that the foundation rocking can occur but prioritizes energy dissipation in the superstructure. For the wall under study, development of the nominal flexural capacity of the wall base was critical suggesting that the dissipation of seismic energy should occur in the superstructure without any noticeable rocking. This level of load corresponds to elastic base shear reduced by R_oR_d = 4.74.

In order to study further the impact of the foundation dimensions on the seismic behaviour of the soil-structure system, two alternate solutions were conceived. In the first option, the foundation was designed considering the overturning moment which corresponded to the initiation of rocking ($R_oR_d = 2$). In the second option, the foundation was designed without applying any capacity design procedure, thus the level of load was identical to that considered for the flexural design of the wall base ($R_oR_d = 5.6$). Two soil profiles, typical for the Montreal area [10], and representative of a class C site were considered to investigate the possible impact of different soil characteristics on the seismic response of the soil-structure system. The shear wave velocity of the selected profiles corresponded to the average (v_s = 550 m/s) and lower bound (360 m/s) shear wave velocity characterizing the design site. The typical properties of selected soil profiles are summarized in Table 2.

	Soil profile INF (v_s = 360 m/s)	Soil profile SUP (v_s = 550 m/s)
Standard penetration index, N_{60}	≈ 50	≈ 90
Poisson ratio, ν	0.3	0.25
Mass density, γ_t (kg/ m^3)	2100	2300
Angle of internal friction, ϕ' (°)	41	43
Young's modulus (static), E (MPa)	160	410
Young's modulus (dynamic), E' (MPa)	700	1780
Dynamic shear modulus, G_{max} (MPa)	270	710

Table 2.Characteristic properties for the selected soil profiles

The design of the foundation followed the recommendations specified in the Canadian Foundation Manual [11]. The size of the footing depends on the tributary vertical load, overturning moment and the soil bearing capacity. To obtain a realistic bottom flexural reinforcement of the foundation, the depth of the foundation, H, was determined considering that the bars should not be larger than M35 and not spaced apart more than 150 mm c/c. Identical foundation cantilever offsets, d_b and d_l were assumed in the directions parallel and perpendicular to the wall and the thickness. The soil factored bearing capacity, q_r, was taken as 50% of the ultimate bearing capacity, q_u, and it was assumed that the soil pressure due to the vertical force and the overturning moment are uniformly distributed over the length B_{eff} = L-2e. In this expression, L is the length of the footing, and e is the eccentricity of the vertical load with respect to the center of the foundation ($e = M_f/P_f$). The design parameters and the foundation dimensions obtained for soil profile INF are listed in Table 2.

Force level	P_f (kN)	M_f (kNm)	L (m)	B (m)	H (m)	q_r (kPa)	Settlement (mm)
$R_oR_d = 2.0$	27287	238 993	18.8	10.3	2.90	2176	9.17
$R_oR_d = 4.7$	14209	100 474	14.3	5.75	1.45	1292	8.35
$R_oR_d = 5.6$	14178	85 319	13.5	5.0	1.30	1137	8.74

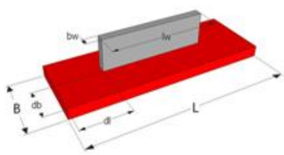


Table 3.Summary of foundation designs

3 NONLINEAR TIME HISTORY ANALYSIS

3.1 Selection of earthquake records

The seismic response of the walls was studied using 2-D nonlinear time history analysis (NHTA) for two sets of 10 ground motion records compatible with the design NBCC spectra. To compensate for the lack of historical earthquake records representative of seismic hazards in Eastern Canada, simulated records were employed. The records were selected and scaled using the procedure described in [12]. The initial selection was done with respect to M-R scenarios that dominate seismic hazard at design locations. In Canada seismic risk desegregation is not available for the spectral accelerations, S_a at periods exceeding 2 s, so the dominant scenarios were established from data given for 2 s period. The first set was composed of 10 records selected from the ground motion records with **M**7.0 at 25 km, and the second set consisted of 5 records **M**6.0 at 15 km and 5 **M**7.0 at 25 km. In the selection process, for each candidate ground motion record, the ratio between the NBCC target spectral acceleration, SA_{targ}, and the response spectral acceleration, SA_{sim}, is first determined at every characteristic period within the period range from 1 to 5 s, and the mean and standard deviation values are calculated. This range was selected to include the periods of the modes that engage 90% of seismic mass. The final selected records have the lowest standard deviation and are scaled by the mean SA_{targ}/SA_{sim} ratio. The scaling factors varied between 0.71 and 1.02 for the first set, and 0.89 and 1.48 for the second set. These values are in line with the recommendations provided in [12].

