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IMPACT OF SOIL-STRUCTURE INTERACTION EFFECTS ON SEISMIC ASSESEMENT IN MODERATE SEISMIC ZONES

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Abstract

Building structures designed in accordance with older versions of building codes and standards are likely to exhibit unsatisfactory behaviour under seismic loads. Existing structures represent an important part of the built heritage and therefore an economic challenge for current governments, as well as a public security issue for users of these buildings. The assessment of seismic response is most often done assuming fixed-base support conditions, thus ignoring the combined response of structure-foundation-soil system. This approach is considered conservative, but it neglects the great potential to reduce seismic demand imposed on the superstructure through controlled foundation yielding, appropriate mobilisation of bearing capacity or the uplifting at the foundation-soil interface. In order to properly assess the seismic behaviour of existing building structures and to make cost-effective decisions regarding their seismic retrofit, it is necessary to put forward reliable evaluation methods that can preferably account for the integrated seismic response of all system elements: the superstructure, the foundation and the underlying soil.

In this study, a three-storey reinforced concrete frame building was designed in accordance with the 1965 National Building Code of Canada and its seismic response was assessed using results of nonlinear time history analysis. The Attention was directed at the impact of soil-structure interaction on the seismic assessment. The selected building geometry represents typical 1970's school buildings in Quebec, Canada. Consistent with design practices of that era, the foundations were designed for gravity loads only and for two different soil conditions: rock (site class B) and stiff soil (site class D). Initial assessment was carried out using an equivalent static force method and response spectrum analysis considering typical fixed-base conditions. Subsequent more refined assessments were done with 2D nonlinear time history analyses of the superstructure with the soil-structure interface, and were conducted in OpenSees for a set of simulated ground motions compatible with the design spectrum. Soil-structure interaction effects were modelled using the beam-on-nonlinear-Winkler-foundation concept. Base shears, roof displacements and displacement profiles are determined and compared to those obtained assuming a fixed-base condition. The foundation displacements and stresses in the soil are examined to assess the consequences on the global structural behaviour. The response of the superstructure is evaluated by tracking the elastic demand in beams and columns.

Keywords: seismic assessment; soil-structure interaction; nonlinear analysis; existing RC moment resisting frames

1. Introduction

In Canada, seismic design provisions were included for the first time in the National Building Code in 1941 whereas special provisions for seismic design and detailing for concrete moment-resisting frames and shear walls were introduced in the 1974 edition of the concrete design standard CAN/CSA-A23.3 [1]. The lower seismic design forces and the absence of ductile detailing in RC structures built prior to 1974 make them susceptible to potential damage and inadequate behaviour under earthquake loading. The majority of schools in Quebec (Canada) were built in the 1970s and before, and are likely to have inadequate strength and ductility due to the absence of adequate seismic design criteria at the time. Although the province of Quebec is located in a seismic zone with moderate seismic hazard, the aging infrastructure conceived prior to the introduction of modern seismic design provisions, increases significantly the seismic risk in large cities such as Montreal or Quebec City. In order to properly assess the seismic behaviour of these structures and to make cost-effective decisions regarding their seismic retrofit, it is necessary to put forward reliable evaluation methods that can preferably account for the integrated seismic response of all system elements: the superstructure, the foundation and the underlying soil.

For simplicity, numerical models of lateral load resisting systems in buildings, used for seismic assessment, are commonly assumed to be fixed at the base. The study of behavior is thus limited to the superstructure, and the impact of the foundation and the soil on the structural response, which can be significant, is not accounted for. Site conditions can amplify seismic input and impose significant seismic demand on the superstructure. At the same time, during large earthquake events, the nonlinear behavior of foundation and soil can provide an excellent mechanism to dissipate seismic energy, and thereby diminish seismic demand on the superstructure [2]. These effects are particularly notable for stiffer superstructures. Representing a global response of soil-foundation-structure system in seismic assessment procedures is necessary to estimate more accurately the structural response and possibly avoid the unnecessary seismic retrofits of older buildings.

In this study, a three-storey reinforced concrete frame building was designed in accordance with the 1965 National building code of Canada [3] and its seismic response was assessed using the results of a nonlinear time history analysis. Attention was directed at the impact of the soil-structure interaction on the seismic assessment. The selected building geometry represents typical 1970's school buildings in Quebec, Canada. Consistent with design practices of that era, the foundations were designed for gravity loads only and for two different soil conditions: rock (site class B) and stiff soil (site class D). The initial assessment was carried out using an equivalent static force method and response spectrum analysis considering typical fixed-base conditions. Subsequent more refined assessments were done with 2D nonlinear time history analyses of the superstructure with the soil-structure interface and were conducted in OpenSees for a set of simulated ground motions compatible with the design spectrum. Soil-structure interaction effects were modelled using the beam-on-nonlinear-Winkler-foundation concept. Base shears, roof displacements and displacement profiles were determined and compared to those obtained assuming a fixed-base condition. The foundation displacements and stresses in the soil are examined to assess the consequences on the global structural behaviour. The response of the superstructure is evaluated by tracking the elastic demand in beams and columns.

