A NEW APPROACH TO RIGOROUS MODELING OF SSI

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ABSTRACT

Various methods of analysis have been proposed to inspect the effects of the underlying soil on structural responses in the existence of dynamic loads. Finite elements are usable when effects of ground half-space infinite radiation are not of interest, i.e. the problem is of static or quasi-static nature. To make the solution more precise, hybrid schemes may be used taking into account effects of the near-field and the far-field. The nonlinear behaviour of the underlying soil especially the region engulfing the foundation may have substantial effects on stories displacements. In this study, finite elements have been used to model structural as well as near-field soil elements and the Scaled Boundary Finite Element Method is used for the far-field. The near-field soil is supposed to react nonlinearly via inserting a constitutive model in the elements. The main purpose is to introduce a newly growing solution method for the inspection of soil-structure interaction (SSI). Results are verified with those derived from previous methods of analysis.

INTRODUCTION

The effect of soil on structural responses under static and dynamic loads goes back to the 19th century. Special structures such as nuclear power plants and offshore industries caused the problem to emerge as technology and structural systems evolved. Solution techniques began to advance rapidly with the appearance of powerful computers able to handle large amounts of computations in relatively short times (Kausel, 2010). As more complicated calculations get possible to be done rapidly, more advanced solution schemes appear to lead to more logical responses of the SSI problem.

The soil does not usually react elastically and this was long obvious to researchers, but modelling the soil as an elastic-plastic material was not easy until relatively recently, which will be discussed briefly shortly. Reissner (1936) explored the behaviour of disks on elastic half-spaces subjected to time-harmonic vertical loads and is known the pioneer of dynamic SSI. Reissner assumed the plate to have frictionless contact with the soil and supposed a uniform stress distribution underneath the plate. Yet, the displacement at the centre of the load was assumed to equal to that of the plate (Reissner, 1937). The problem of dynamic SSI was tried to be solved via various modelling techniques ever since.

Many of the SSI models employed up through the early 1970's were relatively "simple" in the sense that they were restricted to systems in which the foundation rested directly onto the surface of a homogeneous half-space, and the seismic motion in the free-field was invariant in horizontal planes, e.g. the motions resulted from waves propagating vertically in a laterally homogeneous soil. For such models, the intuitively obvious strategy of prescribing the free-field motion directly underneath the soil "springs" supporting the structure in a formulation in the frequency domain was both sufficient and rigorous. However, when discrete methods of analyses, such as finite elements, started being applied to SSI problems, and especially when embedded structures began to be considered, substantial discrepancies were observed between the results of the numerical analyses and the classical analytical method, which demanded an explanation as to why the differences. This motivated the development of the so-called three-step solution, which provided the means to accomplish fully consistent comparisons between the results obtained by purely numerical models with finite elements and those by the lumped parameter method based on foundation impedances or "springs" together with seismic motions prescribed underneath these springs. In this regard, soil-structure interaction was assumed to be comprised of a summation of the effects of *kinematic interaction, foundation stiffness and inertial interaction*. In a series of analytical works, Lysmer and Richart (1966) commenced solving the problem of SSI with numerical methods and suggested techniques to model the damping existing in the soil when the seismic wave reaches and affects the near-field of the soil-structure system. Applying this and such techniques could be, and still is, efficient enough to shift the inspections from the "sub-structuring" method to the "direct" (Lysmer & Richart, 1966).