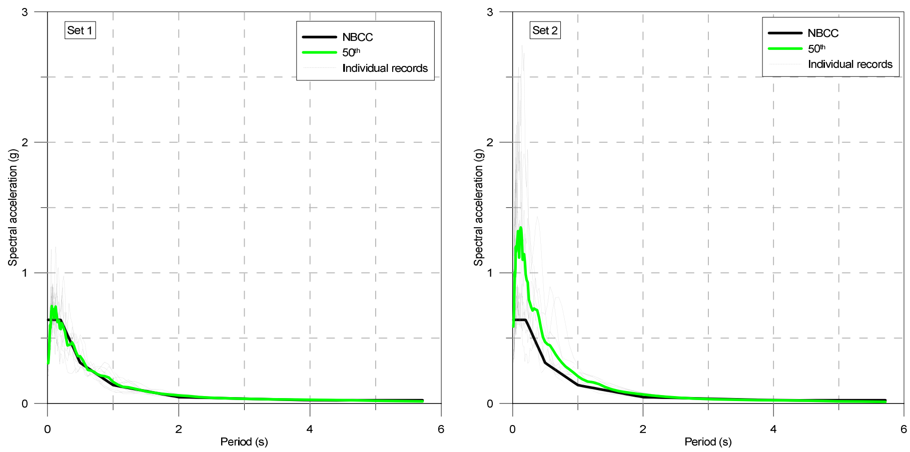


Figure 2. NBCC design acceleration spectrum and median 5% damped elastic acceleration spectra for calibrated ground motions

3.2 Modelling of the soil-foundation system

The nonlinear time history analysis of the soil-foundation-wall system was carried out in the OpenSees program [13]. The model accounts for inelastic wall behaviour and the nonlinear soil response. Nonlinear foundation response was represented using the Beam-on-Nonlinear-Winkler-Foundation concept [2,14]. The foundation is modeled as an elastic beam with a finite number of vertical nonlinear springs. Each spring is represented by a one-dimensional zero-Length element, and nonlinearity is modeled using the qzsimple1 material [15] that permits to represent uplift (geometric nonlinearity) and permanent settlement (material non-linearity). The backbone curve is unsymmetrical and is defined by an ultimate compression load and the reduced tensile strength when the foundation uplift occurs. To achieve this behaviour, three elements are connected in series: an elastic spring connected in parallel with a viscous dashpot, a rigid-plastic spring, and a gap element consisting of a bilinear elastic spring and a nonlinear spring connected in parallel. Several input parameters are required to describe the behaviour of the qzsimple1 material, namely the type of the soil, the ultimate bearing capacity, the tension capacity, the vertical stiffness, the displacement at 50% of ultimate load, suction, and viscous damping.

To characterise the springs below the foundation, it is necessary to define the stiffness, ultimate bearing capacity, and the strain at yielding. In fact, the knowledge of the appropriate distribution of stiffness and bearing below the foundation to individual springs is essential, because of its impact on the global response of the system [3]. Figure 3 illustrates the distribution of the nonlinear springs used in this study. As recommended by [16], the foundation was divided into three zones, two end zones and a middle zone, and the springs were uniformly distributed within each zone. The length of the zones and the stiffness of the springs were calculated following the recommendations by [14]. The springs are more tightly spaced in the end zones compared to the middle zone to reflect more heavily loaded edges of the footing. Different stiffness values were assigned to the nonlinear springs in the middle and end zones to reproduce the non-uniform distribution of soil pressure under vertical loads, as well as to represent the rotational stiffness of the soil. It was assumed in the analysis that sliding cannot occur. The elastic beam used to model the foundation was considered as infinitely stiff in the middle zone to account for the presence of the wall. The elastic beam-column element, that was employed to model the structural foundation, was considered as infinitely stiff in the middle zone to account for the presence of the wall.