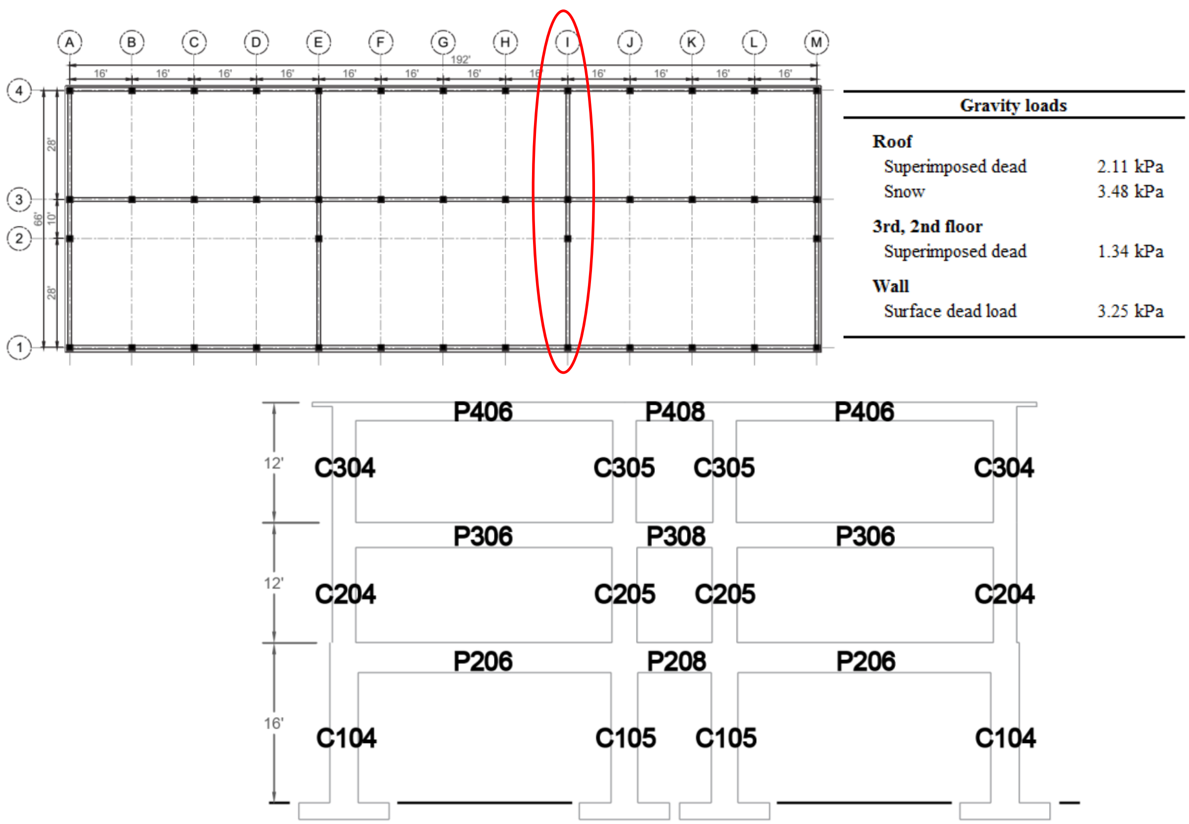
2. Design of the building studied

In order to determine the typical period of construction of schools in the Canadian province of Quebec and the structural system that was most commonly employed, a database of Quebec schools compiled by Nollet and Moretti [4] was initially consulted. It was found that the 63% of the schools were built in the period between 1960 and 1980 and that reinforced concrete (RC) moment resisting frames were used to carry lateral loads in 89% of these schools. The RC building under study is fictitious and represents a typical 1970's school building construction. It is located in Quebec City. The plan view and the typical elevation are illustrated in Fig. 1. Lateral loads are resisted by three RC frames in the long direction and four frames in the short direction. This paper focuses on the seismic assessment of one of the three bay interior frames in the short direction. The design of the building was performed in accordance with the provisions of NBCC 1965 which implements the working stress design method [1]. Gravity loads, given in Fig. 1, are based on data obtained from the plans of existing buildings available in the database. An equivalent static force procedure was applied to determine earthquake induced forces, as permitted for regular structures in NBCC 1965. The design seismic base shear was determined from the Eq. (1):

$$V = R\,C\,I\,F\,S\,W \tag{1}$$

where R is the seismic regionalization factor, C is the coefficient calculated based on the type of construction, I is the importance factor, F is the foundation factor, S is the structural flexibility factor, and W is the total weight. In this study R = 4, C = 0.75, I = 1.3, S = 0.021, W = 28 770 kN, resulting in V equal to 2340 kN. Note that NBCC 1965 made reference to ductile moment resisting RC frames to determine the coefficient C and this was considered in the base shear calculations. However, design at the time did not explicitly consider any ductility provisions or detailing so the ductile response is not anticipated. The NBCC 1965 specifies only two categories of soil: very compressible soil for which the factor F = 1.5 is used, and other soils for which F = 1. Because the impact of very compressible soil was not the subject of this study, F = 1 was used.

The initial design was carried out for gravity and seismic load combinations and later verified for wind loads. For all elements, seismic loads controlled the design. Several structural plans from Quebec school database were reviewed to understand better the design and construction practices of the era studied. Typical beam and column section types are identified in Fig. 1. The calculated fundamental period of the frame is 0.77 seconds.



