EFFECTS OF SUPERSTRUCTURE INERTIA ON LIQUEFACTION SETTLEMENTS OF FOOTINGS

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ABSTRACT

Liquefaction-induced settlements of shallow foundations and associated bearing capacity degradation receive increasing attention in recent literature. However, the bulk of relevant research overlooks the effect of superstructure inertial forces or considers the simplified case of rigid structures. The aim of the present paper is to explore the role of superstructure inertia on the performance of shallow foundations resting on liquefiable sand. To ensure a viable foundation design, it is assumed that a non-liquefiable layer of adequate shearing resistance and thickness is interfered between the foundation and the liquefiable sand layer. The study is based on numerical analyses, performed with the Finite Difference Code FLAC3D, of a Single-Degree-of-Freedom (SDOF) system and a rectangular footing resting on liquefiable soil. The NTUA-Sand constitutive model is used to simulate the liquefiable sand response, while the structure and the foundation are modeled through elastic beam and shell elements respectively. The parametric analyses focus upon the effect of individual SSI parameters, such as the frequency characteristics of the structure - foundation system and that of the free field soil profile, reduced to the predominant frequency of the seismic excitation. Preliminary results show that, although SSI effects are systematic and significant for dry soils, they become minor and not systematic (i.e. settlements may increase or decrease) upon liquefaction. It is noteworthy (and possibly surprising) that, upon resonance, liquefaction-induced settlements are clearly reduced compared to the associated predictions for rigid or mass-less structures.

Keywords: Soil-structure interaction, Liquefaction, Settlements, Shallow foundations, Numerical analysis.

1. INTRODUCTION

Buildings on shallow foundations often experience excessive seismic settlements, often associated with structural damage, which are intensified in the event of liquefaction of the foundation soil. Liquefaction-induced settlements have been well documented in many earthquakes (Niigata, 1964; Luzon, 1990; Kocaeli, 1999; Christchurch, 2011), whereas records of seismic settlements of buildings on dry or not liquefiable soils are less common but also existent (Miyagihen-Oki 1978, San Fernando 1971).
Structure-Foundation-Soil systems accumulate seismic settlements through the following main mechanisms:

- The seismic reduction of the bearing capacity, which is attributed both to the degradation of soil’s strength and to the detrimental contribution of structural and soil inertia to the typical Coulomb-type mechanism of sliding wedges (Richards et al. 1993; Paolucci and Pecker 1997). The particular phenomenon is more severe in the case of liquefaction, since excess pore-pressure build up reduces soil’s strength drastically.

- The soil-structure interaction (SSI)-induced soil softening, since rocking of the foundation during strong vibrations creates significant shear straining and, thus, soil softening under the foundation edges (Dashti et al. 2010a; Knappett et al. 2006). Through this mechanism, building settlements are accumulated every half-cycle, when rocking occurs.

Seismic bearing capacity of shallow foundations on dry soils is typically addressed with an upper limit analysis using a Coulomb-type mechanism that includes soil and foundation inertia forces. With such a procedure, reduced bearing capacity factors are calculated, while foundation settlements are estimated by the Newmark sliding block mechanism (Paolucci and Pecker 1997; Richards et al. 1993). Earthquake accelerations are introduced as pseudo-static loads at the soil wedge and foundation level. In more recent studies, superstructure effects are indirectly introduced, by means of moments acting upon the foundation. According to Knappett et al. (2006), their inclusion can have a detrimental effect on the performance of the foundation. In any case, given that seismic loading is introduced pseudo-statically, dynamic effects such as the synchronization of the applied “superstructural” moment with the soil and foundation inertia are not considered.

The performance of structures on shallow foundations upon liquefiable sand layers has been investigated in recent studies, both experimentally (Dashti et al. 2010b; Liu and Dobry 1997) and numerically (Byrne et al. 2004; Dashti and Bray 2013; Karamitros et al. 2013a; Lopez-Caballero and Modaressi Farahmand-Razavi 2008). Analytical solutions for the bearing capacity of such foundations are provided for two-layered soil profiles, i.e. non-liquefiable crust over liquefiable sand layer, (Karamitros et al. 2013b). The mechanisms of seismic settlements accumulation have been also investigated (Dashti et al. 2010a; Karamitros et al. 2013a) and analytical relations have been recently proposed for preliminary computations (Dimitriadi et al. 2017). As in the case of dry soils, these studies focus mainly on geotechnical aspects and do not provide adequate insight to the effect of superstructures on seismic settlements.

