Numerical modelling of dynamic soil-structure interaction for seismic conditions in Eastern Canada

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ABSTRACT

It is well known that dynamic soil-structure-interaction (DSSI), in the seismic evaluation of existing structures provides, more realistic and accurate results. Most often, DSSI effects are studied using the substructure-based approach in which the structure and the soil are treated separately. As an alternative, a direct method could be used to model and solve simultaneously the soil and structure. The objective of this paper is to investigate the applicability of the substructure-based approach to consider DSSI in the evaluation of existing buildings in Eastern Canadian geo-tectonic context. The study is carried out for a hypothetical 3-storeys 3-bays reinforced concrete (RC) building, located in Quebec City. The structure is constructed on deep post-glacial fine-grained soil deposits with geotechnical properties typical of Eastern Canada. Three finite element numerical models are built in the Opensees platform. The first model neglects DSSI while the other two consider its impact using the substructure-based and the direct approaches. Non-linear time history analysis is carried out for a series of 10 synthetic ground motion records compatible with NBCC-2015 spectrum for design location. Structural response is evaluated by tracking the horizontal accelerations and drifts profiles. Results show that for M_w 6 earthquakes the substructure-based and direct methods give similar values of floor acceleration that are smaller than those obtained from the fixed base model. For M_w 7 earthquakes the two modelling approaches give different results both for acceleration and drift values.

Keywords: Dynamic-Soil-Structure-Interaction, Finite Element Modelling, Non-Linear Time-history analysis, Eastern Canada

INTRODUCTION

Recent propositions to consider dynamic-soil-structure-interaction (DSSI) in the evaluation process of existing structure as a mean to reduce seismic force demand has sparked renewed interest in the study of this complex phenomenon [1, 2]. DSSI evaluation is often conducted by the means of the substructure-based approach [3], also known as the 3-step method [4]. This approach is based on the principle of superposition which allows the decoupling of the soil-structure system into two main components: the soil and the structure. The analysis is done in the time domain. The flexibility and damping characteristic of the supporting soil are modelled using discrete elements (spring & dashpot). The foundation input ground motion (u_{fim}), is obtained from a wave passage analysis. As an alternative, the direct approach can be used. In this approach, the structure and soil are integrated in the same numerical model and solved simultaneously thereby offering a more refined analysis of the problem. However, this approach is harder to implement and more time consuming.

The substructure method has been developed and calibrated for soil conditions and seismic context of California, Japan and Western Canada, regions which greatly differ in terms of seismic and geologic conditions from those found in Eastern Canada [5]. Earthquakes occurrences in Western Canada and California arise from interplate activities while Eastern Canadian earthquakes are intraplate. In addition, Eastern Canadian ground motion records have predominant high frequency content [6]. Typical soil conditions found in Eastern Canada include sensible post-glacial soil [7] presenting a high potential for seismic wave amplification due to the high impedance contrast between the soil and the bedrock [8]. Major urban centres are located on such post-glacial soil, like Ottawa, Montreal or Quebec City [9].

In this context, this paper aims to validate the use of the substructure-based approach to consider DSSI in the evaluation of existing buildings for the geo-tectonic context typically found in Eastern Canada. To do so, the dynamic response of a 3-storeys 3-bays reinforced concrete (RC) structure, located in the city of Quebec is evaluated using three numerical models developed in OpenSees (OS). The building is founded on a deep Champlain's sea natural soil deposit of class site E. This post-glacial sensible deposit has geotechnical properties typical of Eastern Canada. The nonlinear time history analyses are carried out for a set of synthetic ground motion records selected and calibrated following the recommendation for buildings located in Eastern North America [10]. The response of structure-foundation-soil system is examined through floor acceleration and drifts profiles.

METHODOLOGY

The plan view and typical elevation of the studied hypothetical building is illustrated in Figure 1. This 3-storey building is representative of 1970's institutional building construction in the province of Québec. The building was designed in accordance with the provisions of NBCC 1965. Further details regarding the design can be found in [11]. Lateral loads are resisted by three RC frames in the long direction and four frames in the short direction. This paper focuses on one of the three-bay interior frames in the short direction (circled in red on Figure 1). The RC frame is founded on four distinct superficial footings, located 2 m below ground level for frost protection.