Gravity loads	
Roof	
Superimposed dead	2.11 kPa
Snow	3.48 kPa
3rd, 2nd floor	
Superimposed dead	1.34 kPa
Wall	
Surface dead load	3.25 kPa

Fig. 1 –Typical floor plan, elevation and design gravity loads

In this study, particular attention was given to the foundation design. Initially, the foundations were sized considering seismic and gravity load combinations. However, a careful inspection of available plans from existing structures showed that for the majority of buildings, the foundations were much smaller than those required for seismic loads and corresponded to the demand imposed by gravity loads only. Thus, two foundation design options were considered in the design and analysis. The results presented herein are those for the foundations sized for the gravity loads only for which more significant impact of SSI was observed as anticipated.

NBCC 2010 [5] defines a total of six site classes varying from A (hard rock) to F (other soil). Seismic site classification is done on basis of the average shear wave velocity in the first 30 meters of soil (Vs_{30}), the average standard penetration resistance (N_{60}) and the undrained shear strength (s_u). To investigate the influence of different soil conditions on the seismic response, two site classes were examined: rock (class B) and stiff soil (class D). Since the building is fictitious, a reverse design methodology was adopted to set soil properties that corresponded to the NBCC site classes considered. Soil maximal admissible stresses (q_{adm}) were first estimated considering a qualitative description of the soil, and further validated following the procedure given in Peck et al. which associates the standard penetration resistance and q_{adm}. Characteristic parameters for selected soil types are summarised in Table 1.

Table 1 – Soil parameters for selected site classes

Site class	Footing depth (m)	Water level (m)	Average density (kN/m^3)	Soil profile	N_{60}	ϕ (deg)	q_{adm} (kPa)
B	2	2	21	Clay shale	-	-	500-1000
D	2	2	20	Medium sand	20	33	100-300

3. Nonlinear time history analysis

3.1. Selection and scaling of ground motions

Contrary to Western Canada, where the seismic activity is directly related to the tectonic plates' interaction, Eastern Canada is a part of the stable North American plate. However, large intraplate earthquakes have occurred in this region and inevitably will happen in the future. Historic data of ground motion records for this region are sparse. In addition, Eastern Canada ground motions are characterised by important high-frequency content that is not encountered in west North America. It is thus challenging to compile the appropriate database of historical ground motion records for time history analyses. As an alternative, Atkinson [6] provides large sets of simulated earthquake records compatible with NBCC design spectra for the range of magnitude-distance (M-R) scenarios that contribute to the seismic hazard at selected location. In this study, a total of 22 ground motion records, 11 for each site class, were selected considering the dominant magnitude-distance scenarios for Quebec City [7]. For each site, five selected ground motion records represent high frequency demand (M=6, R=25 km) and six ground motion records are selected to represent more distant but more damaging earthquakes originating from the Charlevoix zone (M=7, R=100 km). The records were scaled using the procedure described in Atkinson [6]. The ratios of the NBCC 2010 target spectral acceleration SA_{targ} over response spectral acceleration SA_{sim}, are first determined at every period within the selected period range, and the mean and standard deviation are calculated. Records with the lowest standard deviation are then retained and scaled with mean (SA_{targ}/SA_{sim}). Following the recommendations from Atkinson et al. [8] for the high-frequency records, the matching was done in the period range from 0.2 s to 0.8 s, while the low-frequency records were calibrated between 0.5 s and two times the fundamental period of the frame for fixed-base conditions (1.54 s). It was also verified that the average response spectrum of each time history set did not fall more than 10% below the target spectrum. An additional torsional scale factor of 10% was applied to each ground motion to account for the effects of accidental torsions which were considered in the design.

