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MODIFICATION OF GROUND AND STRUCTURAL RESPONSE DUE TO SEISMIC STRUCTURE-SOIL-STRUCTURE INTERACTION

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ABSTRACT

In order to appropriately determine the seismic response of a structure founded on soft soil, it is important to understand the effects of soil structure interaction (SSI) on the vibrational input to the structure. In the near field, the vibrations of structures may modify the free-field ground accelerations and hence the resulting input motion to the structure at foundation level. This is expected to be especially true for urbanized areas where the effects of multiple adjacent structures will be superposed. This paper presents the results of two-dimensional finite element analyses to investigate the effect of both a single structure and multiple adjacent structures, founded on soft soil, in modifying the structural input motion at foundation level and the resulting seismic performance of the superstructure. The structures are modeled as simple single degree of freedom oscillators with dynamic properties representative of low-rise constructions and founded on shallow raft foundations. It is demonstrated that the effects of SSI on soft soil, which increase the magnitude of the accelerations beneath the structure may be improved or worsened with additional surrounding structures. It is further shown that for a symmetrical problem (i.e. identical adjacent structures and symmetrical harmonic ground motion), the co-seismic performance of the superstructure is worsened for the case of adjacent structures, and this effect is stronger as the inter-building spacing is reduced. The post-earthquake (permanent) response of the structures is also shown to be highly asymmetric and non-negligible, meaning that linear elastic soil models are inappropriate for seismic response analysis within heavily urbanized areas.

Keywords: Soil-structure interaction, sand, multiple structures, finite element modeling

INTRODUCTION

It is common practice in seismic design to analyze the response of building structures in isolation from any other structures which may surround them. In many seismic regions of the world however this does not model reality, in which the local population is concentrated in cities, towns and other urbanized areas. In such settlements, building structures will typically be surrounded by other structures, with the inter-building spacing s_e (from the edge of one building to the edge of another) being substantially less than the width of the structures' footprints. In the worst case, if s_e is very small then pounding may occur between adjacent structures causing significantly greater damage than an analysis of the structural response of an isolated structure would suggest. Pounding may be avoided by ensuring that an appropriately-sized gap remains around all structures. Eurocode 8 (2004) suggests that the minimum gap size between two buildings is the square root of the sum of the squares of the maximum storey displacements (inter-storey drifts) of the two structures. The structure will normally be designed to keep the inter-storey drift,

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relatively small such that the columns and non-load bearing elements (e.g. cladding or infill walls) will not be heavily damaged. As a result, the minimum structural gap will allow building structures to be built very close together.

Even if pounding is avoided with a suitably sized gap, there may still be significant coupling of the structural response of the adjacent structures through the ground, which is the common element connecting them. This effect will subsequently be referred to as structure-soil-structure interaction (SSSI). Thus the actual seismic response of the structure within its surrounding urban environment may not be the same as that calculated by an analysis of the isolated structure. In this respect, existing estimates of seismic hazard may not represent the true situation. Furthermore, EC8 and other codal provisions for pounding assume that there is no through-ground coupling (though the soil-structure interaction of the isolated structures may be accounted for), so that they may not provide conservative guidance.

This paper presents 2D Finite Element (FE) analyses investigating the response of adjacent structures to ground motion. Groups of two and three identical low-rise structures with differently sized structural gaps are considered, having raft foundations and founded on loose sand. There are many aspects to the response of such an interconnected structure-soil-structure system; this paper will focus on the effects of the adjacent structures on the ground motion in the near field (specifically, the input motion to each of the foundations), the resulting co-seismic structural response (in terms of inter-storey drift) and the post-earthquake condition of the structures (here represented by the permanent rotation of the foundations and supported superstructure).

NUMERICAL MODELLING

PLAXIS 2D, Version 9.0 is used to undertake the dynamic FE analyses presented herein. Three series of simulations were undertaken, having one, two and three structures respectively, as shown in Figure 1.