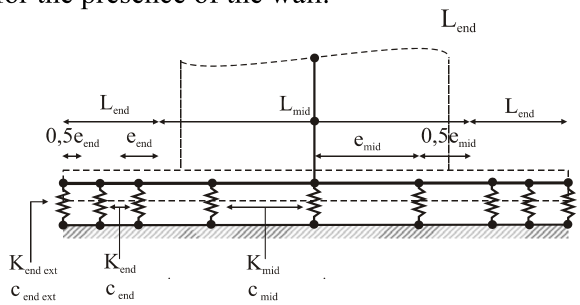


Figure 3. Characteristic parameters for the soil-foundation model

The required number of springs and their spacing was determined from the results of a parametric study. Five configurations were examined including 9,11,13,15 and 17 springs placed in each of the three zones. The response was compared by observing the first three periods of vibration, the force in the spring that is related to the bearing pressure in the soil, the vertical displacements at the center of the wall and at the foundation edge, the wall storey shears and the overturning moments as well as the horizontal displacements at roof level.

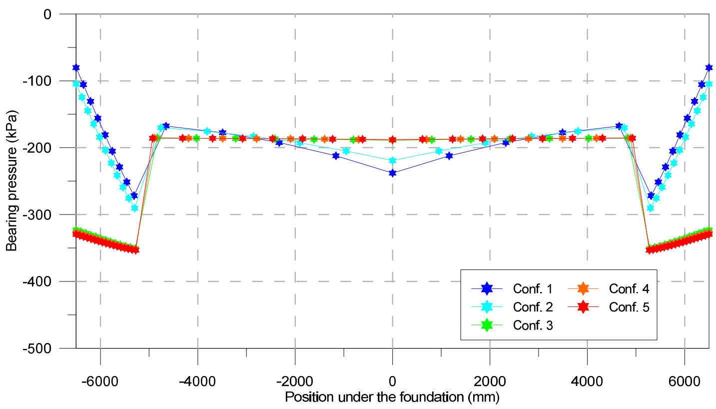


Figure 4. Soil bearing pressure distributions for various spring configurations studied

Figure 4 shows an example of the results for bearing soil pressures for different spring configurations. The anticipated soil pressure distributions were obtained for configurations with at least 13 springs per zone. Overall, for all the parameters studied, the study showed that the number of springs did not influence the response when at least 15 springs were used in each zone. Thus, the configuration including a total of 45 springs was selected for further analyses. The values of the parameters used in the soil-foundation model for soil profile INF are summarized in Table 4.

Force level	$R_oR_d = 2.0$	M_n ($R_oR_d = 4.74$)	$R_oR_d = 5.6$
L_{end} (mm)	1465	1290	1230
e_{end} (mm)	101	89	85
$K_{end\ ext}$ (N/mm)	5.53E+04	4.73E+04	4.38E+04
K_{end} (N/mm)	1.11E+05	9.57E+04	8.76E+04
$C_{end\ ext}$ (N-mm/s)	3.04E+02	2.50E+02	1.24E+02
C_{end} (N-mm/s)	6.08E+02	2.99E+02	2.48E+02
L_{mid} (mm)	15870	11670	11040
e_{mid} (mm)	1058	778	736
K_{mid} (N/mm)	6.98E+05	4.53E+05	4.15E+05
C_{mid} (N-mm/s)	6.37E+03	2.61E+03	2.15E+03

Table 4. Values of characteristic parameters for the soil-foundation model (soil profile INF)

The study carried out by Le Bec [8] for a 10-storey reinforced concrete wall with rocking foundations showed that radiation damping had little impact on the system behaviour. Therefore, the component of radiation damping associated to foundation rotation was represented in the model, while the component associated with the vertical movement was neglected. The same approach was adopted in the present study. Le Bec also indicated that the higher suction capacity of the soil generally reduced the effects of rocking response. Because the observation of the rocking behaviour was the focus of the present study, the suction capacity of the soil was also omitted in the analysis.