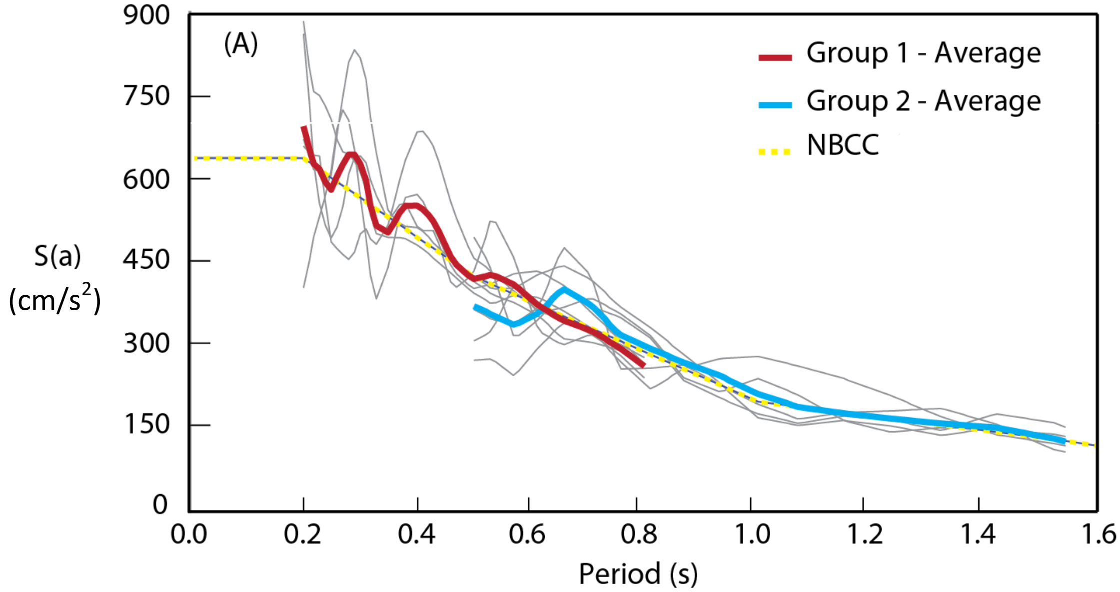


Fig. 2 – Comparison between the median spectra of the simulated ground motions for low-frequency and high-frequency record sets and the target NBCC (CODE) spectrum for class D site.

3.2. Modeling of the frame

In the analysis, three representations of structural behaviour were considered: (i) elastic superstructure with fixed base; (ii) elastic superstructure and nonlinear soil-foundation system, and (iii) elastic columns, inelastic beams and nonlinear soil-foundation system. This paper presents the results obtained for cases (i) and (ii). The 2D finite element models were built in the OpenSees structural analysis platform [9]. To include P-Δ effects in the analysis, fictitious gravity columns carrying the total gravity loads minus the tributary gravity loads supported directly by the frame were added. 5% Rayleigh damping was specified in the first two modes as recommended in ASCE 41-13 [10]. This amount of damping is also consistent with the one used for the design elastic spectra in the NBCC 2010. The building mass was equally divided between the four frames in the direction of the analysis and lumped at storey levels.

For cases (i) and (ii) beams and columns were modelled as elastic beam-column elements. The effects of cracking on flexural stiffness were accounted for by reducing the gross moment of inertia by 60 % and 30% for beams and columns, respectively. The impact of beam-to-column joints' flexibility was not considered in this study and elastic behaviour in shear was assumed for all elements.

3.3. Modeling of the soil-foundation system

The modelling of soil-foundation system was implemented using a flexible boundary substructure approach [11]. This model enables the represention of rocking, sliding and permanent settlement of the foundation. The kinematic effects were neglected as discussed in Kramer and Stewart [12]. Nonlinear soil-foundation response was represented using the Beam-on-Nonlinear-Winkler-Foundation concept [13, 14]. The foundation is modelled as an elastic beam with a finite number of vertical (q-z type) and horizontal (p-x and t-x) nonlinear springs. Each spring is represented by one-dimensional zero-length element, and their nonlinear inelastic behaviour is modeled using modified versions of QzSimple1, PySimple1, TzSimple1materials implemented in OpenSees by Boulanger, et al. [15]. Nonlinear springs were non-uniformly distributed to simulate the rocking behaviour [16]. The use of the variable spring stiffness permitted to represent the higher reactions that can develop in the end-zones under the vertical loads. The width of the end-zone and the coefficient to increase the spring rigidity are defined by expressions derived by Harden and Hutchinson [17].

Several input parameters are required to describe the behaviour of the QzSimple1 material, among others: bearing capacity (Q_{ult}, T_{ult}, P_{ult}), initial elastic stiffness (k_{in}), distribution and magnitude of vertical stiffness, tension capacity (TP) and radiation damping of the elastic section of the Winkler (C_{rad}). For the soft soil site, the ultimate bearing capacity in this study was calculated using the recommendation of the CFEM [18] from the equations based on Meyerhof [19], while for the rock soil site an elastic response in compression was anticipated, and the ultimate strength was not specified. Cohesion was ignored. The ultimate lateral load capacity is defined as the passive earth pressure per length of the footing and is calculated according to Raychowdhury [20]. The ultimate sliding resistance is based on Coulomb's analogy with a sliding block load which considers cohesion and internal friction angle as the main contributors. Tension capacity was neglected to permit rocking of the footings. Elastic stiffness and radiation damping were calculated as specified by Gazetas [21].

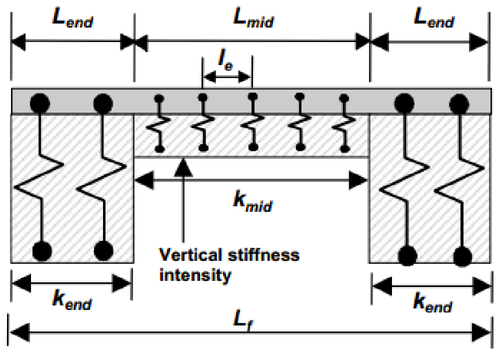


Fig. 3 – BNWF mesh discretization with variable stiffness ratio and end-length ratio.

The distribution of Winkler springs is shown in Fig. 3. A minimum number of 25 springs along the footing length is suggested by Gajan, et al. [22], and therefore a spring spacing ratio (I_e/L) of 4% was selected. Following the NEHRP [23] recommendation, a footing end-length ratio (L_{end}/L) of 20% was taken and a stiffness augmentation ratio (K_{end}/K_{mind}) of 4.0 and 3.7 was calculated for site classes D and B, respectively.