Soil

In this paper, a single soil profile was considered for all of the simulations, consisting of a single layer of homogeneous, loose, dry sand H = 20 m deep and 200 m wide, with a unit weight of 17 kN/m^3. The constitutive behavior of this layer was represented in PLAXIS using linear elasticity (V_s = 150 m/s, ν' = 0.3) coupled with Mohr-Coulomb plasticity (ϕ' = 35, c′ = 0.3 kPa, associative flow). These properties represent ground type D (‘soft’ ground) according to EC8 (2004). The small apparent cohesion, c′, was used to prevent numerical instability at the upper surface of the soil where the effective stress is zero. The use of an elasto-plastic material model permits study of structure-soil-structure interaction under strong ground motions where permanent deformations of the ground and/or structure may accumulate. This is a substantial improvement on previous numerical studies of structure-soil-structure interaction (e.g. Tsogka & Wirgin, 2003; Kham et al., 2006; Ghergu and Ionescu, 2009; Padron et al., 2009) whose soil was linear elastic, and therefore focused on small strain dynamic behavior. Rayleigh damping was also modeled with coefficients of α = 1.5 (mass) and $\beta = 4.0 \times 10^{-3}$ (stiffness) to model an equivalent viscous damping ratio of ~8% which was approximately constant over a range of frequencies (± 1% between 1.5 – 6 Hz). The fundamental natural frequency for the layer of soil, f_s = 1.9Hz, was estimated using:

$$f_s = \frac{V_s}{4H} \tag{1}$$

A very fine, unstructured, triangular mesh of 15-noded elements was used to mesh the soil domain as shown in Figure 2. The mesh was not refined locally around the foundations to avoid interfering with wave propagation in these zones. The size of each element was between 1.5 – 3 m; this is acceptable for applied ground motions up to ~6 Hz following the suggested minimum element size of Kuhlmeyer and Lysmer (1973). Absorbent boundaries (PLAXIS Standard Earthquake Boundaries) were applied along the vertical edges of the soil domain to minimize wave reflections and approximate a semi-infinite soil domain in the free-field.

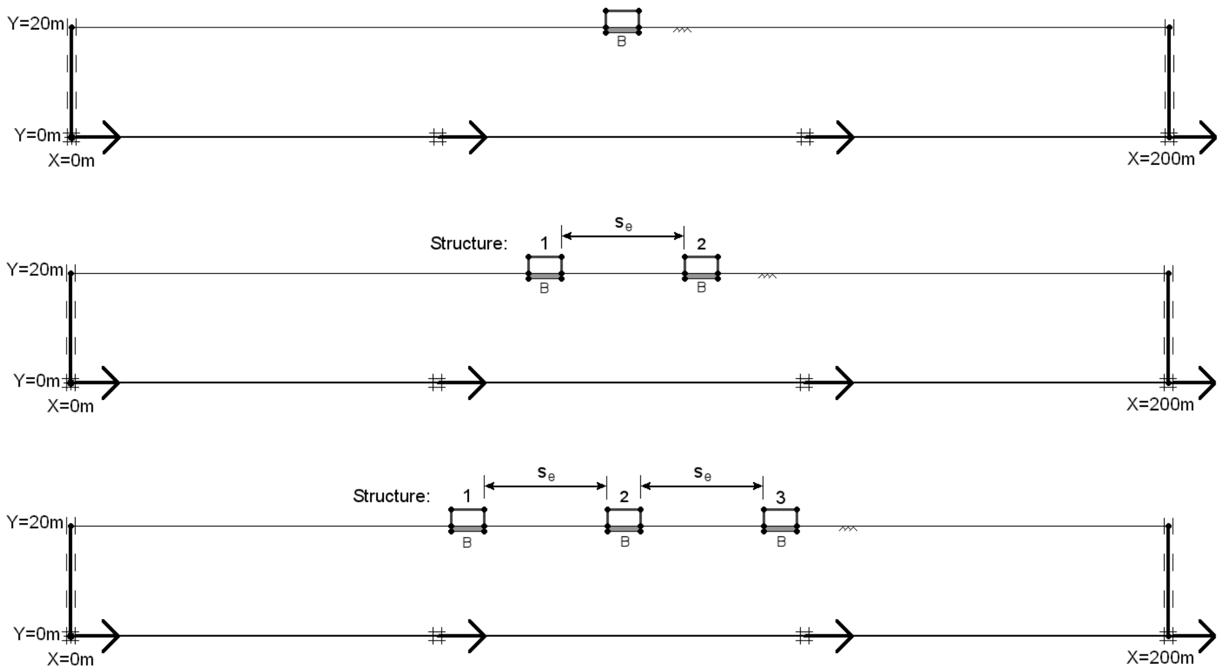

Figure 1. Model description

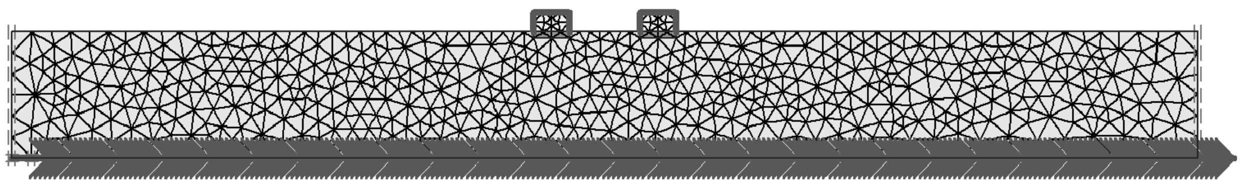

Figure 2. Example finite element mesh (nodes omitted for clarity)