3.3 Modelling of the shear wall

The studied OpenSees model of the shear wall is illustrated in Figure 5. The walls were modeled using one force-based nonlinear beam column element per floor. Based on results of the parametric study, the elements were divided into 2 vertical sections. Each section was discretized into fibers for which the nonlinear material stress-strain response was defined. Distinct fibers were defined for confined and unconfined concrete zones and for the steel reinforcement.

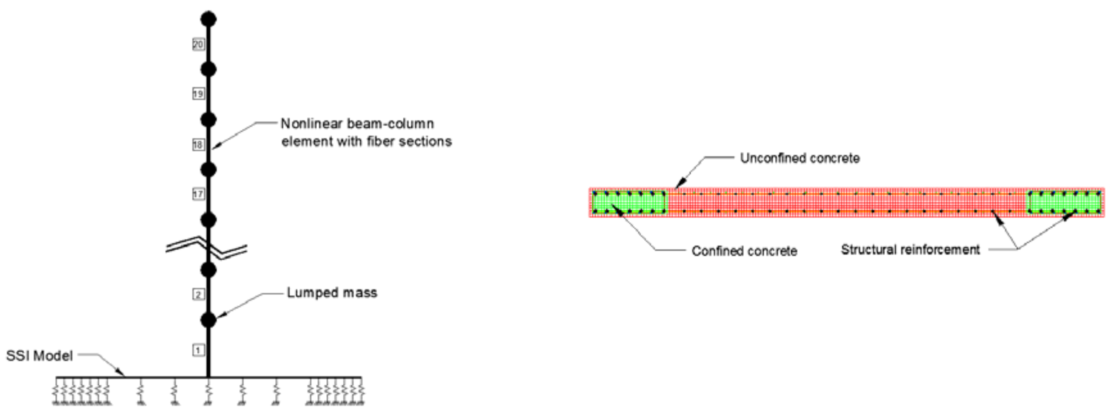


Figure 5. Shear wall model for nonlinear time history analysis with OpenSees

The fiber section model considers the bending moment and axial load interaction, but the shear-bending or shear-axial load interactions cannot be represented. Concrete behaviour was modeled using the uniaxial Kent–Scott–Park model with linear tension softening (Concrete02). The Giuffré-Menegotto-Pinto (Steel02) hysteretic material was employed to describe the inelastic behaviour of the reinforcing bars.

The parametric study was conducted for the fixed-base conditions to determine the adequate number of integration points for the analysis. After examining the structural periods, wall base shear and storey shears, overturning moments and roof displacements, three integration points per element were selected as a rational compromise offering adequate accuracy, convergence and reasonable computational time. This is in line with the practice reported in the literature [17].

3.4 Discussion of results

The response of the soil-foundation-wall system was examined by tracking the wall overturning moments and the shear forces, the horizontal wall displacements, the foundation uplift and the settlement of the soil as well as the maximal force in the nonlinear soil springs, which is indicative of the soil bearing pressure. The comparison is made for median response, but the peak and 84th percentile response values were also recorded. The results obtained for two earthquake sets were comparable. Because more important rocking response was observed for the soil profile INF, the results are presented for this soil profile.

The structure with the foundation designed to develop the nominal capacity of the wall, M_n, (R_oR_d = 4.74, design 2) and the one with the foundation conceived without implementing capacity design principles (R_oR_d = 5.6, design 3) experienced a rocking behaviour. The response of the structure with the foundation designed for the overturning moments corresponding to the incipient rocking limit (R_oR_d = 2, design 1) did not include rocking and was almost identical to that of the fixed-base structure. The rocking movement lengthened the first-mode period by 12% and 15% for design cases 2 and 3, respectively. Previous studies [3] indicate that the increase in vibration period due to soil-structure interaction is influenced by the wall aspect ratio, as well as by the relative stiffness between the structure and the soil, and that, in general, for taller structures the lengthening is usually smaller.