To capture the effects of the underlying soil on the overlying structure in the occurrence of dynamic loads such as earthquake, reciprocating machinery, blasts, etc., beside conventional methods of analysis which mostly include replacing the soil with springs and dashpots, direct methods of analysis exist which serve to satisfy the necessity of exact responses for high importance and critical structural systems. These methods which as a result of their being hard to handle as well as exact in responses are also called *rigorous methods* have make use of numerical simulations through e.g. finite elements to model each portion of the media with its specific characteristics. This way, no impedance or stiffness needs to be calculated since the structure is placed directly on the soil and interacts with it with no intermediary. Considering seismic waves through the soil, things get a bit more complicated as a result of the possible inherent reflection and refraction of these waves. This will lead to looking for a suitable manner of truncating the far-field out of the soil domain. To obtain this aim, the Scaled Boundary Finite Element Method (SBFEM) is applied in this research. To replace the far-field with its reaction forces Eq. (1) is used (Wolf & Song, 1996):

$$\left\{R(t)\right\} = \int_0^t \left[M^{\infty}(t-\tau)\left\{\ddot{u}(\tau)\right\}\right] d\tau \tag{1}$$

where R is the time-pulse dependent reaction force, M^{∞} is the unit impulse response matrix, and $\ddot{u}$ is the earthquake acceleration in each time step. The discretization of the whole system into near- and far-fields and the superstructure is illustrated in Figure 1.

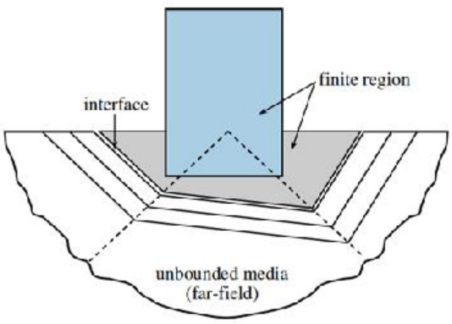

Figure 1. Decomposition of the soil-structure system into three sub-regions (after Genes, 2012)

The total equation of motion (Eq. 2) is used to engulf Eq. (1) to yield the final solution scheme of the far-field (Wolf, 2003).

$$\begin{bmatrix} M_{ss} & M_{sb} \\ M_{bs} & M_{bb} \end{bmatrix} \begin{Bmatrix} \ddot{u}^t{}_s(t) \\ \ddot{u}^t{}_b(t) \end{Bmatrix} + \begin{bmatrix} K_{ss} & K_{sb} \\ K_{bs} & K_{bb} \end{bmatrix} \begin{Bmatrix} u_s^t(t) \\ u_b^t(t) \end{Bmatrix} = \begin{Bmatrix} \{0\} \\ -\{R(t)\} \end{Bmatrix} \tag{2}$$

After exerting the Scaled Boundary Finite Element Method to the total equation of motion, Eq. (3) will result which brings into account effects of the far-field and radiation damping.

$$\begin{bmatrix} M_{ss} & M_{sb} \\ M_{bs} & M_{bb} \end{bmatrix} \begin{Bmatrix} \ddot{u}_s^t(t) \\ \ddot{u}_b^t(t) \end{Bmatrix} + \begin{bmatrix} [0] & [0] \\ [0] & [C_\infty] \end{bmatrix} \begin{Bmatrix} \dot{u}_s^t(t) \\ \dot{u}_b^t(t) \end{Bmatrix} + \begin{bmatrix} K_{ss} & K_{sb} \\ K_{bs} & K_{bb} \end{bmatrix} \begin{Bmatrix} u_s^t(t) \\ u_b^t(t) \end{Bmatrix} + \begin{Bmatrix} \{0\} \\ \int_0^t \left[S_r^\infty (t - ²) \right] \left\{ u_b^t(²) \right\} d² \end{Bmatrix} \tag{3}$$

$$= \begin{Bmatrix} \{0\} \\ [C_\infty]\left\{ \dot{u}_b^g(t) \right\} + [K_\infty]\left\{ u_b^g(t) \right\} + \int_0^t \left[S_r^\infty (t - ²) \right] \left\{ u_b^t(²) \right\} d² \end{Bmatrix}$$

In the equation above $[S_r^\infty]$ represents the dynamic stiffness matrix of the unbounded media in time domain, $[K_{ij}]$ is the static stiffness matrix of the elements of the bounded near-field, unbounded far-field and elements on the near- and far-field interface. ² is each time step at which the earthquake pulses are recorded.