In view of the above, the current study aims to address systematically the effect of superstructure characteristics on the seismic settlement accumulation of shallow foundations both on liquefiable as well as dry soils by employing typical soil-structure interaction (SSI) parameters. First, the paper describes the procedure used in order to identify the characteristics of the Structure-Foundation-Soil systems. In the sequel, two SSI cases of typical system on both dry and liquefiable soil are examined in order to gain insight into the mechanisms of bearing capacity failure and settlement accumulation. Finally, the paper concludes with a parametric investigation, aiming to assess the quantitative effect of the most critical SSI parameters on the accumulation of seismic settlements.
2. METHODOLOGY OUTLINE

2.1 System identification

In order to isolate superstructure inertial effects, numerical analyses are performed in three dimensions for Structure-Foundation systems (hereafter referred as SFS), where the superstructure is an idealized Single-Degree-of-Freedom (SDOF) system, and simpler Foundation-Soil systems (hereafter referred as FS) where there is no superstructure. The equivalent FS systems has the same soil properties as the SFS systems, while the foundation contact pressure corresponds to the total weight of the SFS system. Parametric analyses are performed considering the effects of the following key SSI parameters, defined in Table 1: i) Structure over excitation period ratio \( T_s/T_e \), ii) slenderness ratio \( h/r \) and iii) relative mass ratio \( \gamma \). The range of the above parameters, listed in Table 1, were selected so as to capture the governing mechanisms of seismic settlement and also to adhere to common engineering practice.

Table 1. Range of SSI parameters.

<table>
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<tr>
<th>SSI Parameter</th>
<th>Range</th>
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<tr>
<td>Frequency ratio</td>
<td>( \frac{T_s}{T_e} ) 0.40-2.20</td>
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<tr>
<td>Relative mass ratio</td>
<td>( \gamma = \frac{m}{\pi \cdot \rho_s \cdot h \cdot r^2} ) 1.24-2.04</td>
</tr>
<tr>
<td>Slenderness ratio</td>
<td>( \frac{h}{r} ) 1.04-1.88</td>
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*\( m \): structural mass, \( h \): structural height, \( r \): equivalent radius of footing, \( T_s \): natural period of equivalent SFS system, \( T_e \): period of excitation.

Estimation of the above structure and soil parameters is straightforward, with the exception of the equivalent Natural Period of the SFS system \( T_s \). In principle, existing analytical procedures (Veletsos, A. S., & Meek 1974) can be employed for that purpose. However, the severe shear modulus degradation that occurs during liquefaction as well as the importance of the evolution of the system’s properties during the seismic event deem the traditional SSI analytical procedures insufficient for the present study. Hence, \( T_s \) is evaluated herein by examining the transfer functions of output-to-input motion in the frequency domain. In particular, the frequency spectrum of the motion at the mass level \( u_{mass} \) (output) is divided by that of the free-field ground motion \( u_g \) (input) to provide the eigenfrequencies of the SFS system, taking into account all flexibility components (structural, foundation rocking and foundation translation) of the system.
The process of identifying the eigenfrequencies of a SFS system is demonstrated in Figure 1 for the case where the base of the model is excited by the Kobe (1995) seismic motion recording (Kakogawa CUE90) (Figure 1a) as well as by a harmonic motion (Figure 1b). Results are presented both for the case of dry and liquefiable soil (gray and black lines respectively). It is shown that the identification of the eigenfrequencies of a SFS system is greatly dependent on the frequency content of the excitation. Namely, signals with rich frequency content, like the Kobe (1995) seismic recording, are definitely more suitable as they can provide natural period estimates with greater accuracy. In the case of Figure 1, the use of Kobe (1995) earthquake recording yields a predominant equivalent natural period of the SFS system which is equal to $T_s=0.45$ sec for the liquefiable case and $T_s=0.55$ sec for the dry case. This counter-intuitive finding will be addressed in the following.

2.2 Numerical Model

All numerical analyses are performed using the finite-difference-code $FLAC^{3D}$ (Itasca 2012). The particular codes utilizes an explicit integration algorithm, which is more efficient for nonlinear problems such as soil liquefaction, and enables coupling between pore water flow and dynamic loading. The 3D numerical model of the SFS system is illustrated in Figure 2. It consists of an idealized Single-Degree-of-Freedom (SDOF) superstructure and a square foundation, resting upon a uniform layer of liquefiable Nevada sand. A thin crust of improved (densified) sand has been created at the top of the liquefied sand layer. A sinusoidal seismic excitation is applied at the base of the model. The “tied node” method is used to simulate free field lateral boundaries, which imposes equal horizontal and vertical displacements at opposite boundary nodes of the same elevation.