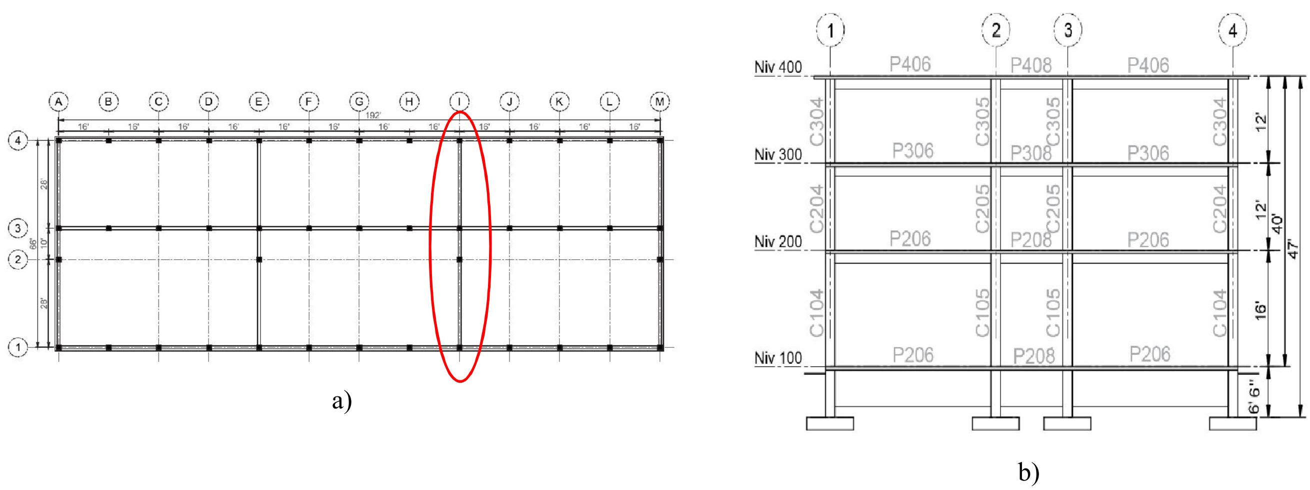

Figure 1: a) Floor plan [11] and b) elevation view of studied building

Typical Eastern Canadian soil geotechnical properties used in the analyses are derived from field [12] and laboratory [13] data. This allows a clear definition of the soil constitutive relationships. The natural soil deposit is considered to be 64.5 m deep and is a Champlain's sea post-glacial sensible clay.

In the first series of analyses (M1-Fixed,) the structure is considered fixed at its base, thereby disregarding the effects of DSSI. In the second model (M2-Substructure), the substructure approach is applied to account for DSSI effects. In the third model (M3-Direct), more realistic DSSI consideration is achieved by implementing a direct approach [3] in which the soil and the structure are modelled together in the same numerical model and solved simultaneously.

All models (structure and soil) are constructed in two dimensions (2D) in the finite element platform OS [14] and are solved in the time domain. Each model is subjected to a set of 10 synthetic ground motions selected from Atkinson database [15] considering dominant magnitude-distance scenarios for Quebec City. The ground motions are scaled to achieve compatibility with NBCC-2015 design spectrum for class A site to be applied at the base of the soil model. The selected set is composed of five records from magnitude M_w 6 earthquakes and five records from magnitude M_w 7 earthquakes. The calibrated elastic response spectra with 5% damping (ERS) are shown in Figure 2. Spurious numerical acceleration generation during analysis resulting for the application of the Newmark algorithm is prevented by smoothing each of the signals with a linear baseline correction.