For the near-field, the site is supposed to consist of once sand and once clay using the UCSD soil constitutive model which will be embedded in the finite elements. Details of this constitutive model are too extravagant for the scope of this abstract. The sample principle effective stress, deviatoric plane and octahedral shear stress versus shear strain for the sandy soil of this model is depicted in Figure 2. It should be noted that the study is carried on both sandy and clayey fields.

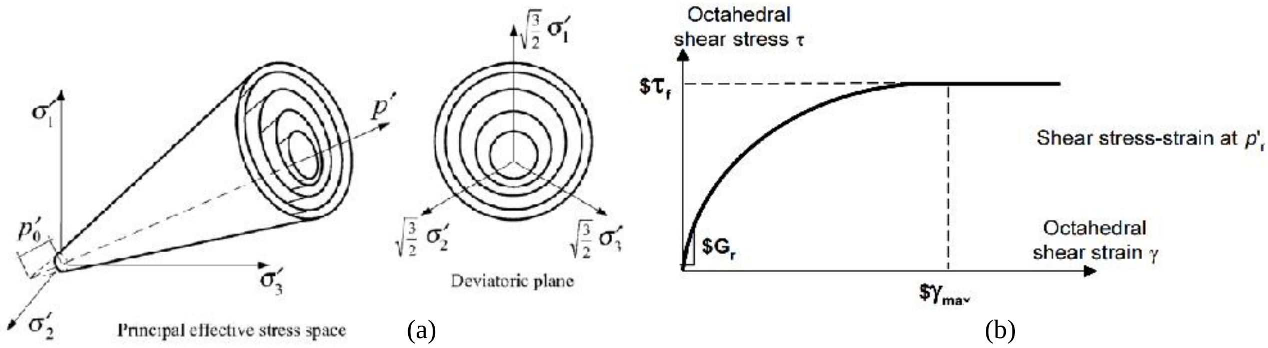

Figure 2. (a) USCD soil model in principal effective stress space and deviatoric plane, (b) Octahedral shear stress vs. shear strain (after Yang et al., 2008)

where G_r, B_r and γ_{max} are respectively reference low-strain elastic shear modulus, reference bulk modulus and the maximum shear strength at reference strains, at which the maximum shear strength is reached. p'_r is the reference mean effective confining pressure at which G_r, B_r and γ_{max} are defined. Reference parameters are defined in such models to account for changes of mechanical properties with reference to the highest elastic deformation after which non-elastic strains begin to appear. All parameters indexed r are measured and utilized at same strains in each step. σ_1', σ_2' and σ_3' are effective principal stresses. γ is defined by Eq. (4).:

$$¥ = 2/3 \left[\left(¶_{xx} - ¶_{yy} \right)^2 + \left(¶_{yy} - ¶_{zz} \right)^2 + \left(¶_{xx} - ¶_{zz} \right)^2 + 6¶_{xy}^2 + 6¶_{yz}^2 + 6¶_{xz}^2 \right]^{1/2} \tag{4}$$

Using Eq. 4 octahedral shear strains may be achieved. $¶_{ij}$ are normal strains parallel to j and normal to the i axes. The three coordinate axes x, y and z are considered based on which normal strains are assumed to be considered. τ_f is the maximum octahedral shear strength which is related to the effective confining pressure by the internal friction angle of the soil as shown in Eq. (5):

$$²_f = \frac{2\sqrt{2}\sin ¨}{3 - \sin ¨} p' \tag{5}$$

Eq. (5) represents how the shear strength of the soil is mobilized in practice, where ¨ is the mobilized internal friction angle of the soil. The total octahedral stress equation is:

$$^{2} = 1/3\,[(\pm_{xx} - \pm_{yy})^2 + (\pm_{yy} - \pm_{zz})^2 + (\pm_{xx} - \pm_{zz})^2 + 6\pm_{xy}^2 + 6\pm_{yz}^2 + 6\pm_{xz}^2\,]^{1/2} \qquad (6)$$

σ_{ij} are normal stresses parallel to j and normal to the i axes.