The constitutive model used in this study is the NTUA-SAND critical state model, developed at the National Technical University of Athens. This model captures soil nonlinearity in both monotonic and cyclic loading. It utilizes three cone-type surfaces in space: dilatancy, critical state and bounding surface that are related to the state parameter $\psi$. For small shear strains, it adopts a Ramberg & Osgood hysteretic formulation, which accounts for shear modulus degradation and hysteretic damping increase. Pore pressure build-up is related to the directional effect of fabric evolution via an empirical factor. The accuracy of the constitutive model has been verified against a centrifuge experiment of the VELACS research project, which studies the seismic performance of shallow footings on liquefiable sand (Karamitros 2010).
The current study is based on a numerical model that serves as the reference case, followed by 95 parametric analyses with varied characteristics. The reference model assumes a superstructure with fixed-base natural period $T_{str\text{-}fix}=0.35$ sec, a 4m×4m square footing and sand relative density $D_r=60\%$ for the improved crust and $D_r=45\%$ for the underlying liquefiable native soil. The parametric analyses are performed for excitation frequencies $T_e=0.25$-$2.00$ sec, column height $h=3.0$-$5.1$ m and structural mass $m=54.4$-$114.4$ tn. Finally, note that the analyses for dry soil conditions were performed using the buoyant unit weight so that the effective consolidation stresses are directly comparable to those of the saturated (liquefiable) subsoil.

3. SSI EFFECTS FOR DRY SAND

As mentioned in the introduction, pseudo-static consideration -in upper limit analyses- of structural inertia forces alone (Budhu and Al-Karni 1993; Richards et al. 1993) or combined with the corresponding overturning moments (Paolucci and Pecker 1997; Soubra 1999), has resulted in reduced bearing capacity of the foundation. Additionally, the experimental study of Knappett et al. 2006 on the dynamic performance of a rigid block, has identified rocking as the dominant failure mode. In extend of these observations, it may be indirectly concluded that structural inertial forces affect seismic settlements of buildings on dry soils by two main mechanisms: i) they contribute as shear force acting upon the foundation to the soil inertial force that drives the wedges of the Coulomb-type mechanism to slide and ii) they control the magnitude of overturning moments at the foundation level, which cause additional shearing of the foundation soil.

This chapter examines the contribution of both settlement mechanisms, i.e. bearing capacity degradation and rocking, for SFS and FS systems on dry soils, for a wide range of frequency ratios $T_s/T_e$. Figure 3 compares various response aspects of the SFS and the FS system for SSI parameters $T_s/T_e=1.10$, $\gamma=2.00$, $h/r=1.04$, i.e. a system with low slenderness, average mass ratio and close to resonance with the seismic excitation. In all figures, the response of the SFS and FS system is shown with continuous and dashed lines respectively.

In more detail, Figure 3a compares the accumulation of settlements with time showing that the SFS system settles considerably more during the shaking. Figure 3b compares the horizontal oscillation at the mass of the structure (only for the SFS system) to that at the foundation level (for the SF and the
SFS systems). The structural mass of the SFS system oscillates horizontally with phase lag relatively to the horizontal oscillation of the foundation (compare gray and black continuous lines). This asynchronous movement of the structural mass restrains the movement of the SFS foundation and decreases its amplitude compared to the amplitude of the FS system. Base rotation (Figure 3c) is, as expected, only present in the SFS system. This results to significant cyclic deviatoric stresses under the footing edge for the SFS system and negligible ones for the FS system (Figure 3e).

The higher settlement accumulation of the SFS system is more clearly explained in Figure 3d and 3f, which shows the evolution of deviatoric strains and settlement rate with time: Firstly, in the SFS system, deviatoric strains under the right edge of the foundation exhibit significant spikes caused by the clockwise rocking of the foundation. These spikes coincide with maximum settlement rates during the first 2.5 sec of motion. Therefore, during this time-period, settlement rates peak twice per cycle, considering, also, deviatoric straining at the left edge of the foundation. After the first 2.5 sec, settlement rates reduce in value and they peak four times per cycle of motion, which means that a more generalized soil failure is responsible for settlement accumulation. Secondly, in the case of the FS system, deviatoric strains develop smoothly and are significantly smaller both in rate and in permanent value.

Concluding, the vast difference between deviatoric strains under the footing edge in the SFS and the FS systems indicates that rocking-induced settlement accumulation is the governing mechanism in the case of the SFS system. Only a smaller portion of the excessive settlements can be attributed to wedge sliding due to the degradation of bearing capacity.