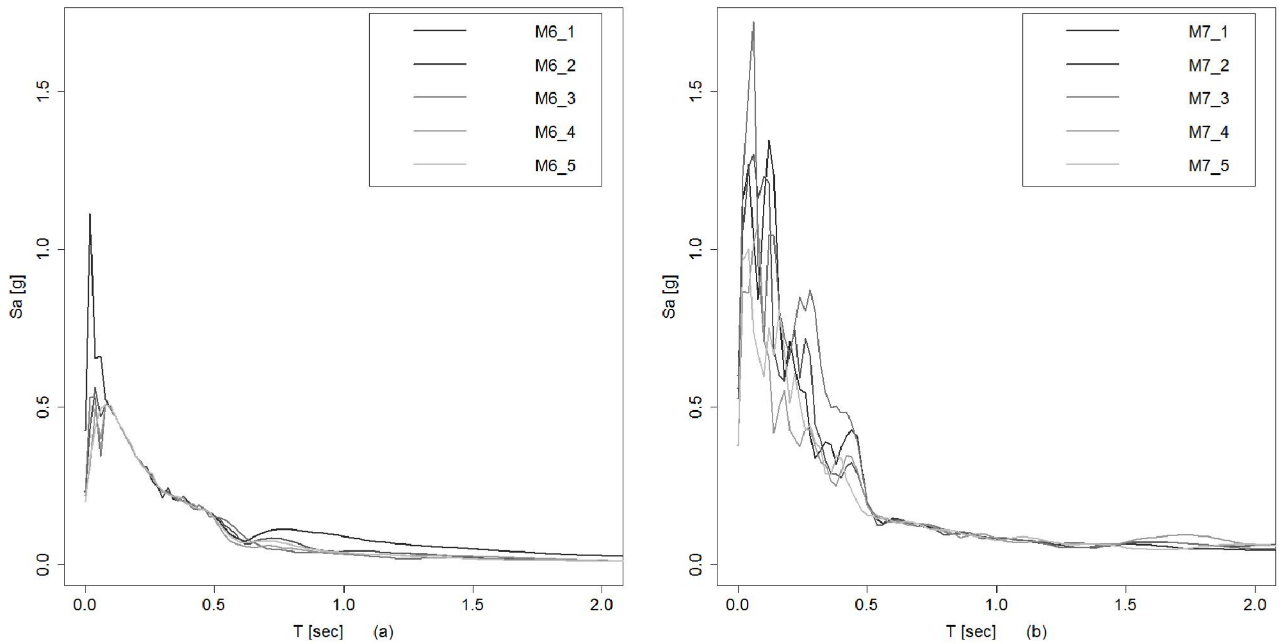

Figure 2 : ERS - calibrated seismic signal (a) Mw 6 (b) Mw 7

DESCRIPTION OF THE NUMERICAL MODEL

Structural model

The numerical model of the frame structure is built using non-linear *BeamColumn* elements available in OpenSees element library. Each section was discretized into fibres for which the nonlinear material stress-strain response was defined. Distinct fibres were defined for confined and unconfined concrete zones and for the steel reinforcement. The fibre section model considers the bending moment and axial load interaction, but the shear-bending or shear-axial load interactions cannot be represented. To represent concrete inelastic behaviour, the uniaxial Kent–Scott–Park model with linear tension softening (*Concrete02*) [16] was applied. The Giuffré-Menegotto-Pinto (*Steel02*) [17] hysteretic material is employed to describe the inelastic behaviour of the reinforcing bars. Specific values used in the definition of the steel and concrete model can readily be found in [11]. Non-linearity solution of the *BeamColumn* elements is based on the iterative force procedure using Gauss-Lobatto integration point. To ensure a proper local solution, a total of 7 integration points are selected on each member following the recommendations found in literature [18].

To capture the amplification effect of the soil column, the seismic signal (scaled for class A site) is first applied at the base of the numerical soil model in OS. A wave passage analysis is conducted considering the massless structural elements and the resulting foundation input ground motion u_{FIM} is then used for models M1-Fixed and M2-Substructure. For model M1-Fixed, the bases of the column of the structural model are fixed. The seismic signal is therefore directly applied to these nodes. For model M2-Substructure, a series of springs and dashpots is added at the end of each column to simulate the flexibility and damping of the supporting soil. The u_{FIM} is then applied at the supporting node of these elements.

For model M3-Direct, the node composing the footing and the corresponding node constituting the soil element are directly linked using coupling relation. Nodes used in the soil domain have 2DOF while nodes of the structural model have 3DOF. An additional series of nodes, referred to as ghost nodes, is inserted at the soil-structure interface. The ghost nodes have no physical meaning; they merely serve as a numerical bridge. The ghost nodes are defined in the structural domain so they have 3DOF but have their 3rd DOF fixed, i.e. zero rotation. The soils nodes displacements along their 2DOF are prescribed to coincide with the 2 translational DOF of the ghost node. The ghost nodes are then linked to the structural nodes located at the base of the footing using *zerolength* element. The latter can be assigned specific uniaxial material properties independently of the direction considered. This approach allows to define different contact conditions in the horizontal and vertical direction and in rotation. In this study a glued type of link at the soil structure interface was selected so that no slippage nor lifting of the footing is possible. The seismic signal is thus applied at the base of the soil model which readily transfers the excitation to the structure.