Foundation

The structures are founded on an embedded raft foundation 1 m deep, with the foundation level 1 m below the ground surface. This is modeled as a linear elastic continuum with unit weight of 24 kN/m^3 with Young's modulus E = 3 × 10^{10} kN/m^2 and ν = 0.3 to be representative of a concrete slab. When combined with the weight of the supported superstructure, the applied bearing pressure is ~48 kPa. The use of such a large foundation is not strictly necessary to support such a low bearing pressure and the resulting static safety factor against bearing capacity is of the order of 60. However, stiff rafts are popular in seismic areas and this foundation would be able to sustain shaking of high peak ground acceleration (PGA) without suffering seismic bearing capacity failure (approximately 0.6g would be required to induce this using the method proposed by Soubra, 1999).

Structure

The structural model used was a single storey, single bay steel frame structure in the plane of shaking. As the analyses are all two-dimensional, the structure is infinitely long in the plane perpendicular to this. 2D light linear elastic plate elements of unit thickness were used to model a periodically repeating row of steel Universal Column Sections (UC 254x254x107), h = 3 m high along either side of the structure. The spacing between columns was set at 6 m, both as the width of the frame in the plane of shaking and in the perpendicular direction. This arrangement gave an equivalent bending stiffness of each of the plate elements of EI = 6128 kNm^2/m. Rayleigh damping was ascribed to the plates with $\alpha = 0.4$ and $\beta = 1.0 \times 10^{-3}$ to model an equivalent viscous damping ratio of ~2% which was approximately constant over a range of frequencies (± 0.5% between 1.5 – 6 Hz).

The columns support a lumped mass at the top of the storey which was also modeled by a plate element. The connections of the columns to this mass and the foundations (described below) had full rotation and translational fixity (encastré/fully fixed). The upper plate was given a high bending stiffness (EI = 3×10^{11} kNm^2/m) so as to behave in a rigid manner compared to the columns and was given identical Raleigh damping parameters as the steel column. The weight per unit length of these beams was set at 24 kN/m/m to model the weight of a concrete slab 1 m thick. For the given column fixity conditions, the lateral (sway) stiffness of the frame is given by

$$k = 2\left(\frac{12EI}{h^3}\right) \tag{2}$$

Equation 2 gives k = 5447 kN/m/m. The vibrating mass of the system is 14679 kg/m which gives a fundamental frequency of the building structure of $f_b = 3.1$ Hz.

Ground motion

Harmonic ground shaking of 15 s duration was applied at the bottom of the soil deposit (hereafter termed ‘bedrock level’). Sinusoids of both 0.1g and 0.3g amplitude were applied to all models at this level; for the purposes of this paper only low frequency motion is considered at 1.5 Hz. This is below the natural frequencies of both the soil and the structure(s) and can be expected to place a high displacement demand on the system. The advantages of using harmonic motions in this study are that the resulting behavior of the complex structure-soil-structure system is simplified as shaking can be quantified in terms of the natural frequencies of the individual system elements, easing interpretation. It is clear that future study will be required to extend this work to realistic frequency-rich ground motions.

SITE RESPONSE AND STRUCTURAL INPUT MOTION

Ground accelerations were extracted from the simulations for the soil in the free-field from three depths: 19 m (input motion within soil layer), 10 m (mid-depth of layer) and 0.5 m (ground acceleration at soil surface). These are shown for the two different strength earthquakes in Figure 3. It can be seen that for shaking at 1.5 Hz, which is just below the fundamental frequency of the soil layer, there is significant amplification of shaking as the shear waves propagate to the surface, consistent with behavior in the first mode of vibration. The actual intensity of the ground shaking at the surface (i.e. where the structures are located) is therefore 0.26g and 0.53g for the 0.1g and 0.3g input motions respectively, and therefore represent continuously strong shaking. In the subsequent discussion, the peak value of acceleration at 0.5 m depth (surface) will be termed the peak ground acceleration in the free-field (PGA_{ff}).