4. Seismic assessment

Assessment of the building seismic response was done using the results of different analyses including the equivalent static force method (ESF), the response spectrum analysis (RSA), and the time history analysis (TH), and for the multiple boundary conditions at the base. Initially, the elastic demand obtained from different analyses was compared with the fixed-base boundary conditions. Seismic demands were then compared for two types of soil in order to evaluate the impact of soil-structure interaction.

Earthquake loads were calculated according to the NBCC 2010 provisions. The seismic hazard was characterized by uniform hazard spectral ordinates, S_a, that are determined at periods of 0.2s, 0.5s, 1.0s, and 2.0 s and for a probability of exceedance of 2% in 50 years. These values are modified by the foundation factors F_a and F_v to obtain the design spectrum *S* used to determine the seismic design base shear given in Eq. (2):

$$V_{ESF} = S(T_a)M_vI_EW/R_oR_d \tag{2}$$

In this equation, T_a is the fundamental period of the fixed-base structure, M_v is the factor that accounts for the increase in base shear due to higher mode effects, I_E is the importance factor, W is the seismic weight, and R_d and R_o are the ductility- and overstrength-related force modification factors, respectively. The existing frames were considered as moment-resisting frames of conventional construction as defined by NBCC2010 because no particular ductile detailing was implemented, and were assigned R_d = 1.5 and R_o = 1.3. The results of response spectrum analysis were calibrated to achieve minimum base shear of $0.8V_{ESF}$ as required by NBCC for regular structures.

4.1. Assessment for fixed-based conditions

Seismic base shear calculated by NBCC 2010 ESF procedure for site class B is equal to 859 kN and exceeds the NBCC 1965 design base shear (644 kN) by 33 percent. For site class D, the difference is more pronounced; NBCC 2010 base shear (1783 kN) is more than 2.5 times larger compared to the one used in the design. Even when a reduced seismic load factor of 0.6 is considered, as suggested by the commentary L of NBCC 2010 as a suitable criterion to trigger seismic retrofit, the imposed seismic demand at the base is still excessive. Median base shears obtained from time history analysis show good agreement with the RSA results, while the maximum values match the base shears obtained by the ESF method.

Elastic demand on beams and columns, determined for the full NBCC 2010 seismic load, is shown in Fig. 4 in blue and red for site B and D, respectively. The elastic bending moments in the beams shown in Fig 4 (a) are normalised by the factored beam flexural resistance calculated according to the provisions of the current Canadian concrete standard A23.3-04-R2010 [24]. For site class B, exterior beams show adequate response, while the imposed seismic demand exceeds the capacity of interior beams at two bottom storeys. For site class D most of the beams do not have the adequate resistance for the imposed seismic demand, regardless the type of analysis used.

Columns are examined for axial force-moment interaction. For site class B, all columns exhibit satisfactory response. Seismic loads induce more significant demand in the interior columns. The capacity of exterior columns is adequate for all but the first storey columns. For site class D, none of the columns has adequate response and the demand, particularly for interior columns, significantly exceeds capacity.