Soil model

The soil model is built using an assembly of quadrilateral single stabilized integration points [19]. Sizes of the element are 0.5 m x 0.5 m and the total mesh of the soil is 140 m x 64.5 m, for a total of 36120 elements. The material behaviour used to model the dynamic behaviour of the soil is the *PressureIndependentMultiYield* material (PIMY) [20]. The PIMY material is suited to simulate the cyclic response of material, such as clay, whose shear behaviour is insensitive to confinement change. The backbone curve of the PIMY material is piece-wise defined and is directly related to the number of yield surfaces of the material (from 1 to 40), as shown in Figure 3.

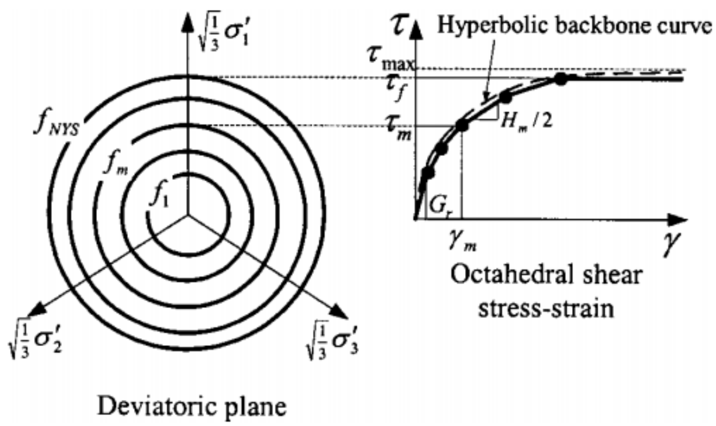

Figure 3 : PIMY - Relation between the number of yield surface and the backbone curve [20]

Lateral confinement of the numerical model is ensured using lateral forces applied on each side of the model. The magnitude of confinement forces is determined from high stiffness springs attached to each lateral node during the static analysis phase. The magnitude of the resisting forces is then used as confinement forces. To ensure proper wave propagation, the tied degree of freedom approach (TDOF) [21] is used. The bottom frontier is considered to be rock, thereby fixed in the vertical direction but free to move in the horizontal direction.

RESULTS

This section presents a comparison of the results for floor accelerations and drifts obtained from the three modelling approaches. Drifts are expressed as a percentage of the building height (Δ %) and determined relative to the displacement of the structure's base.

Acceleration

Figure 4 (a) and (b) compare the maximum and minimum floor accelerations obtained for individual ground motion record from M6 and M7 group for each studied model. Mean values are also plotted in thicker lines for easier comparisons.

The three models submitted to the M_w 6 records give relatively similar results. Figure 4(a) shows that the Positive mean accelerations of model M2-Substructure are lower than those recorded for model M1-Fixed (-26.18%) and for model M3-Direct (-17.7%) at all storeys, except at the roof level where the acceleration of M2-Substructure exceeds that obtained for model M3-Direct (+23%). Mean negative accelerations of model M2-Substructure are lower than those of model M1-Fixed (-34.23%) and of model M3-Direct (-62.37%). The *mean interval values,* determined as a difference between the positive and negative values on each floor for each of the model, are equal to 3.8 m/sec², 3.35 m/sec² and 2.96 m/sec² for models M1-Fixed, M2-Substructure and M3-Direct models, respectively. These values demonstrate that model M1-Fixed experienced larger acceleration shift compared to M2-Substructure and M3-Direct models.

