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Seismic demands in gravity load frames of shear wall buildings, including soil-structure interaction

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ABSTRACT

This paper presents the effects of considering soil-structure interaction (SSI) on the calculation of seismic demands in the gravity load resisting system (GLRS). A large number of nonlinear time history analyses (NLTHA), including SSI, have been performed on a typical 12-storey shear wall building for three different linear soil media ranging from stiff to soft. SSI has been modelled using the sub-structure method. A model in which SSI has been modelled with 3D finite elements has also been used to validate the latter for the building. The effect of underground structure cracking is also investigated by performing the analysis on uncracked and severely cracked underground structures. Ground motions for NLTHA have been selected by the conditional spectrum method. NLTHA results have shown that the bending moment effectively increases in gravity columns as the soil becomes softer and that the larger bending moment demand occurs in the corner column at ground level when severely cracked properties for the underground structure are used. Based on these results, code-based linear analyses have been performed with and without SSI. These analyses have shown that it may be unsafe to use a simple, fixed-base model for soil classes D and E when calculating seismic demands in the GLRS. The behaviour is better captured when foundation movements are modelled using the sub-structure method; however, using a well-calibrated single rotational spring under the core walls may be a reasonable assumption. For soil class E, the behaviour of the building is not well-captured for all linear analyses. Hence, for soil class E, nonlinear analysis is required.

Keywords: reinforced concrete, building, seismic design, soil-structure interaction, gravity load frame

INTRODUCTION

The design of elements that are not part of the seismic force resisting system (SFRS) is an important part of the seismic design process, and this importance has been well-recognized in many past major events. While the design of the SRFS is quite straightforward because it is designed to resist all seismic forces, the design of the gravity load resisting system (GLRS) is more challenging. Indeed, elements that are part of the GLRS are not required to take a portion of the total seismic forces. However, these elements must be capable of withstanding the large deformations induced by earthquakes.

The displacement profiles used by engineers to assess elements of the GLRS have long been the simple linear profiles calculated through seismic analysis. However, this practice has been demonstrated to underestimate demands on the GLRS in certain cases; principally, in the first floor slim columns of concrete shear wall buildings [1]. These new developments have led the National Building Code of Canada (NBCC) and the Canadian Standard Association (CSA) to add restricting requirements for the seismic design of GLRSs. The standard CSA A23.3-14 now requires that the deformations used to assess GLRSs include the inelastic displacement profile of the SFRS, as well as the foundation movements [2].

Calculating the inelastic displacement profile of the SFRS usually requires a nonlinear analysis. However, such an analysis is complex to implement and requires advanced modelling knowledge; thus, it is rarely executed by engineers. Accordingly, a simplified method is proposed in the standard CSA A23.3-14. This simplified method is based on deterministic curves of the envelope of the relative interstorey drift ratios depending on the displacement at the top of the GLRS that is being designed. Even though this method works, it is quite cumbersome to implement since it is limited to buildings with simple SRFSs. A building with two types of SFRSs in the same load direction cannot be analyzed using this procedure. Because of this, another simple method has been proposed by Beauchamp *et al.* [3]. The idea behind this simple method is to perform a response spectrum analysis (RSA) on the complete building model in which the stiffness of the GLRS is reduced by dividing by a large factor. Demands in the GLRS are then obtained directly in each element by multiplying by that factor. To account for the inelastic displacement profile, the stiffnesses of the shear walls are reduced by approximately half their values over the plastic hinge region. This method is superior and more general since it is performed directly on the finite element model of the building.

Foundation movements can be considered by explicitly modelling the soil-structure interaction (SSI). However, again, this is rarely done by engineers because of the complexity inherent in this kind of analysis. The consideration of SSI for shear wall concrete buildings generally decreases the forces in the walls. However, it increases displacements and thus the deformation of secondary elements in the GLRS. Thus, this paper presents the effect of considering SSI on the calculation of seismic demands in the GLRS. Many nonlinear analyses that include SSI have been performed on a typical 12-storey shear wall building for three different linear soil media ranging from stiff to soft. The effect of underground structure cracking is also investigated by performing analysis on uncracked and severely cracked underground structures. Based on these results, code-based linear analyses are performed with and without SSI to determine if foundation movements must be included in models used to compute seismic demands in the GLRS.