The occurrence of rocking modified the nature of the seismic response of the building. While for design case 1, dissipation of the energy took place exclusively in the superstructure, for two other cases, some yielding initiated at the base of the wall and was followed by rocking. The imposed curvature ductility demand at the base of the wall was much smaller than design prediction with the median value of 2. The maximum value recorded for this design case was about 5 which is still bellow the values that correspond to global ductility of 3.5 assumed in the wall design. The rocking response was the most significant for the structure with the smallest foundations (design case 3). This wall responded elastically, and the seismic energy was converted into kinetic energy associated with the rocking motion. Note that, contrary to design predictions, the rocking behaviour was observed before the overturning moment attained the nominal capacity of the wall. Although the design procedure aimed at achieving energy dissipation in the wall, the analysis showed that the structural system and the soil-foundation provided mechanisms for energy dissipation.

Foundation rocking limited the shear forces and the overturning moments that developed at the base of the wall. In turn, roof displacements increased but by a small margin. In Table 5, median results produced by the Set 1 ground motion records for three designs on soil profile INF are compared with the fixed-base case. Rocking had more impact on decreasing the overturning moments compared to base shears. A similar trend was observed in the study of the 10-storey building [8].

	Shear force (kN)		Overturning moment (kNm)		Roof displacements (mm)	
	V_{50th}	% decrease	M_{50th}	% decrease	Δ_{50th}	% increase
Fixed-base	7790		83 767		209	
$R_oR_d = 2.0$	7454	4.3	81 758	2.4	213	1.9
$R_oR_d = 4.7$	7105	8.8	71 243	14.9	222	6.2
$R_oR_d = 5.6$	6823	12.4	65 754	21.5	224	7.2

Table 5. Impact of foundation size on seismic response parameters of the shear wall: (a) seismic base shear, (b) overturning moments at the base and (c) roof displacements

For design cases 2 and 3, the maximum normalized force in the edge spring exceeded $0.3q_{ult}$, indicating an inelastic soil response. The median soil bearing pressures were larger for the structure with smaller foundation, reaching $0.59q_{ult.}$ Note that for one ground motion record a peak bearing pressure of $0.78q_{ult}$ was recorded. For design case 2, with a larger foundation footprint, the mean soil bearing pressure under the edge of the foundation was approximately 30 percent lower. However, as seen in Figure 6 (a), these values are restricted to the springs at foundation edges and they quickly decrease over the end foundation zone. The impact of localised increase of bearing pressure in the soil and its impact on the overall system behaviour should be further examined in experimental and analytical studies.

(a) (b)

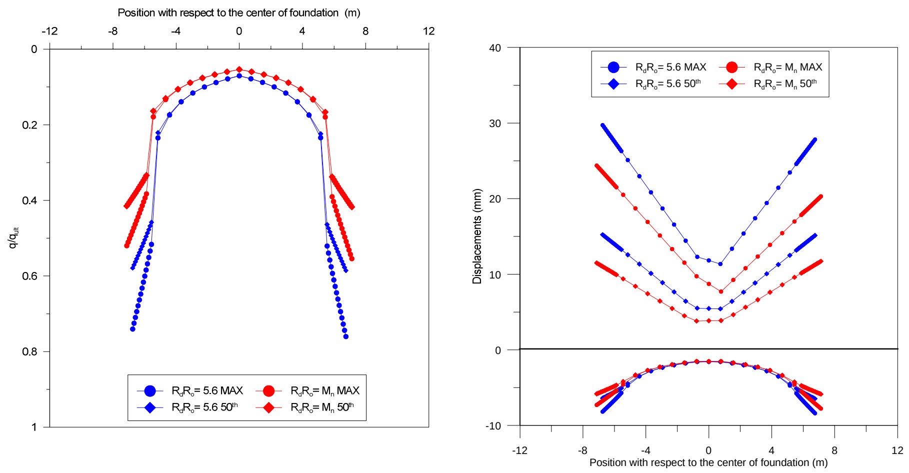


Figure 6. (a) Maximum normalized forces in springs and (b) peak foundation uplift and soil settlement (soil profile INF)