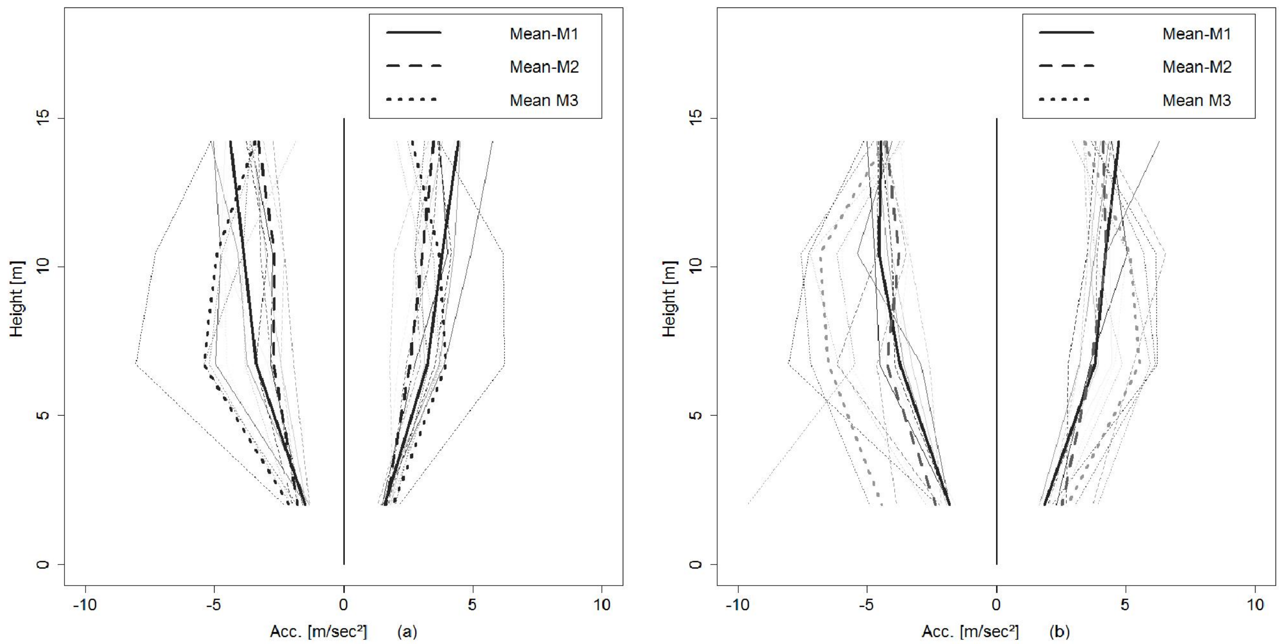

Figure 4 : Floor accelerations profiles: Comparison of model M1-Fixed, M2-Substrucure and M3-Direct (a) M_w=6 (b) M_w=7

As seen in Figure 4 (b), positive mean accelerations for M_w 7 ground motion records obtained for model M2-Substructure are lower than those obtained for models M1-Fixed (-6.09%) and model M3-Direct (-18.61%). Negative mean accelerations recorded from model M2-Substructure are also lower than those of model M1-Fixed (-4.09%) and of model M3-Direct (-26.61%). The mean interval values on each floor are 4.25 m/sec², 4.06 m/sec² and 5.29 m/sec² for M1-Fixed, for M2-Substructure and M3-Direct model respectively. For M_w 7 records, model M3-Direct shows a larger shift in acceleration values when compared with the other models.

For both M_w 6 and M_w 7 records, floor accelerations predicted by model M3-Direct tend to be lower than those predicted by M2-Substructure and M1-Fixed models at the top of the structure but they are higher at floor 1 and 2. It should be noted that the result for individual ground motion records show higher variation which makes it difficult to define a clear pattern. For cases for which larger soil displacements are expected, as in the case of M_w 7 earthquakes, the difference between M3-Direct and the other two modelling approaches is more pronounced with increased accelerations observed at the first two storeys. The amplitude of the soil displacement during the dynamic analysis also has an impact on the trend of the maximum and minimum accelerations; as seen in Figure 4, accelerations values from model M1 and M2 tend to increase with the height of the structure whereas values of accelerations obtained from model M3 do not follow this pattern and show maximum and minimum values at different floor level of the structure.

Drift results

Drift profiles of the three models calculated relative to the displacements of the base of the structure are illustrated in Figure 5 (a) for M_w 6 and (b) for M_w 7 ground motions respectively. Mean values are also shown using thicker lines for easier comparison.

Results show that mean negative values of Δ obtained from model M2-Substructure exceed those obtained from model M1-Fixed (+17.06%) and model M3-Direct (+45.03%). Mean positive drifts obtained from model M2-Substructure are also larger than those from model M1-Direct (+21.55%) but are smaller than those of model M3-Direct (-23.81%). A similar tendency is observed for M_w 7 records for which mean negative Δ values of M2-Substructure are larger than those of model M1-Fixed (+29.57%) and of model M3-Direct (+24.16%). Mean positive Δ values of M2-Substructure are larger than those of model M1-Fixed (+16.07%) but smaller than those of M3-Direct (-18.80%).