The scope of this paper is limited to assessing the influence of the flexibility of a linear homogeneous soil deposit on the seismic demands in GLRSs. Uplift of the foundation is not considered since it has been verified that no uplift occurs in the studied models. Seismic demands in the GLRS are presented only for columns in the cantilever walls direction; although analysis in the coupled walls direction has also been performed and the same conclusion about the influence of SSI can be drawn. The effects of torsion on the determination of seismic demands in the GLRS have been studied by Beauchamp *et al.* [3]; thus, torsion is not included in the present paper to simplify calculations and to focus on the effects of flexible soil conditions.

NONLINEAR ANALYSIS

Studied building

This paper presents an extensive study on the influence of soil flexibility on the demands in the GLRS of a typical building. The study is made on a 12-storey building located in Montreal (Fig. 1). This building has been chosen for many reasons: First, it is well-known by engineers in Canada, since it is one of the two buildings presented in the seismic design chapter of the *Concrete Design Handbook* (CDH) [4]. Second, its SFRSs consist of coupled walls and cantilever walls in each perpendicular direction, which permits to study the effect of these two types of SFRSs. Lastly, it has two underground storeys. Therefore, the effect of underground structure cracking can be investigated as well as the behaviour of underground columns.

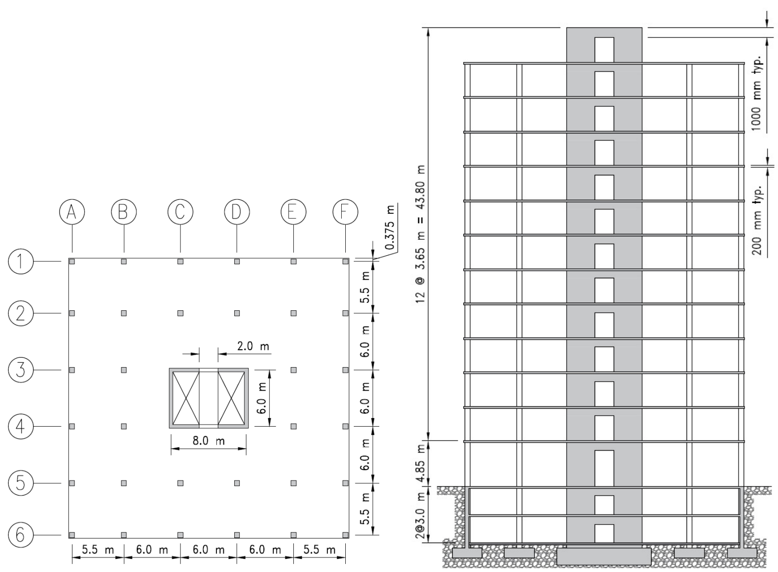

Figure 1. Plan (left) and elevation (right) of the studied building.

The seismic design of the SFRS has been performed as presented in the CDH example for a soil class D. Since the design is governed by the minimum steel requirements, the same reinforcement is used for soil classes C and E. According to the capacity design principle, reinforcement in walls under the ground level has been increased to ensure that yielding does occur in the plastic hinge zone. Hence, flexural concentrated reinforcement bars have been upgraded from 25 M to 30 M in the underground storeys.

As mentioned above, seismic design of the building has been performed and includes the underground storeys. All aboveground structural elements have been designed using the uncracked section properties of these storeys. Indeed, this model is the most stiff, which means that it has a shorter period and produces the largest seismic forces in the shear walls at the ground level. To be consistent with this approach, all analyses in this paper are first performed with uncracked section properties to determine demands in the aboveground parts of the GLRS. However, the cracked section properties of the underground structure may increase the displacements of the building and thus the forces in the GLRS. Therefore, analyses are also performed with severely cracked section properties in the underground structure. The shear stiffnesses of the diaphragms and foundation walls are then taken to be 5% and 20% of their gross values, respectively, which corresponds to the lower bound recommendation of the PEER/ATC 72-1 report [5].