The scheme of the problem including the superstructure, the near-field and the near-field and far-field interface in illustrated in Figure 3. The mechanical properties used in order to define the soil in the near-field are given in Table 1.

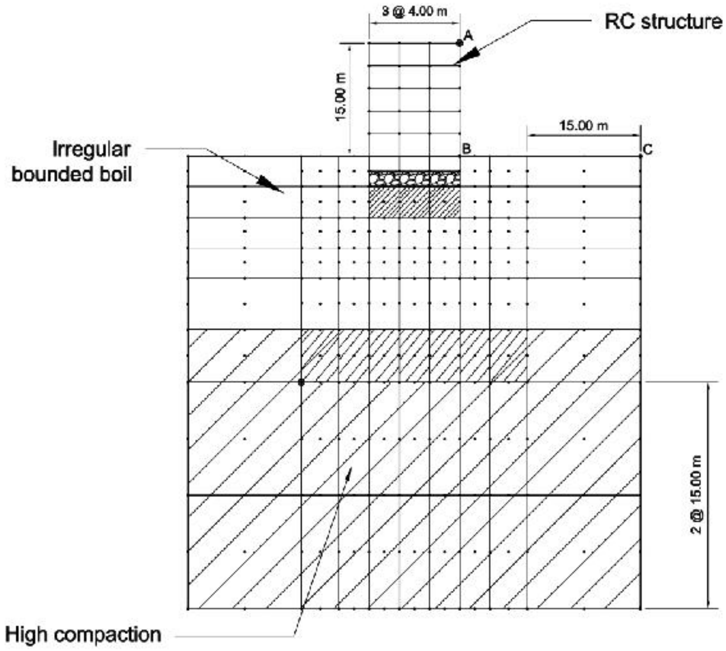

Figure 3. Near-field soil along with the RC frame placed upon

Table 1. Suggested values for soil parameters in the UCSD model (after Yang & Lu, 2008)

	Loose Sand (D_r = 15%-35%)	Medium-dense Sand (D_r = 65%-85%)	Dense Sand (D_r = 85%-100%)
Density (ton/m^3)	1.7	2.0	2.1
Reference shear modulus at p_r' = 80 (kPa)	5.5×10^4	1.0×10^5	1.3×10^5
Reference bulk modulus at p_r' = 80 (kPa)	1.5×10^5	3.0×10^5	3.9×10^5
Friction angle (degrees)	29	37	40
Peak shear strain at p_r' = 80 (kPa)	0.1	0.1	0.1
Reference pressure (p_r')	80	80	80
Pressure dependence coefficient	0.5	0.5	0.5
Phase transformation angle (degrees)	29	27	27
Porosity (e)	0.85	0.55	0.45

Figure 4 manifests the cross section of all beams and columns of the frame so as to meet the stiffness of the work carried by Çelebi et al. (2012).

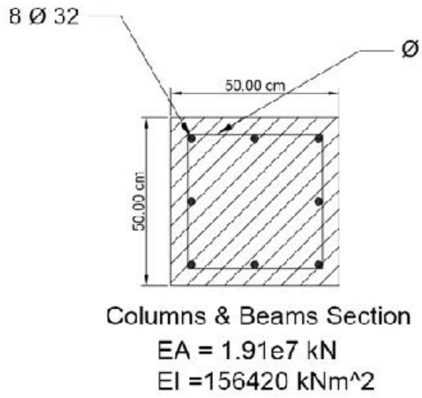

Figure 4. Beam and column elements cross section

The mechanical properties of the comprising materials of the superstructure are given in Tables 2 and 3.