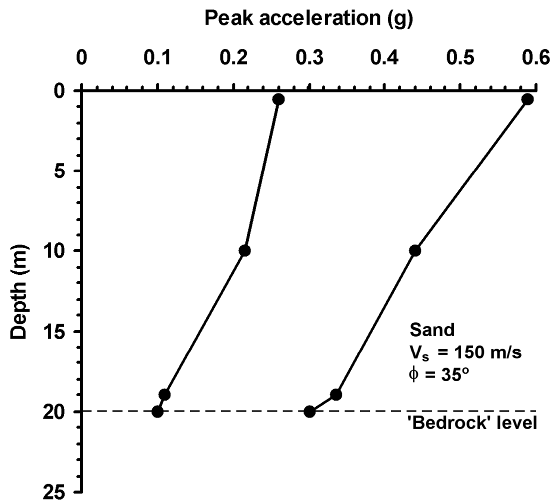

Figure 3. Peak acceleration as a function of depth in the free-field

Further accelerations were extracted from points 0.5 m below the underside of the foundation (i.e. 1.5 m beneath the ground surface) for all of the simulations. These were assumed to represent the input motions to the structures at foundation level. The peak values of these accelerations are henceforth termed PGA_{input}. Figures 4 and 5 compare these values to the free-field values. It can immediately be seen that the presence of even a single structure modifies the accelerations in the near field, increasing PGA by approximately 17%. This is to be expected considering that soil-structure interaction (SSI) is known to be significant for structures on 'soft' soils such as the loose sand tested herein (EC8, 2004).