Finite element modelling strategy

All nonlinear analyses in this study have been performed with *SeismoStruct* [6], which offers a user-friendly interface and a large range of uniaxial material laws for fibre-based elements. The movement of a building during an earthquake is mainly controlled by the behaviour of the SFRS. Hence, only the SFRS is modelled with nonlinear elements to obtain a realistic, inelastic displacement profile of the building.

In SeismoStruct, core walls are modelled with C-shaped nonlinear fibre-based elements. The fibre-based model is based on the plane section hypothesis. Hence, the behaviour of all fibres in the section is determined by the degrees of freedom (DOF) of a single point, which explains the efficiency of the model. Because they are based on this hypothesis, fibre-based elements are principally efficient for flexurally dominated members. On the other hand, coupling beams are modelled with elastic beams and nonlinear flexural springs, as suggested by Naish *et al.* [7].

Contrary to linear analyses, nonlinear analyses are load path dependent. Indeed, the neutral axis position in sections of fibre-based elements depends on a combination of the moment and the axial load. Thus, the specified mass needs to correspond to the expected mass and not to the factored gravity loads. In this study, the gravity load in NLTHA is taken as the seismic mass recommended in NBCC 2015 [8] plus 20% of the live load, as recommended in PEER/ATC 72-1 [5].

Since the SFRS is modelled with nonlinear elements, part of the damping is inherent to its hysteretic behaviour. Additionally, in this study, damping of the soil is explicitly modelled with dampers distributed on the base of the structure, as explained subsequently in this paper. Other sources of energy dissipation are included via Rayleigh damping that is proportional to the mass matrix and the tangent stiffness matrix with a critical damping ratio of ξ=2%.

Conditional spectrum method

NLTHA numerically imposes an earthquake on the structure and monitors its behaviour at every defined time step. Indeed, every earthquake is different and the behaviour of the structure in response to any one event may not be representative of how it will behave during the next earthquake. Thus, NLTHA requires the selection of multiple ground motions to obtain a realistic response of the structure. This response may correspond to the mean behaviour of all selected motions. In this project, each case is analyzed with 16 ground motions selected from the conditional spectrum (CS) method.

In this project, all selected ground motions are scaled to a target intensity level that corresponds to a probability of exceedance of 2% in 50 years, which is the design intensity level defined in the NBCC 2015 [8]. Ground motions are then scaled so that their spectral acceleration, at a chosen period (T^*), corresponds to the uniform hazard spectrum (UHS) prescribed by the NBCC 2015. For this project, the chosen conditioning period is the first lateral period of the structure. For concrete wall structures, the demands in the GLRS depend on the interstorey drift, and this parameter is mainly controlled by the first vibration mode.

At periods other than T^*, the acceleration spectrum need not correspond to the UHS. This would not be realistic since the UHS are defined from many records and ensure the same probability of exceedance for all periods. Baker [9] showed that using motions scaled to the UHS is too conservative and thus proposed an approach based on the conditional mean spectrum (CMS). In this study, the spectral ordinate of the CMS is forced to correspond to the value of the NBCC 2015 UHS at the period of interest, T^*. However, it will be lower at other periods, which is consistent with the probabilistic seismic hazard analysis. Ground motions are selected with an algorithm developed by Baker and Lee [10]. The algorithm ensures that spectral accelerations of the selected motions, at high and low frequencies, are sufficiently scattered to obtain a representative envelope of the response. The variability is ensured to be in the range of approximately $0.15T^*$ to $2.0T^*$. Since SSI is modelled in this study, the period of interest changes for each soil class. The CMS is then adapted for each soil class by modifying the ground motion prediction equations (GMPEs) used in the algorithm.

SOIL-STRUCTURE INTERACTION

Globally, SSI has three effects on the behaviour of a structure during an earthquake. First, it modifies the dynamic properties of the complete system. Second, kinematic effects modify the ground motion because there is a different signal at the base of the foundation than at the ground level. Third, the deformation of the soil itself can increase displacements in the structure and modify forces in structural elements. In North American standards, kinematic effects are considered using modification factors on the seismic excitation [8], [11]. Conversely, the movement of the soil deposit itself is rarely considered. Indeed, considering the flexibility of the soil deposit lengthens the fundamental period of the soil-structure system leading to a reduction in the seismic excitation. However, even if it decreases forces in the walls of concrete buildings, considering the flexibility of the soil can increase displacements and thus the deformations in the GLRS.