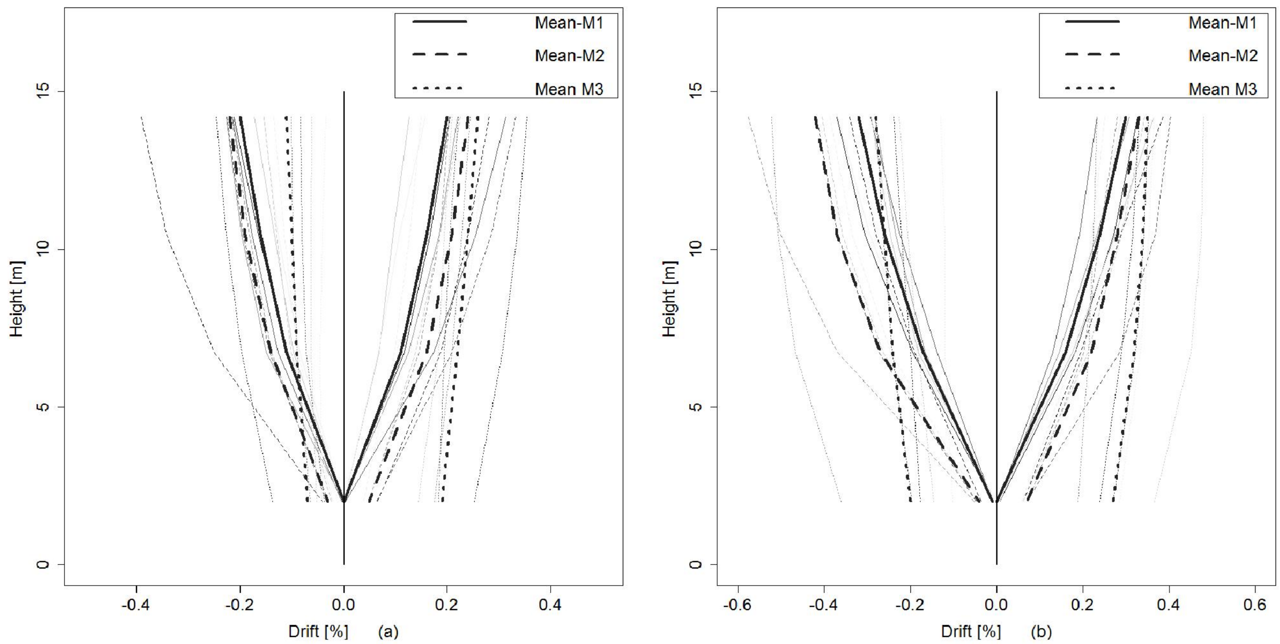

Figure 5: Drift profile - comparison of model M1-Fixed, M2-Substructure and M3-Direct (a) M_w=6 and (b) M_w=7

Contrary to M1-Fixed for which the symmetrical drift pattern is observed, when model M3-Direct is used, the resulting drift profiles are non-symmetrical with preferential display on the positive side of the vertical axis (Figure 5). This is consistent with the soil settlement observed during the dynamic analysis that produced a lateral permanent displacement of the structure.