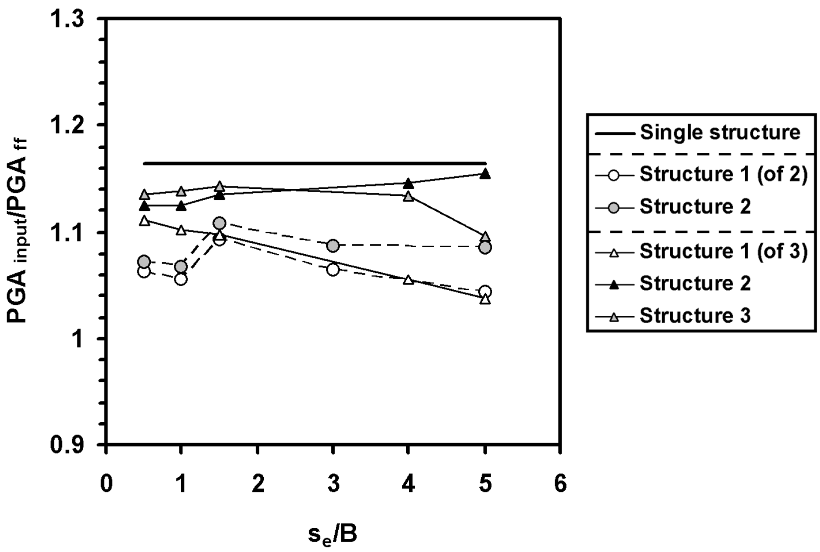

Figure 4. Effect of SSI and SSSI on PGA, 0.1g input motion

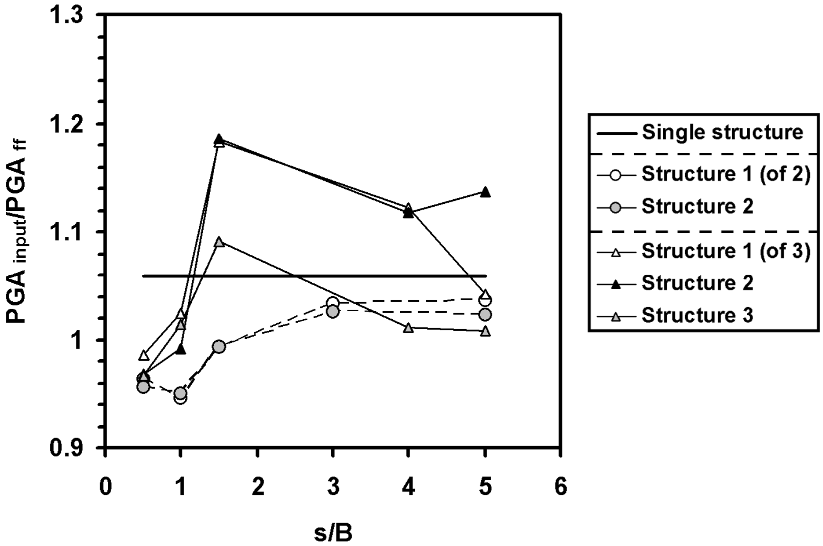

Figure 5. Effect of SSI and SSSI on PGA, 0.3g input motion

When additional structures are incorporated within the model, the behavior becomes more complex and varies with the structural gap s_e. From the preliminary analyses presented in Figures 4 and 5, it appears that the presence of two structures reduces the seismic input to the structures (reduced PGA_{input}); with increasing earthquake strength, values of PGA_{input} at $s_e < 3B$ are similarly reduced compared to the case of a single structure, while at larger spacing, the reduction in PGA_{input} is less apparent compared to a single structure. For the case of three structures, the SSSI is similarly beneficial at close spacing, though the effect is much weaker than with two structures with PGAinput being closer to the single structure values. With increasing earthquake strength, the three structures begin to further amplify the accelarations beneath the structures, over-and-above that due to conventional SSI for a single structure. Although additional data is clearly required to extend these conclusions, it appears that for the light low-rise structures considered in this paper that additional adjacent structures initially have a beneficial effect on the magnitude of accelerations within the near-field, but that as the complexity of the urban environments is increased by adding further structures, there is increasing destructive interference which can lead to further amplification of PGA in the near-field.

CO-SEISMIC STRUCTURAL RESPONSE

Having established how the dynamic input to the structures is altered due to seismic SSSI, the resultant structural dynamic response is now considered. Figure 6 shows time histories of structural response for the single isolated structure, and for the case of two structures at $s_e/B = 0.5$ (closely spaced).

In the remainder of this paper, the co-seismic behavior will be principally described in terms of the inter-storey drift, Δ, as this is an index of the shear force and bending moment demand placed on the columns to avoid structural collapse, and the foundation rotation, θ, which represents the overall tilt of the buildings. Vertical movements of the foundations were observed to be small due to the high bearing capacity described earlier.