Direct modelling of SSI

The most complete and precise way to model SSI is called the direct method. This approach consists of modelling the complete soil-foundation-structure system by discretizing the soil deposit into finite elements. Even though this method is the most precise, it is rarely used by engineers since it is time consuming and challenging to implement. Many challenges arise with regard to performing a time history analysis on a model in which SSI is modelled by the direct method. In this project, a model has been built with an SSI modelled with the direct method to serve as a reference. It has been used to validate the precision of the other models in which SSI is modelled by the sub-structure method. This model will be described in the next section.

This model has been built in SAP2000 [12] using only elastic linear elements. The soil is modelled using 3D finite elements. The sides of the foundation are connected to soil elements by uniaxial elastic springs. On the other hand, the base of the foundation is made of thick slabs under each column with common nodes to the soil elements, which can well represent isolated footings. In this way, tension is transmitted to the soil elements during the analysis. For a linear analysis, this behaviour is acceptable since gravity loads are not applied to the model during lateral analysis, and it has been verified that no uplift would occur in the studied models if this were the case. In reality, the size of the soil deposit is essentially infinite. In the finite element model, however, the size of the soil deposit must be finite. To reproduce the infinite behaviour, non-reflecting, radiative boundaries are used in the direction of loading. These boundaries are made of viscous dampers with a value per unit surface of $V_s\rho_s$ where V_s is the shear wave velocity in the soil medium and ρ_s is the soil density. The size of the soil deposit still needs to be chosen to be large enough to ensure that the boundary conditions do not affect the building's behaviour. In this project, the width and the depth of the soil deposit are approximately 6 times and 2 times the width of the building, respectively. The last challenge is the input mechanisms for the ground motions. Available ground motions are often signals recorded on the surface (called free-field motions). However, seismic waves are modified by the mass and stiffness of the soil and by the interactions with the structure and its foundation, which means that free-field motions cannot be input at the base of the complete soil-foundation-structure system. Acceptable input mechanisms have been studied by Léger and Boughoufalah [13]. The mass of soil elements can be set to null or the signal can simply be input at the ground surface. However, a more realistic excitation of the structure can be obtained by computing a motion at the base of the soil deposit that could have caused the free-field motion. This process is called deconvolution. In this project, deconvolution has been performed by an iterative process in which the motion is modified in the frequency domain according to the signal recorded at the ground surface of the model when the free-field motion is applied at the base of the complete system.

As explained earlier, the direct method is the most rigorous approach to model SSI, and thus it has been used to validate the precision of other models in which SSI is modelled with the sub-structure method. Comparisons are made for a class D soil (Vs=250 m/s) using a single unscaled seismic record. For this building subjected to horizontal excitations, the sub-structure method shows results that are in good agreement with the reference method (Fig. 2). Interstorey drift is slightly underestimated at all heights. The shear envelope is similar for the aboveground storeys, while it is slightly underestimated for the underground storeys.

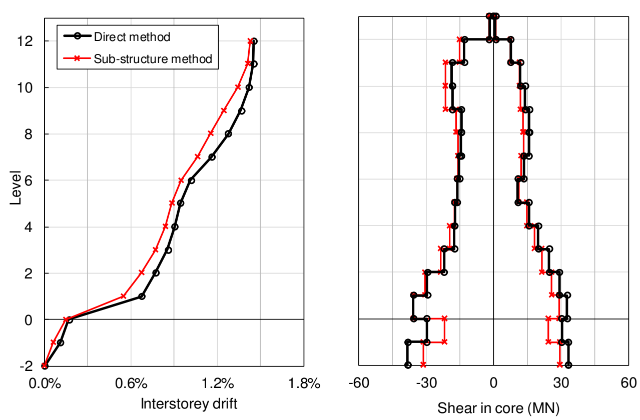

Figure 2. Comparison of the sub-structure method with the direct method.