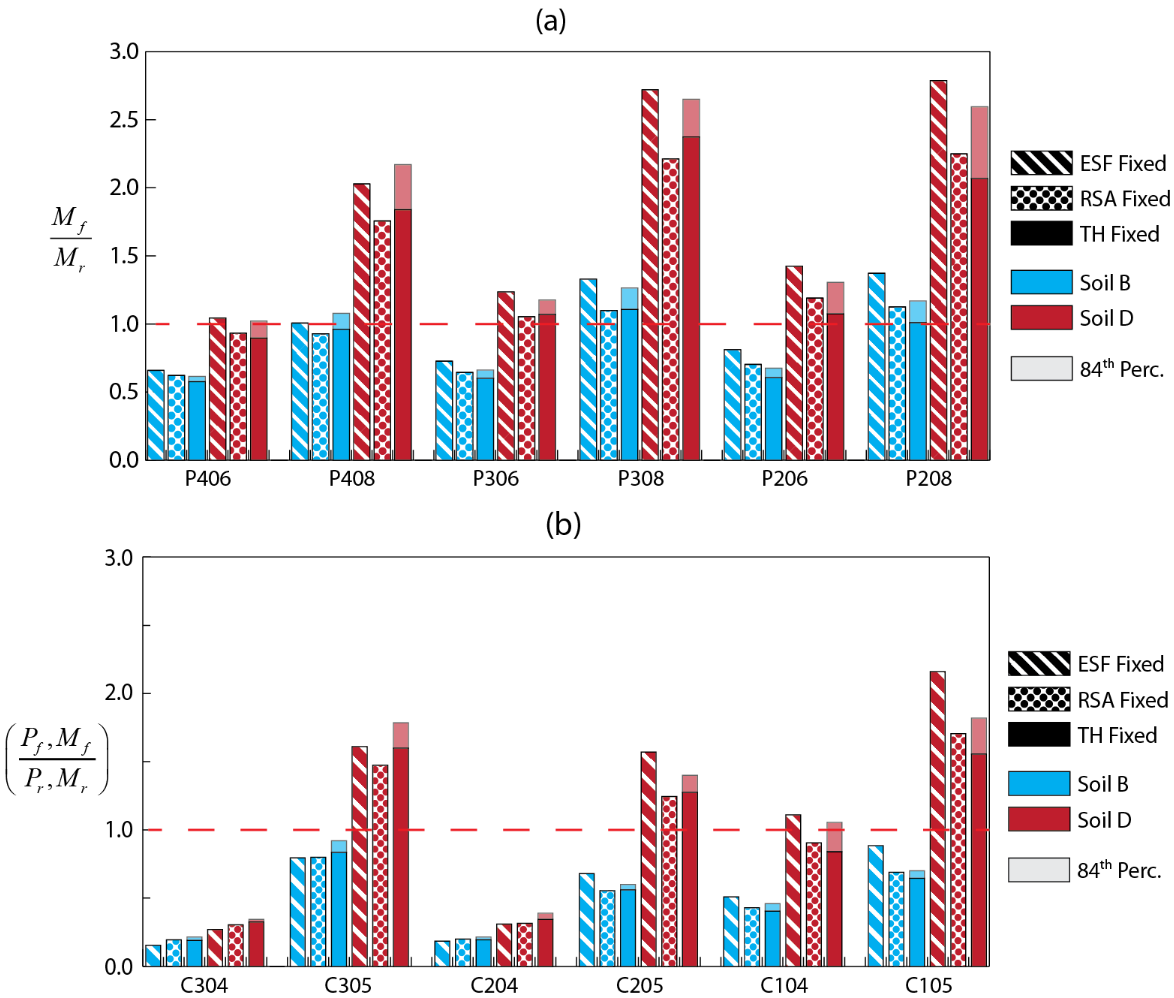


Fig. 4 – Fixed boundary conditions: Comparison of (a) beam end-moment demand-to-capacity ratio and (b) axial force-moment demand-to-capacity ratio for columns.

4.2. Assessment considering soil-structure interaction

As anticipated, the inclusion of SSI effects in the model resulted in the lengthening of the building period. The period increase was much more pronounced for the softer soil (33% and 2% for site classes D and B, respectively). Consequently, a small decrease of base shear was recorded for class B site while for the site class D the median value of base shear (824 kN) was 40% larger compared to the one obtained by the RSA method. Note that for class D site, a larger spread of results was observed; the 84th percentile value reached 1145 kN and a maximum value recorded was 1262 kN.

Figs. 5(a) and (b) present demand-to-capacity ratio for beams and columns respectively. In view of the similar results obtained for different analyses for fixed boundary conditions, member forces are only shown for the TH method and compared for fixed and flexible boundary conditions. As expected, similar ratios are obtained for class B site since the soil rigidity is high and the response of structure-foundation-soil system is close to the fixed boundary response. All exterior beams have sufficient capacity while the shorter, interior beams are overloaded. Even though the difference between the fixed-base and the flexible-base response is relatively small, the inclusion of SSI leads to satisfactory median response. For site class D, an important reduction in force demand for the flexible foundation is observed, lowering the median demand-to-capacity ratio to one for all but one beam. It also appears that the foundation rocking and settlement helps equilibrate the moment demand on beams P206 and P208 in comparison with the fixed boundary condition.