Table 2. Mechanical properties of concrete for non-linear structural behaviour

Mechanical Properties	Characteristic Strength (kPa)	Strain in Maximum Strength	Crushing Strength (kPa)	Strain before Crushing	Tension Strength (kPa)
Core Concrete	24×10^3	0.0024	5.6×10^3	0.015	0
Cover Concrete	21×10^3	0.002	5×10^3	0.005	0

Table 3. Mechanical properties of steel for non-linear structural behaviour

Mechanical Properties	Yield Stress (kPa)	Initial Modulus of Elasticity (kPa)	Strain Hardening Ratio
Reinforcing Steel	420×10^3	2×10^8	0.01

The Loma-prieta (1989) earthquake record as depicted in Figure 5 was used to carry out time-history analyses with.

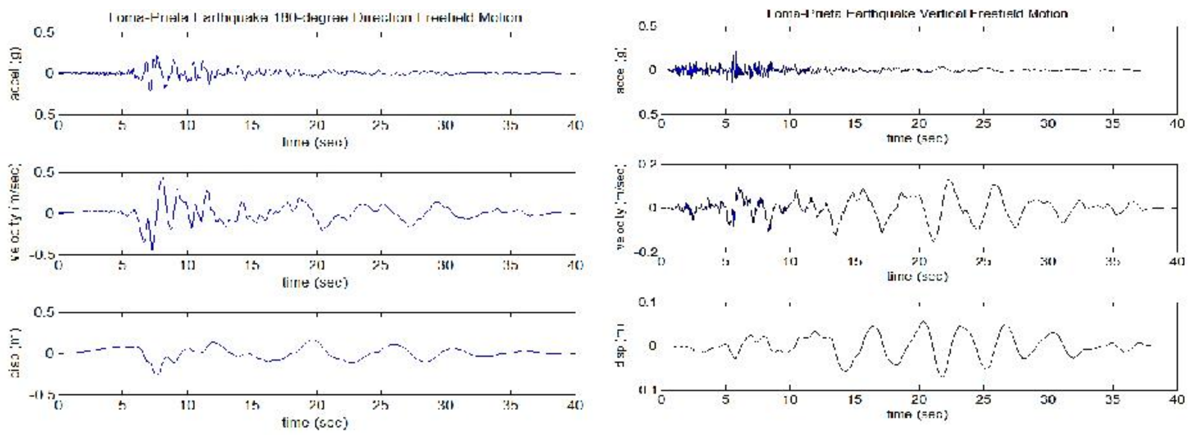

Figure 5. Acceleration, velocity and displacement time histories of free-field NS component of Loma Prieta recorded during the 1989 earthquake, scaled to g

After computing transfer functions, effective input motions and unit impulse responses interaction forces to the near-field far-field interface are determined and the superstructure along with the near-field soil is analysed under these earthquake-induced forces. Figure 6 illustrates peak story displacements along with a comparison with analyses using the UCD model.

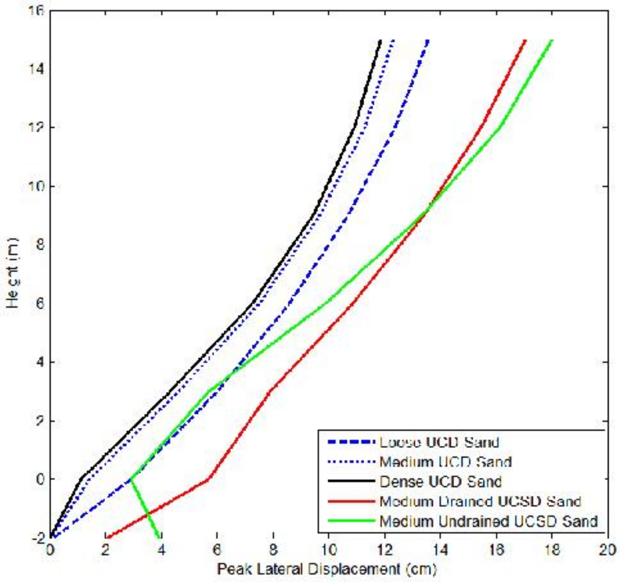

Figure 6. Frame displacement amplitudes, different sand states under the foundation

Displacement time-histories for drained and undrained cases of the underlying sand can be observed on Figure 7.