Sub-structure modelling of SSI

In all models used to calculate results presented in the next sections, SSI has been modelled using the sub-structure method. This method is an equivalent approach to model SSI. It consists of applying the motion to the structure in which soil properties are simulated by a group of equivalent springs and dashpots distributed on the foundation. In this project, the sub-structure method has been modelled using the bathtub approach (Fig. 3). This approach, which is recommended in the report NIST GCR 12-917-21 [14], primarily consists of applying the same signal to all springs and dashpots as if the signal was transmitted through a rigid "bathtub".

The sub-structure method requires the computation of equivalent soil properties that are assigned to each spring and dashpot. Soil dynamic properties, known as *impedance functions*, depend on the foundation geometry, the nature of the soil and the excitation frequencies. The derivation of the impedance functions is very challenging. Thus, impedance functions for this project have been calculated using equations for the simple cases of foundations laying on the linear homogeneous half space proposed in Pais and Kausel [15] and recommended in NIST GCR 12-917-21 [15]. All parameters that need to calculate these properties are presented in Table 1. The shear wave velocities (V_{s30}) are taken according to the NBCC 2015 values for each soil class [8]. The values of the soil hysteretic damping ratio (β_s) and effective soil modulus ratio (G/G_0) are taken from tables proposed in Chapter 19 of ASCE 7-16 [11]. The maximum shear modulus of the soil at small strain levels (G0) is calculated according to Eq. (1). Other soil parameters are average values taken from the literature and experience.

$$G_0 = \rho_{soil} V_{s30}^2 \tag{1}$$

The sub-structure method, shown in Fig. 3, has been developed for a concrete building with isolated footings under cores and columns. At the surface, the soil is often not sufficiently compacted to offer a passive lateral resistance; thus, horizontal springs and dashpots are not assigned at the ground level. Simple impedance functions are based on the hypothesis that the foundation acts as a whole. However, isolated footings allow the core and the columns to rotate independently. Thus, the use of uniaxial springs and dashpots distributed under the foundation neglects the important rotational stiffness of the core footing. To solve this problem, all joints under the core are constrained to act as a "rigid plate", and a rotational spring is added (Fig. 3).

This method has been preferred to the direct method to model SSI for all analyses. It was the intention of the authors to propose an approach to consider foundation movements in the calculation of seismic demands in the GLRS that is workable for practicing engineers. Nevertheless, as shown in the last section, this method has been validated with the direct method, and it has shown satisfying results.

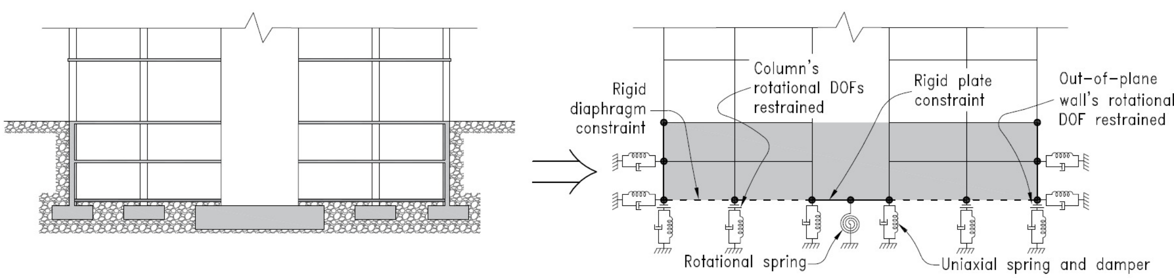

Figure 3. Conceptual model of the sub-structure method for modelling SSI.

Influence on demands in GLRS

To analyze the influence of SSI on seismic demands in the GLRS, NLTHA have been carried out for soil classes C, D and E. Moreover, for each soil class, analyses have been performed on uncracked and on severely cracked underground structures to investigate this effect. All the important SSI related parameters used for the analyses are shown in Table 1. As explained in the introduction, the results are presented only for the cantilever walls direction; although analysis in the coupled walls direction has also been performed and the same conclusions about the influence of SSI can be drawn.

Table 1. Soil related parameters used in the analyses.