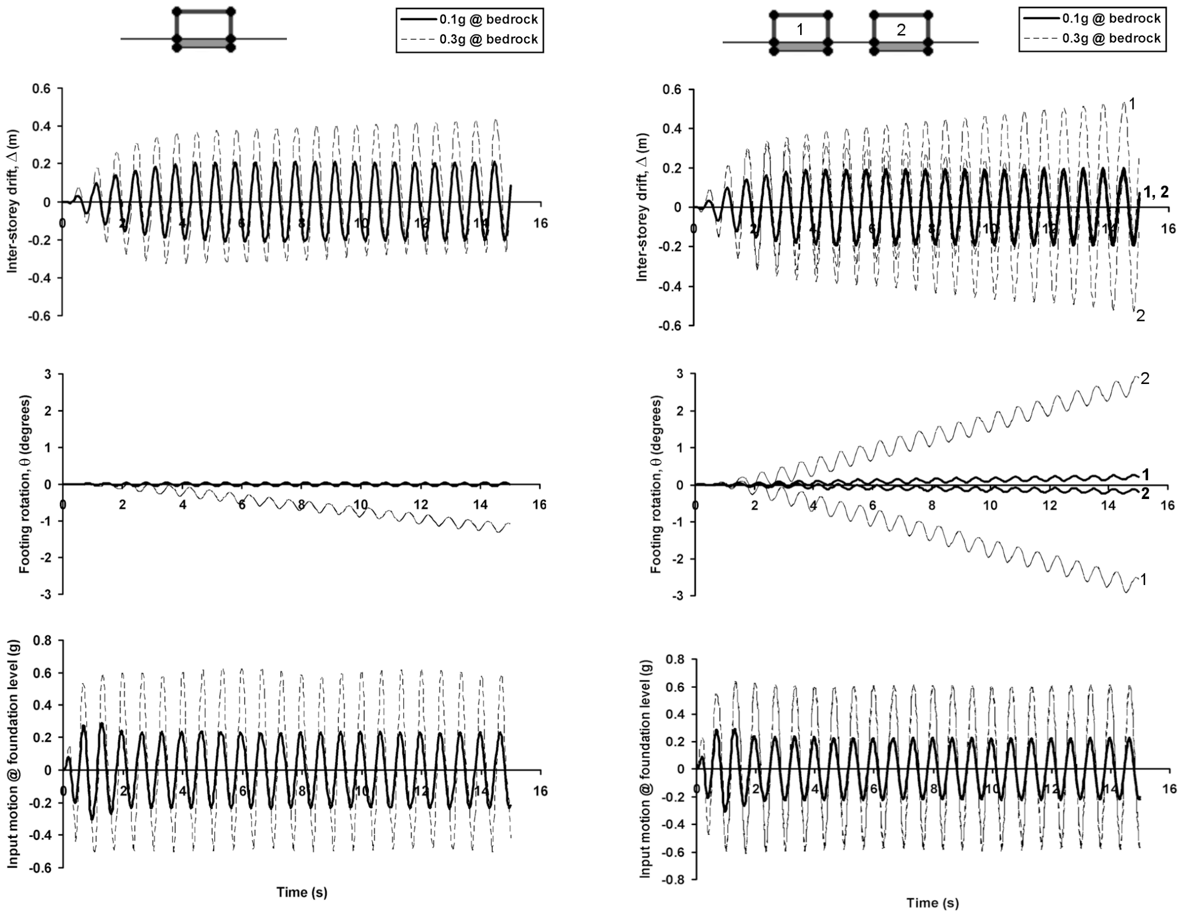

Figure 6. Dynamic structural response of a single structure (left) and two adjacent identical structures at s_e = 0.5B (right)

Considering first the behavior of the isolated structure, Δ and θ both increase with strength of shaking. For the smaller earthquake, there is no permanent rotation, i.e. the soil has responded elastically. In the stronger motion, the structure is also observed to permanently tilt by 1.1°, which is reflected in the drift which also has a permanent component at the end of shaking, increasing the demand on the columns. It is clear then that even for an isolated structure, linear elastic soil models are not appropriate to quantify soil-structure interaction under strong shaking.

For the case of two adjacent structures, permanent rotation is observed to occur even for the smaller earthquake, though the amount is small. In the larger earthquake the rotations are increased compared to the case of the single structure and in both cases, the structures can be seen to rotate in opposite directions. The permanent (non-cyclic) component of the rotations are again evident in a permanent component of inter-storey drift.

As mentioned previously, the inter-storey drift can be considered an index of seismic structural performance as it governs the applied seismic shear forces and bending moments within the columns.

Indeed, in a linearly elastic structure such as the one described here, these quantities are directly proportional. Figure 7 shows how the peak inter-storey drift in the adjacent structures compare to those of a single isolated structure (Δ_{iso}). It will be seen that for low strength shaking, the effect of SSSI is marginally beneficial, with Δ reducing by approximately 5% (2 structures) or 10% (3 structures). As the earthquake strength is increased however, the SSSI increases the drifts for the case of two structures at low s_e/B. In the case of three structures, the central structure (structure 2) is actually improved at s_e/B = 0.5 and 1.0, at the cost of a 40% increase in Δ in the outer structures. It is also noticeable that when there are three structures, the response of the outer structures (1 and 3) is no longer symmetrical between s_e/B = 1 – 3, despite the system itself being symmetrically arranged. This may be due to a combination of the anti-symmetry in the ground motion and soil response in the inelastic range. For both the two and three structure cases, Δ/Δ_{iso} returns to a value of approximately 1.0 at s_e/B = 3, implying that this gap is sufficient for the structures to be behave as if they were isolated (no SSSI).