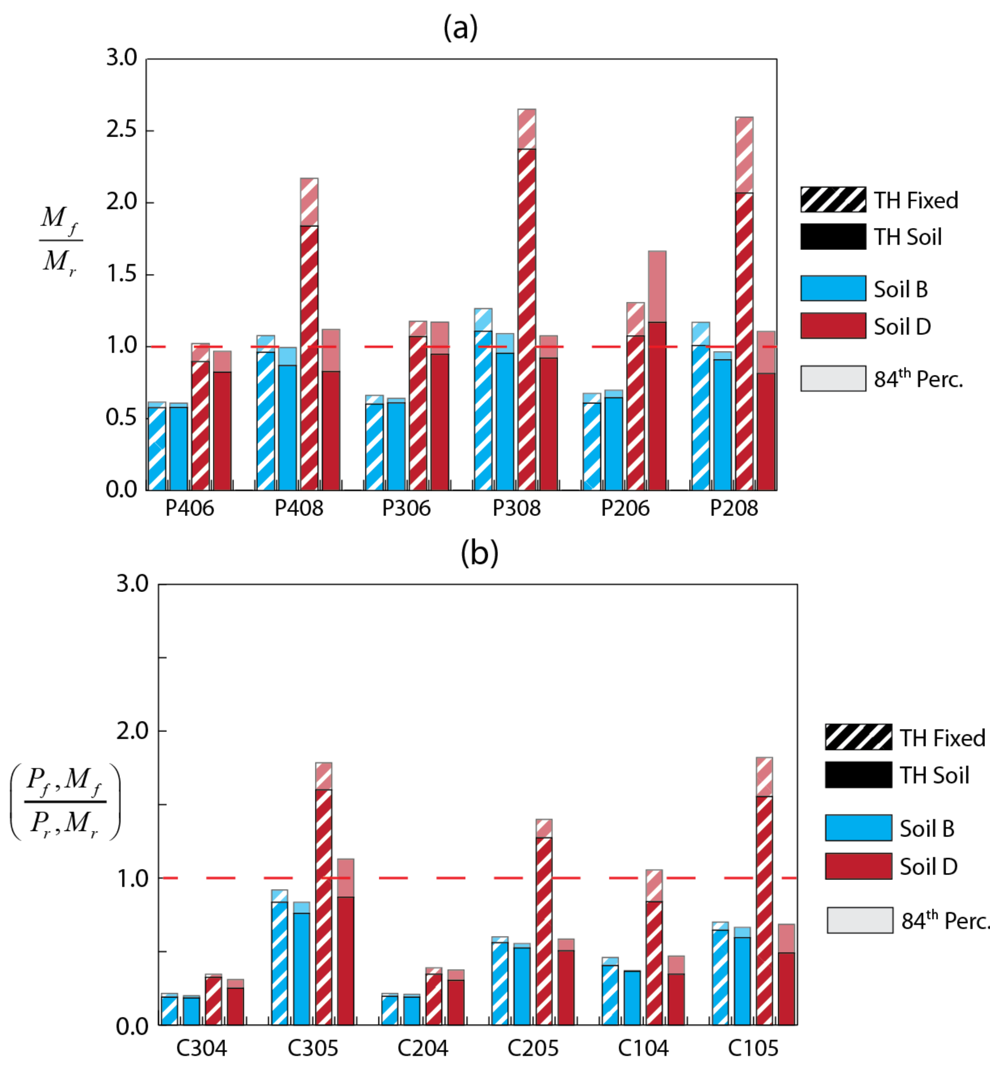


Fig. 5 – Comparison of (a) beam end-moment demand to capacity ratio and (b) axial-force-moment demand to capacity ratio for fixed and flexible boundary conditions.

Since tension capacity of the soil was set to zero for the two soil conditions, the footing was able to freely rock and lift, thereby preventing tension to develop in the columns. Fig 6 (a) shows that for the fixed base conditions, moment-axial force interaction resulted in large tensile stress in the interior column C105. When SSI effects are included in the analysis, no tension develops and the demand is significantly reduced.

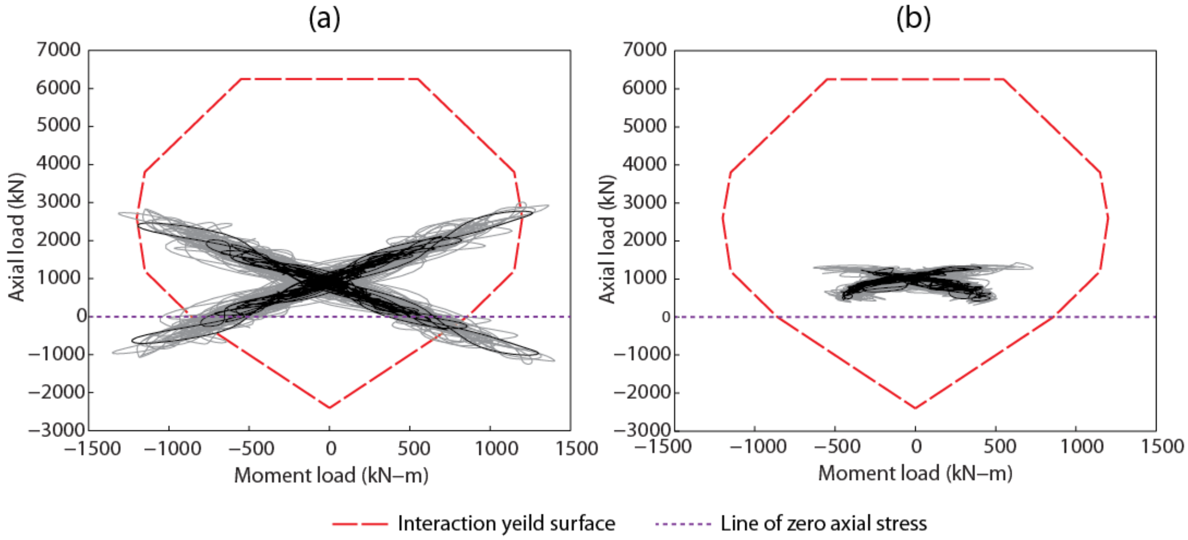


Fig. 6 – Site class D: Axial-force-moment demand of column C105 for (a) fixed boundary conditions and (b) flexible boundary conditions

The response of the soil-foundation system was observed by tracking the foundation uplift and the settlement of the soil as well as the maximal force in the nonlinear soil springs, which can be related to the soil bearing pressure. The comparison is made for the median response.