Soil Class	β_s	$B_{Rayleigh}$	G/G_0	V_{s30}	ρ_{soil}	G_0	ν
	-	-	-	(m/s)	(kg/m³)	(kPa)	-
C	1.39%	2%	0.91	450	2000	405000	0.33
D	2.98%	2%	0.82	250	1900	118750	0.33
E	8.30%	2%	0.48	115	1800	23805	0.33

The results of NLTHA are shown in Fig. 4. In the first graph (Fig. 4a), the elastic absolute bending moment is shown for three locations in the columns of the GLRS. This is shown for columns F6 and C5 (Fig. 1) at the ground level and for the column C5 at the base. Corner and interior columns present quite different behaviours since the latter extends into the underground storeys and the former extends into the rigid foundation walls. Dashed and solid lines illustrate results for the severely cracked and uncracked underground structures, respectively.

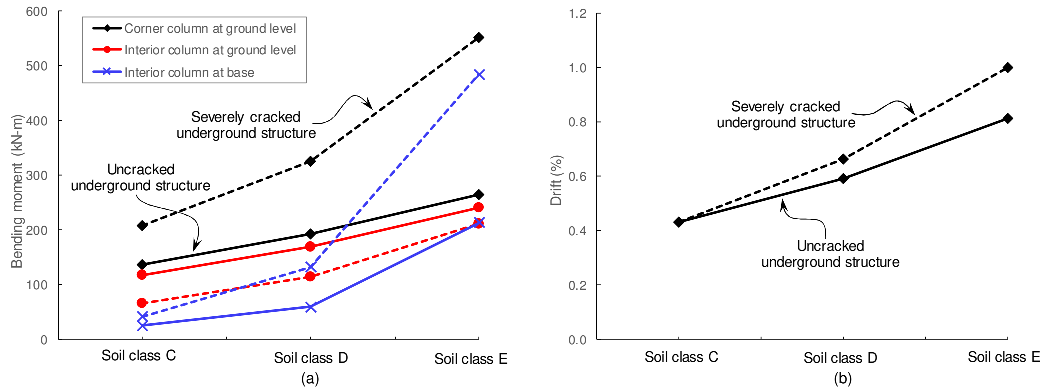

Figure 4. Results of NTLHA: (a) bending moment in columns, (b) drift at top of the building.

As expected, the bending moment increases as the soil gets softer for all models. The flexibility of the soil seems to have more influence on the interior column at the base, especially for soil classes D and E. For the corner column and for the interior column at the base, the bending moment is larger when the underground structure is severely cracked. The opposite behaviour is observed for the interior column at the ground level because when the bending rigidity of the ground slab is reduced, the base of the interior column at ground level can rotate more freely which reduces its bending moment demand. Overall, the larger bending moment demand is on the corner column at ground level when the underground structure has cracked properties.

The drift at the top of the building is shown in the second graph (Fig. 4b). Again, the drift increases as the soil gets softer and increases faster when the underground structure has cracked properties.

CONSIDERING SSI IN LINEAR CODE BASED GLRS ANALYSIS

NLTHA has shown that demands in the GLRS increase as the soil gets softer. This behaviour was expected, and the NBCC has already recommend soil class related modification factors for the seismic excitation. However, the movement of the soil itself may increase the deformations even more in the GLRS, and it may be important to explicitly consider foundation movements in code based GLRS analyses. Thus, linear models have been built in SAP2000 [12] to calculate demands in the GLRS as it would be calculated by engineers. The first model is fixed-base. A second model includes SSI with the sub-structure method, as described earlier. A third model that is between these first two is also analysed. In this third model, only the rotational spring under the core is kept, and all columns and perimeter walls are fixed-base. All these models are compared to NLTHA to evaluate their precision.