Figure 6 (b) shows the distribution of median and maximum peak foundation uplifts and settlements recorded along the length of foundation for design cases 2 and 3. Displacements were measured with respect to the initial position of the foundation under gravity loads and include elastic and inelastic components. As anticipated, higher values were recorded for the smaller foundation. Consistent with the location of the largest forces in the springs, the most important displacements were observed at the foundation edges. Inspection of the corresponding level of the force in the soil springs revealed that most of the peak displacement resulted in permanent soil deformations. However these deformations are relatively small and should not be detrimental the overall response of the system.

One of the objectives of this study was to examine if rocking can successfully mitigate higher-mode response. In Eastern Canada, shear walls are particularly prone to exhibit response influenced by higher-mode contributions because typical ground motion excitations are particularly rich in high frequencies. Many researchers reported inadequate seismic design strength envelopes for shear walls [18,19, 20]. A more recent experimental study [21] conducted specifically for Eastern Canadian seismic conditions, showed increased flexural and shear demand in the upper storeys of the shear walls leading to the creation of second flexural plastic hinge and potential fragile shear failure.

The inspection of the time history of storey shears, bending moments, displacement profiles and rotational curvature demand confirmed that the seismic response of the studied structure is influenced by the higher modes. However, no significant difference in the distribution of response parameters over the height of the wall was observed regardless whether rocking occurred or not. For the wall under study, rocking did not result in the reduction of bending moments or shears in the upper part of the structure although it had a beneficial effect on the forces at the base of the wall. The occurrence of rocking seems to produce similar effects on the rest of the structure as the formation of the plastic hinge at the base of the structure. This similarity could be anticipated because both phenomena rely on the formation of a mechanism at the base of the wall to limit seismic response.

4 CONCLUSIONS

- Traditional seismic design procedures aim to achieve inelastic response in the superstructure and avoid any damage to the foundations. In the Canadian provisions, rocking of the shear walls on their foundations is not intended to be a principle source of seismic energy dissipation. In this study, the behavior predicted by the design procedures could not be validated using the analysis which included the soil-structure interaction effects. In code compliant design both the yielding of the superstructure and inelastic soil response occurred and contributed to dissipate seismic energy induced in the building. Permanent soil deformations were limited and peak median values of soil bearing pressure remained within the design predictions.
- The foundation footprint was significantly reduced when capacity approach was not integrated into foundation design. The response of the superstructure was elastic and more prominent rocking was observed compared to the code compliant design. Permanent soil deformations did not increase significantly, but soil bearing pressures under the edge of foundation reached high values and exceeded the factored resistance assumed in design. However these high values were localized under the edge of foundation and rapidly decreased within the end foundation zone.
- Compared to the fixed-base structure, rocking reduced seismic force demand. The reduction was more appreciable for overturning moments compared to the base shears. Rocking response increased wall roof displacements, but only by a small margin.
- Rocking response was not efficient in mitigation the impact of higher mode on seismic response of the shear wall studied.
- The study was carried out for firm soil conditions and a specific building configuration. Response of soil-foundation-structure system founded on a softer soil should be investigated as well as different building configurations before more the general conclusions can be drawn.

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REFERENCES

[1] G. Gazetas, Seismic design of foundations and soil-structure interaction. First European Conference on Earthquake Engineering and Seismology (ECEES), Geneva, Switzerland, September 3-8, 2006.

[2] Tarra, T. Belytschko, I. Babuska, T.J.R. Hughes, Research directions in computational mechanics. *Computer Methods in Applied Mechanics and Engineering*, **192**, 913-922, 2003.