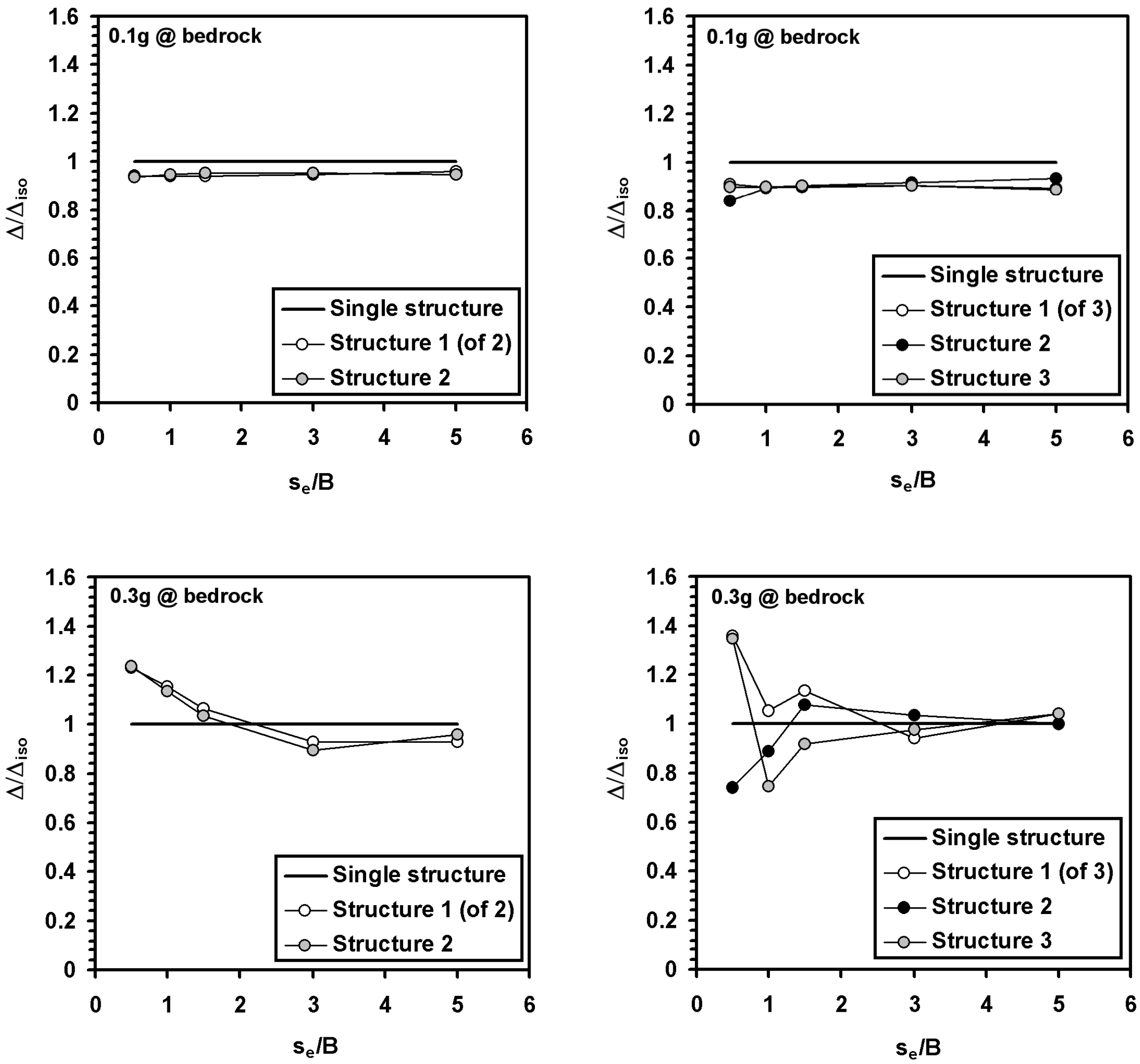

Figure 7. Effect of SSSI on peak inter-storey drifts observed during shaking

POST-EARTHQUAKE STRUCTURAL CONDITION

For the inter-storey drift the cyclic component is much larger than any permanent component due to tilt (Figure 6), such that the critical conditions for this parameter are during the earthquake (co-seismic). The footing rotations however have a strong permanent component and a relatively small cyclic component and are therefore a good indicator of the post-earthquake condition/serviceability of the structures. The absolute final permanent rotations were obtained at 15 s for all of the simulated foundations by filtering out the cyclic component using a low-pass filter. These results, as a function of spacing, are presented in Figure 8. It is immediately obvious from this figure that the permanent rotations are much larger in the stronger earthquake. However, unlike the drift data, the rotations are increased at close spacing for all cases of adjacent structures at all strengths of ground motion. This would suggest that in a dense urbanized area where buildings are closely spaced, post-earthquake serviceability may be substantially lower than would be predicted by a conventional analysis of an isolated structure.