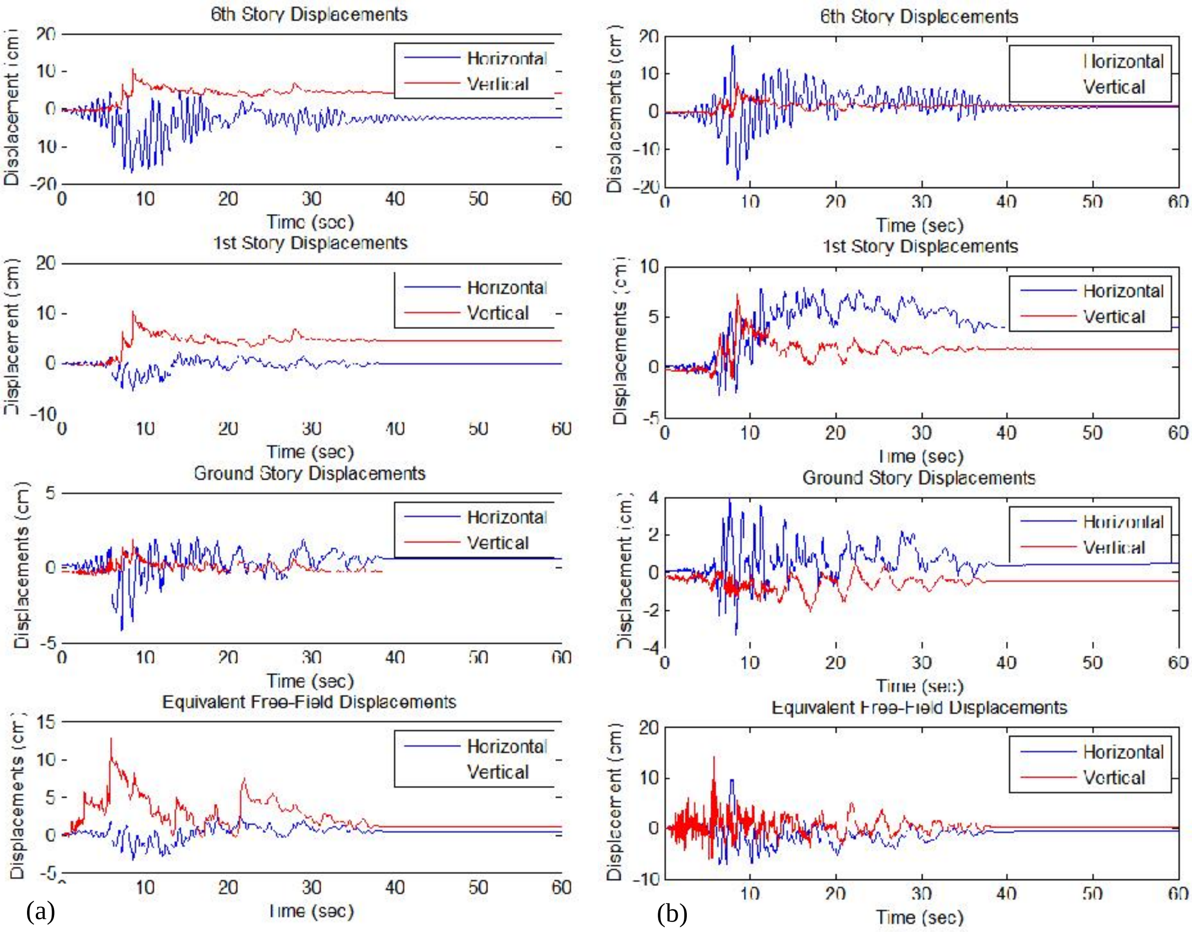

Figure 7. Stories displacement time-histories for inelastic structure, (a) drainage allowed, (b) drainage not allowed

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Base shear time-histories using the two constitutive models for the soil are depicted in Figure 8.