[3] B.L. Kutter, G. Martin, T.C. Hutchinson, C. Harden, S. Gajan, J. Phalen, *Workshop on modelling of nonlinear cyclic load-deformation behavior of shallow foundations. Report no. PEER-2005/14*, Pacific Earthquake Engineering Research Center (PEER), 2006.

[4] R. Paolucci, M. Shirato, M.T. Yilmaz, Seismic behavior of shallow foundations: Shaking table experiments *vs* numerical modeling. *Earthquake Engineering and Structural Dynamics*, **37:4**, 557-595, 2008.

[5] CSA, *Design of concrete structures, Standard CAN/CSA-A23.3-04.* Canadian Standard Association, Rexdale, ON, 2004.

[6] D.L. Anderson, Effect of foundation rocking on the seismic response of shear walls. *Canadian Journal of Civil Engineering.* **30:2**, 360-365, 2003.

[7] NRCC, *National building code of Canada 2010*. National Research Council of Canada, Ottawa, ON, 2010.

[8] A. Le Bec, *Effects of rocking of shallow foundation on seismic response of reinforced concrete shear walls*. M.Sc. Thesis. CGM Dept., Ecole Polytechnique de Montréal, Montréal, QC. *(In French),* 2009.

[9] E. Bentz, P. Collins, *Response2000: reinforced concrete sectional analysis using the modified compression theory* Version 1.0.5: www.ecf.utoronto.ca/~bentz/r2k.htm, 2000.

[10] V. K. Prest, J. Hode-Keyser, Geological and engineering characteristics of surficial deposits, Montreal island and vicinity, Quebec. Geological survey of Canada, Paper 75-27, 577-595, 1977.

[11] CGS, *Canadian foundation engineering manual (4th ed).* Canadian Geotechnical Society, Richmond, B.C, 2006.

[12] G.M.Atkinson, Earthquake time histories compatible with 2005 National building code of Canada uniform hazard spectrum. *Canadian Journal of Civil Engineering.* **36:6**, 991-1000, 2009.

[13] F. McKenna, G.L. Fenves, M. H. Scott, *Open system for earthquake engineering simulation: OpenSees.* Pacific Earthquake Engineering Research Center (PEER), University of California, Berkeley, 2008.

[14] C. Harden, T.C. Hutchinson, G. R.Martin, B. L Kutter, B. L, *Numerical modeling of the nonlinear cyclic response of shallow foundations. Report no. PEER-2005/04*, Pacific Earthquake Engineering Research Center (PEER), 2005.

[15] R. W. Boulanger, The QzSimple1 material. *Document for the OpenSees Platform,* 2003.

[16] ASCE (2000). *Prestandard and commentary for the seismic rehabilitation of buildings (FEMA 356)*: Federal Emergency Management Agency, Washington, D.C.

[17] Y. Boivin, P. Paultre, Seismic force demand on ductile reinforced concrete subjected to Western North American ground motions: Part 1 – parametric study. *Canadian journal of civil engineering*, **39**, 723-727, 2012.

[18] M.J.N Priestley, Does capacity do the job? Examination of higher mode effects in cantilever walls. Bulletin of New Zeeland Society fo Earthquake Engineering, **36:4**, 276-292, 2003.

[19] A. Rutenberg, E. Nsieri, The seismic shear demand in ductile cantilever wall systems and the EC8 provisions. *Bulletin of Earthquake Engineering* **4**,1–21, 2006.

[20] Y. Boivin, P. Paultre, Seismic force demand on ductile reinforced concrete subjected to Western North American ground motions: Part 1 – mew capacity design methods. *Canadian journal of civil engineering*, **39**, 738-750, 2012.

[21] I. Ghorbanirenani, R. Tremblay, P. Léger, M. Leclerc, Shake table testing of slender RC shear walls subject to Eastern North America seismic gound motions. *ASCE Journal of structural engineering*, **138:12**, 1515-1529, 2012.