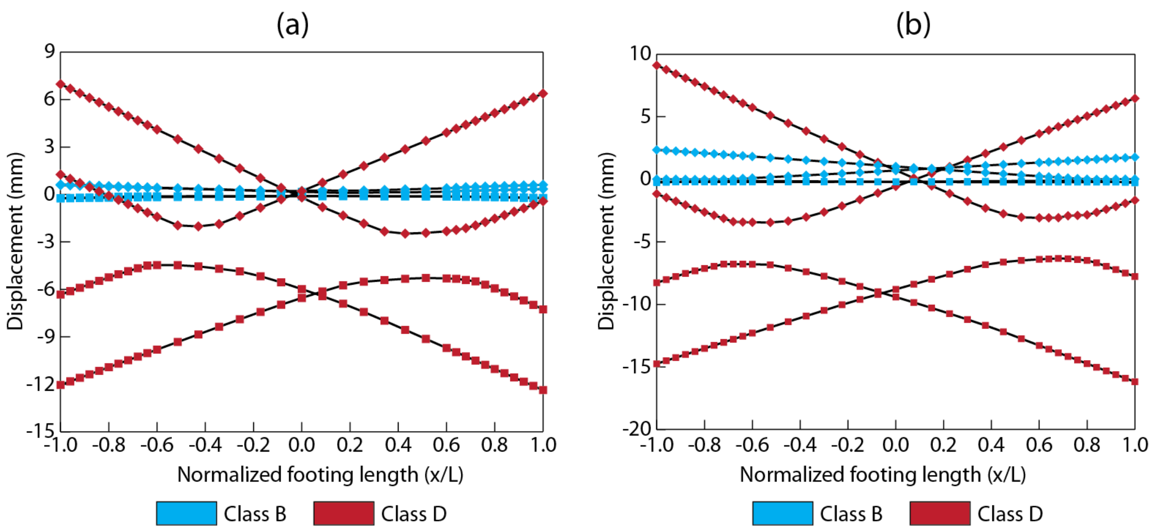


Fig. 7 – Median peak foundation uplift and soil settlement (a) exterior footings and (b) interior footings

Fig. 7 shows the distribution of median peak foundation uplifts and settlements recorded along the length of foundation for the exterior and the interior footings, respectively. Displacements were measured with respect to the initial position of the foundation under gravity loads and include elastic and inelastic components. Displacements recorded for site class B are negligible and show that for such sites, the fixed-based assumption is justifiable. As anticipated, much higher values were observed for the softer soil (site class D). The largest displacements occurred at the foundation edges. Soil underwent permanent deformations; however they were relatively small and should not be detrimental to the overall response of the system.

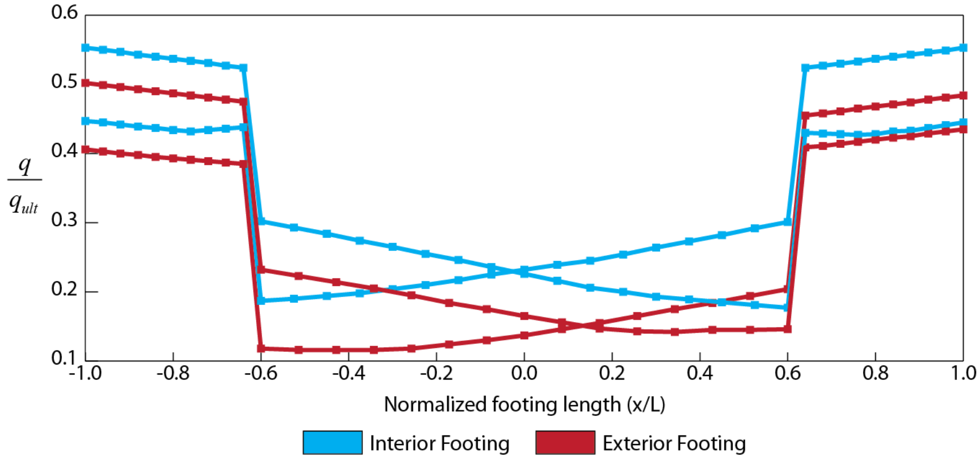


Fig. 8 – Median maximum normalized forces in springs for interior and exterior footings (site class D)

In Fig.8, median value of maximum forces in the springs for four frame footings were normalised by the ultimate soil bearing pressure q_{ult} and compared. The results in blue are for the left and right interior footings and those in red for the left and right exterior footings for class D site. Consistent with the location of maximum displacements, the maximum normalized forces were recorded at the edge of the footing. Results in blue are for left and right interior footings in blue, and left and right exterior footings in red for site class D. A similar distribution of spring forces is obtained for site class B, however the values are low and in the edge springs do not exceed $0.3q_{ult}$, indicating an elastic soil response. For site class D, an inelastic soil response is observed; normalised spring forces reached $0.55q_{ult}$ at the foundation edge for more loaded interior footings. As seen in Fig. 8, these values remain rather uniform over the end foundation zone, but they are not excessive. Further experimental and analytical studies are required to determine the acceptable levels of bearing pressures in the soil that assures the integrity and the safety of the combined structural response.

5. Conclusions

Following conclusions can be drawn from this study:

- As anticipated, the inclusions of SSI lengthened the structural period and reduced the base shear force. The effects were much more pronounced for the soft soil (site class D) and the smaller foundation designed for gravity loads only. For this case, the period increase was about 33% and base shear decreased by about 40%. The SSI augmented inter-storey displacements, but they still remained well below code limits.
- SSI had a more significant impact on the response of the superstructure for softer soils (site class D). The reduction of seismic demand was more prominent at the bottom storeys. The inclusion of SSI in the analysis efficiently reduced seismic demand on the beams and increased the number of beams with the satisfactory response. Another beneficial impact was the redistribution of beam bending moment which also helped improved the seismic response of the beams
- Inclusion of SSI had a significant positive impact on column assessment. Foundation rocking and soil settlement eliminated tension force in the bottom storey columns and reduced force demand by 50%.
- The permanent settlements were observed in the soil for class D site, but they were not excessive. Although higher bearing stresses were recorded in the edge foundation zone, they don't appear to undermine the integrity and the safety of the structural response.

6. Acknowledgements

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