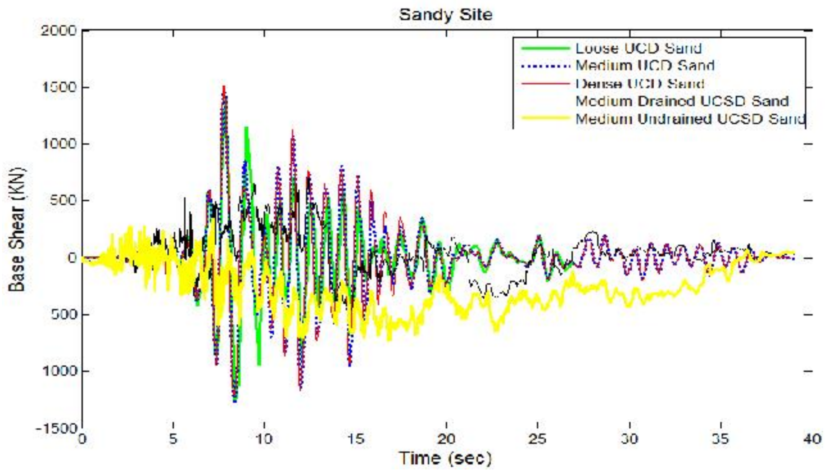

Figure 8. Time history of base shears for different conditions of sand

References should be sorted in alphabetical order with respect to surnames. References should be in English. If several works by the same author are cited, entries should be in chronological order, as the latest one given first. References should be written using 10 point Times New Roman font. Point and comma should not be used following names and surnames, only authors of the same publications should be separated by a comma. Conference and journal titles should be written in italic, book, thesis and reports should be given underlined. Point should not be used within References.

CONCLUSIONS

A reinforced concrete frame was modelled on potentially elastoplastic sandy soil the far-field of which was modelled with the SBFEM method substituted by interaction forces through convolution integrals using the unit impulse response of the medium. Two constitutive models, namely UCD and UCSD, were inserted for modelling the behaviour of the near-field soil, the former for non-linear sub-structuring and the latter for direct modelling of the soil-structure system.

In the drained state modelled by the UCSD model, interaction forces are much less than those of the undrained modelled by the same constitutive model, and the interaction forces time-history is apparently smoother. The near-field soil, modelled by finite elements implemented by the UCSD soil constitutive model, and responses of the structure were captured in the sense of forces and displacement amplitudes. It was observed that considering drainage potential for the sand results in discrepancies of displacement amplitudes of the structure. In case the soil is assumed undrained, displacements of the basemat of the structure may at times be more than those of the bottom stories. Also, drainage potential affects the base shear such that the soil in which the state of being drained/undrained is modelled reveals smaller but more extended-in-time base shear maxima. Results suggest that the denser the soil gets, the smaller the interaction-induced displacements yet the bigger support reactions and base shears will be.

The hybrid FEM-SBFEM approach, which is a novel SSI analysis procedure, was verified with substructure approach, which was in its turn manipulated to capture nonlinear response effects of the underlying soil on the superstructure. Overall results seem close enough for both methods of analysis. Soil density is of high importance and controls SSI effectiveness on final displacements and support reactions. The hybrid method, which was the main focus of this study, is of a few strong points compared to conventional approaches. Firstly, the far-field is logically modelled and hence radiation damping is decently considered. Second, responses are more accurate since modelling analyses are based on more analytically-supported theories and thus more exact. For retrofitting problems here destruction pattern must be considered beside performance levels, the analysis method which yield results closer to reality must be applied since not only the outcome will be based on a more practical model, but it will be clear where more expense and operation concentration should be placed.

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