In all models, demands in the GLRS have been calculated following the procedure proposed by Beauchamp *et al.* [3]. In this procedure, the stiffness of the elements part of the GLRS is reduced by $F_{sr}=10^{-2}$ so that they do not affect the behaviour of the SFRS. This model is then identified as "*gravity nearly null stiffness*" (GNS). Forces in the elements part of the GLRS are directly computed for each mode using RSA analysis and then combined with the appropriate method. These forces are very small because the reduction factor is applied; therefore, the design forces in the GLRS are obtained by inversely multiplying by Eq. (2).

$$F_{SRCG} = F_{GNS} \times \frac{1}{F_{sr}} \times \frac{V_d}{V_e} \times \frac{R_d R_o}{I_e} \tag{2}$$

To consider the inelastic displacement profile of the SFRS, the core walls' stiffnesses have been reduced by half over the height of the plastic hinge region, which extends over the first three levels of the building studied in this paper. For the purpose of comparison, demands in the GLRS have also been calculated with the simplified method proposed in CSA A23.3-14 (labelled CSA in figures below), including foundation movements [2].

Comparison of models to NLTHA

Figs. 5 through 7 show the elastic bending moment in the gravity columns of the building calculated with the different models. The results are presented for the same column locations as in the NLTHA results discussed earlier. Left and right graphs present results for the uncracked and severely cracked underground structure, respectively. One standard deviation error is also illustrated for NLTHA.

Fig. 5 shows the bending moment in the corner column (F6) at the ground level. All models are precise for soil class C. For soil classes D and E, the fixed-base model underestimates the demand. The sub-structure and rotational spring model overestimate the demands for soil class E when the underground structure has cracked properties. Fig. 6 shows the bending moment in the interior column (C5) at the ground level. For this location, all models are precise for soil C for uncracked underground structures; however, they all overestimate the demands when the underground structure has cracked properties. The fixed-base model stays closer to NLTHA compared to all other methods, especially for soil class E where all other methods greatly overestimate the bending moment. Fig. 7 shows the bending moment in the interior column (C5) at the base. Again, all models are somewhat precise for soil class C. For this location, the fixed-base model does not capture the behaviour of the column and underestimates the demand. Only the sub-structure model seems to capture the behaviour; although it slightly underestimates the demand.

Globally, the sub-structure model is the best fit compared to NLTHA. This was expected since SSI in NLTHA was also modelled with the sub-structure method. Figs. 5 and 7 show that, when calculating seismic demands in the GLRS, it may be unsafe to use a simple, fixed-base model for soil classes D and E. However, for all methods, the behaviour of the building is not well captured for soil class E. In all cases, the simplified method of CSA A23.3-14 generally overestimates demands for soil classes D and E. It is also not capable of estimating the demands of columns in underground storeys.

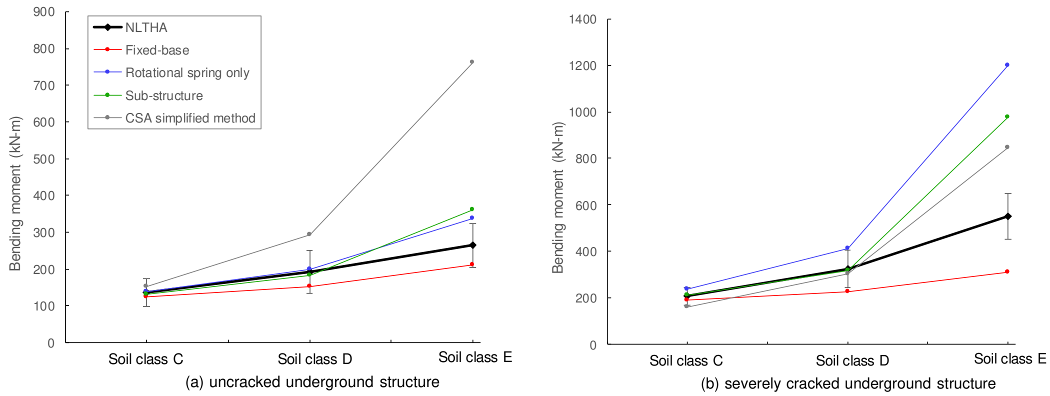

Figure 5. Elastic bending moment in the corner column at ground level. [Colour online].