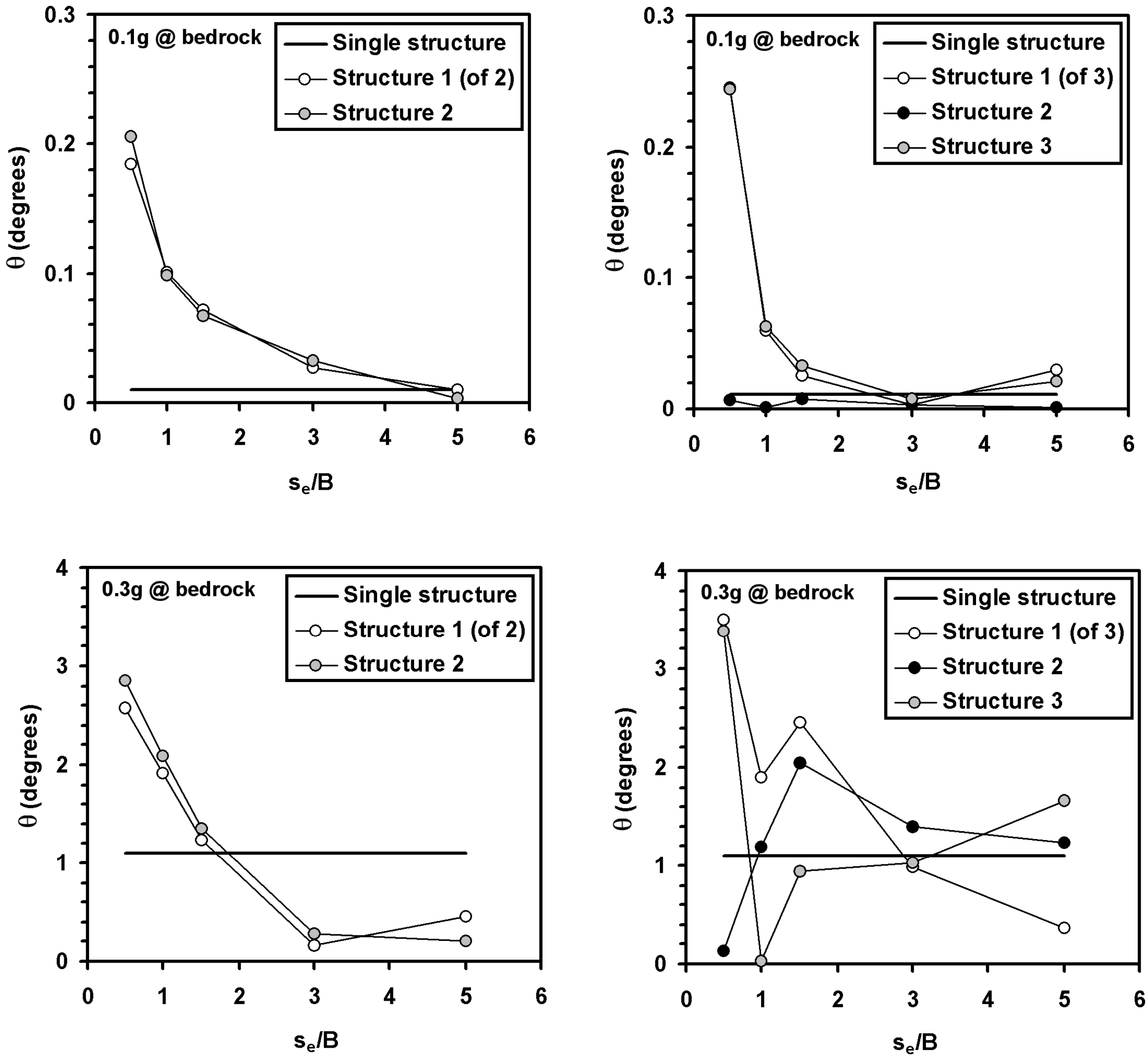

Figure 8. Effect of SSSI on final structural tilt observed post-shaking

Considering the two structure data, although the magnitudes of the rotations were similar at all values of s_e and for both input motions, in all cases the structures rotated in opposite directions as shown previously in Figure 6. For the case of three structures, the response becomes asymmetrical as the earthquake strength is increased (as observed for Δ in Figure 7). This reinforces the need to estimate SSSI effects by simulation for strong earthquakes, which can account for soil inelasticity and the resultant asymmetrical response of the structure-soil-structure system, rather than using simplified models based on linear-elastic soil behavior. Considering that these complexities of behavior have been observed for relatively symmetrical ground motions, simulation by numerical tools will be even more important for real systems of adjacent buildings which may be dissimilar and which will be subjected to frequency-rich asymmetric input motions.

CONCLUSIONS

This paper has presented preliminary data investigating the dynamic response of systems of adjacent structures on 'soft' ground (loose, dry sand) under harmonic ground motions. 2D finite element simulations were conducted using a simple elasto-plastic constitutive relationship to model the mechanical behavior of the soil. The data have shown that the structure-soil-structure interaction has only a small effect on the accelerations in the vicinity of the foundations; despite this however, the co-seismic structural response, as quantified by the inter-storey drift, may be substantially increased at close building-to-building spacing during strong ground motion. It has further been demonstrated that the post-earthquake condition of the structures, as quantified by the permanent tilt (foundation rotation), is always substantially worsened at low spacing due to the presence of adjacent structures. It would appear also that as the system is made increasingly complex (i.e. as additional structures are added) both the co-seismic and post-earthquake responses become increasingly asymmetrical when subjected to very strong ground motion. It should be noted that the work presented herein has been limited to cases with identical single storey (low-rise) structures on raft foundations founded on loose sand under low-frequency harmonic ground motion. Further work is required to examine a wider range of structural properties, foundation designs, building arrangements and soil deposits to provide more complete guidance on the actual seismic hazard associated with highly urbanized areas.

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