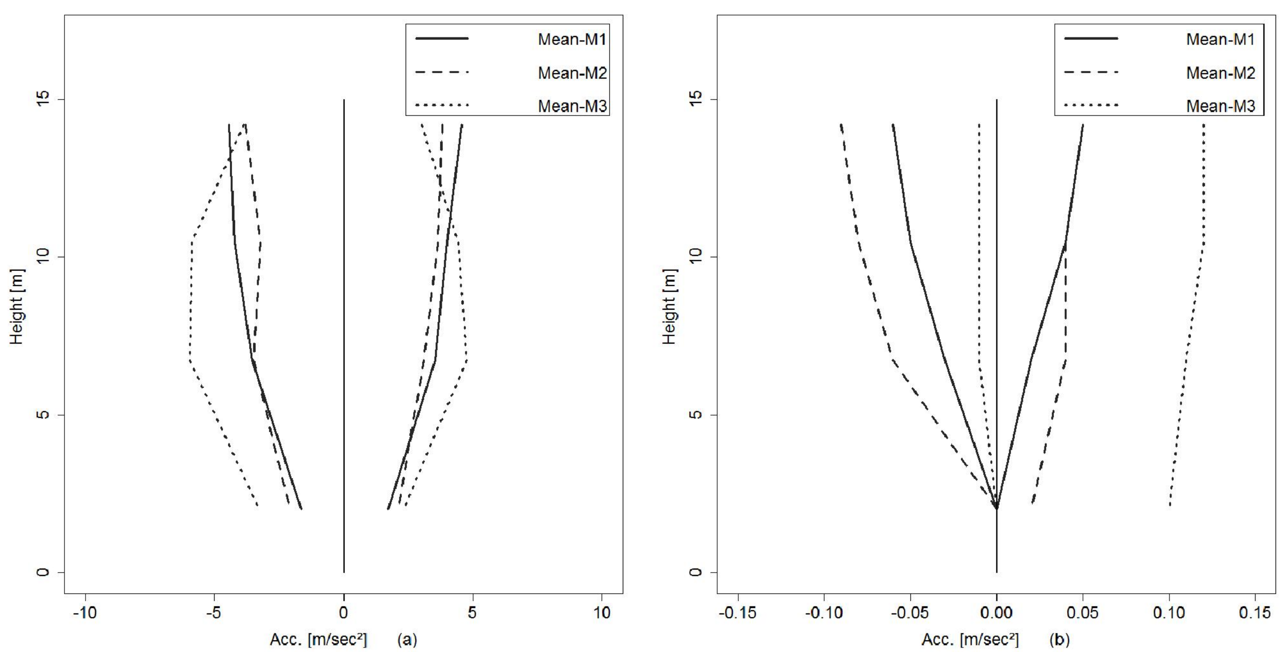

Figure 6 : Acceleration (a) and drift profiles (b) for complete set of records (M_w 6 and M_w 7)

Figure 6 (a) and (b) illustrate the profile of the mean acceleration and drifts for the complete set of records. From the result it is observed that the acceleration on the first two storeys of the structure is comparatively higher with model M3-Direct than with model M2-Substrutrure or with model M1-Fixed. However, predicted roof acceleration is higher in model M1-Direct.

Mean drift value of model M1-Fixed shows a relatively symmetrical pattern with ultimate values lower than those predicted by both model M2 and M3. Mean drift values of model M2 are not symmetrical and show higher value in the negative (left) direction. For model M3, mean value shows a preferential direction that is due to the settlement of the supporting soil which produced tilting of the structure above ground during the dynamic phase. This effect was not captured by the other models.

CONCLUSIONS

The objective of this paper was to investigate the applicability of the substructure approach in the evaluation of existing buildings for the geo-tectonic context typically found in Eastern Canada. The study was done on an example 3-storey building located in Quebec City and founded on a deep sensible soil deposit. Three modelling techniques have been compared: (i) fixed-base approach with no DSSI representation, substructure method and (iii) direct method. The latter two permitted the inclusion of DSSI effects to a different degree of sophistication. Nonlinear time-history analysis was carried out for two sets of artificial ground motion records, scaled to NBCC design spectrum. The results obtained for floor accelerations and drifts were extracted and compared.

Results have shown that for M_w 6 ground motion record, the substructure and fixed modelling of the DSSI give similar results while by comparison the direct modelling shows higher acceleration value on first and second floor and lower values on third floor. For M_w 7 records, for which expected displacements are larger, noticeable differences, are observed between the substructure and the direct approach. While the direct approach results in the highest acceleration values, the substructure method yields the highest drift values.

The results also indicate that neglecting DSSI effects (fixed-base condition) for moderate (M_w 6) earthquakes leads in general to an overestimation of the third floor accelerations. However, this observation cannot be generalized, as the complex coupling between the structure's rigidity, frequency content of the earthquake and geotechnical properties of the underlying soil could significantly affect the response.

The results have also highlighted the profound impacts of the soil settlement on the structural response during the analysis. This effect was captured by the direct approach only.

The aforementioned conclusions are based on comparative analysis hence are dependent on the modelling choice, technique employed and the system considered. The study is presently being conducted to assess the impact of these limitations so that the more general conclusion could be reached.

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