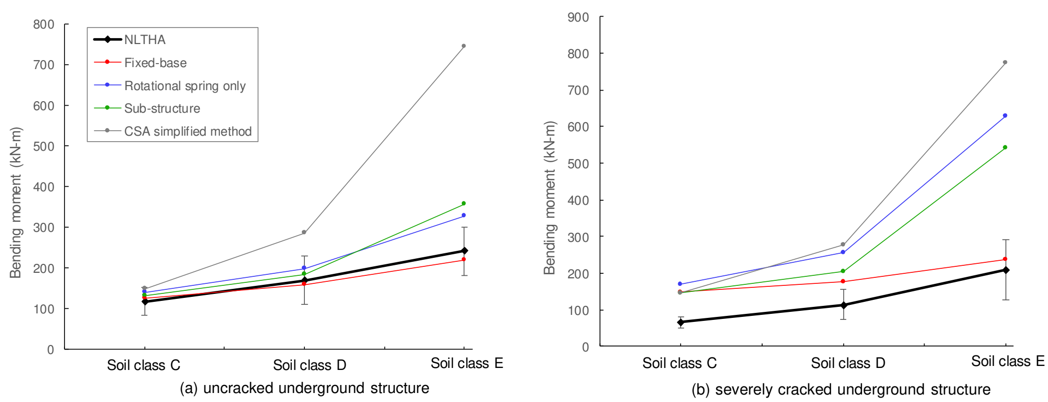

Figure 6. Elastic bending moment in the interior column at ground level. [Colour online].

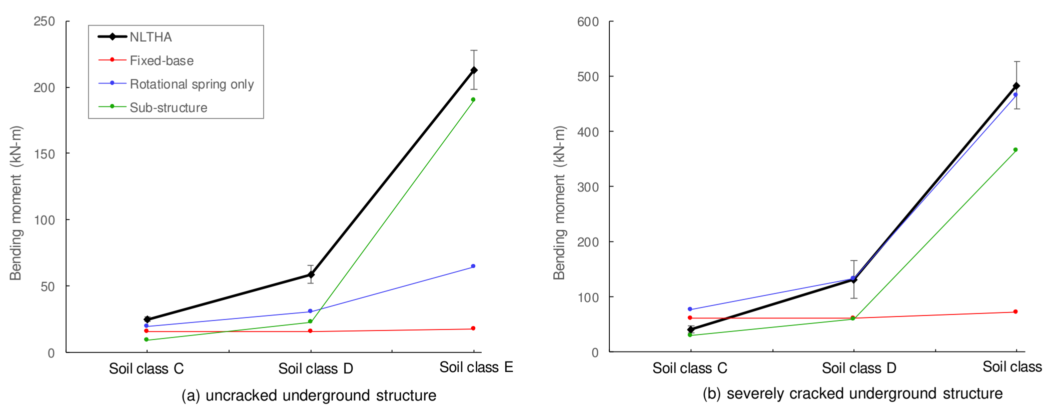

Figure 7. Elastic bending moment in the interior column at the base. [Colour online].

CONCLUSIONS

This report describes the influence of SSI on the calculation of seismic demands in the GLRS. Many nonlinear analyses, including SSI, have been performed on a typical 12-storey shear wall building for three different linear soil media, ranging from soil class C to E. SSI has been modelled using the sub-structure method. A model in which SSI has been modelled with 3D finite elements has also been used to validate the latter for the building studied in this paper. The effect of underground structure cracking is also investigated by performing analysis on uncracked and severely cracked underground structures. NLTHA results have shown that the bending moment effectively increases in gravity columns as the soil gets softer. The larger bending moment demand has been calculated on the corner column when the underground structure has severely cracked properties. Based on these results, code-based linear analyses have been performed with and without considering SSI. These analyses have shown that, when calculating seismic demands in the GLRS, it may be unsafe to use a simple, fixed-base model for soil classes D and E. The behaviour is better captured when foundation movements are modelled with the sub-structure method; however, using a well-calibrated, single, rotational spring under the core walls may be a reasonable assumption. For soil class E, the behaviour of the building is not well captured by all linear analyses. Hence, for soil class E, nonlinear analysis is required. However, analyses of many other buildings in which parameters such as the height, number of underground storeys and layout configuration are varied would be needed to more effectively assess the conclusion proposed herein regarding the importance of considering foundation movements in the calculation of seismic demands in the GLRS.

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