**Figure 3:** Response time-histories of the SFS and FS system (dry subsoil) in terms of: (a) footing settlements, (b) horizontal displacements, x and (c) rotation; (d) moment-rotation of the foundation; (e) deviatoric stresses and (f) deviatoric strains under the footing edge.
Since inertial forces that develop at the structural mass govern foundation rocking, the characteristics of structural vibration are crucial. Two key vibrational parameters are examined in Figure 4a:

i) The normalized absolute acceleration of the structural mass $S_a/a_g$, which determines directly the magnitude of inertial forces.

ii) The phase angle $\phi$ between the vibration of the mass and the vibration of the free field, which is measured as:

$$\phi = 2\pi \frac{\Delta t}{T_e} = 2\pi \left[ \frac{t_{\text{max}}^{\text{mass}} - t_{\text{max}}^{\text{free field}}}{T_e} \right]$$

The phase angle determines, essentially, the synchronization between structural inertial force and movement of soil’s surface. For small values of phase angles the inertial force is in-phase ($\phi<\pi/2$) with soil movement and therefore it aggravates phenomena of soil wedge sliding, since it is concurrent with soil inertial force. For large values of phase angles the inertial force is out-of-phase ($\phi>\pi/2$) with soil movement and therefore it acts as an inhibiting factor in soil wedge sliding.

The following can be observed from the presentation in Figures 4 and 5:

- **Rigid** SFS systems ($T_s/T_e \to 0$) settle more than FS systems ($\rho_{\text{SFS}}/\rho_{\text{FS}} \approx 1.5$). This is because, although spectral accelerations are not amplified ($S_a/a_g \to 1.0$), they act in-phase with the foundation movement ($\phi \approx 0^\circ$) and consequently foundation rocking and the associated seismic settlements are aggravated.

- **Flexible** SFS systems ($T_s/T_e \to \infty$) settle equally to the FS systems ($\rho_{\text{SFS}}/\rho_{\text{FS}} \approx 1.0$). This is because spectral accelerations are de-amplified ($S_a/a_g \leq 1.0$) and they also act out-of-phase ($\phi \to 180^\circ$) with the foundation movement. Thus, although rocking is still evident, it occurs out-of-phase with foundation and soil wedge sliding and thus counteracts rocking-induced settlements.

- **At resonance** ($T_s/T_e \approx 0.50-1.50$) SFS systems settle considerably more than FS systems ($\rho_{\text{SFS}}/\rho_{\text{FS}} \approx 4.0$). This is because spectral accelerations are considerably increased ($S_a/a_g \approx 2-2.5$), while the phase lag between mass inertia and foundation displacement is average. Thus, the large inertial forces make rocking the governing mechanism of settlement accumulation, with relatively minor counteracting effects due to the asynchronous mass versus foundation vibration.

![Figure 4](image.png)

Figure 4: Response spectra of a) normalized absolute accelerations of structural mass, b) seismic settlement of the FS system to the one of the SFS system.
Figure 5: Schematic representation of wedge formation in the soil and phase difference between structural and soil inertial forces for three characteristic cases, i.e. a) rigid structure: concurrent inertial forces, b) structure close to resonance: ≈ quarter cycle difference between inertial forces, c) flexible structure: opposite inertial forces.

4. SSI EFFECTS FOR LIQUEFIABLE SAND

In the case of liquefiable soil, experimental (Adalier et al. 2003, Liu and Dobry 1997, Dashti et al. 2010b), as well as, numerical (Elgamal et al. 2005; Karamitros et al. 2013a; Karimi and Dashti 2016) studies have shown that building settlements accumulate mainly during the shaking period and they are attributed to deviatoric strains caused by the foundation contact pressure and developing soil and structure inertial forces. Pre- and Post-shaking volumetric strains, due to excess water pore pressure built up and subsequent dissipation account only for a small portion of the accumulated seismic settlements. The aforementioned deviatoric-type mechanism of settlement accumulation is affected by the presence of liquefied soil layers, since they prohibit shear wave propagation to the surface, creating a natural seismic isolation effect (Karamitros et al. 2013a). Thus, seismic ground accelerations and spectral accelerations are significantly reduced after the onset of liquefaction. Consequently, rocking-induced settlements are de-amplified in comparison to the dry soil case, examined above. On the contrary, the severe friction angle degradation caused by the excess pore-pressure generation, amplifies the settlements associated with bearing capacity degradation in the subsoil.
Figure 6: Response timehistories of the SFS and FS system (liquefiable sublayer) and the underlying soil under the right edge of the footing (at depth $z=2.5$ m) in terms of: a) settlements, b) horizontal displacements, c) foundation rotation, d) excess pore pressures, e) moment-rotation of the foundation, f) deviatoric stresses.

Figure 7: Horizontal displacement contour at $t=1.30$ sec and velocity vectors corresponding to bearing capacity failure mechanism.

To verify the above, Figure 6 examines the response of the SFS and FS systems, using a similar format as in previous Figure 3 for dry subsoil conditions. In this case, both systems rest on a natural deposit of liquefiable sand, covered by an improved non-liquefiable soil crust. The examined SSI parameters are the same as for dry soil conditions (i.e. $T_s/T_e=0.90$ $\gamma=2.00$ $h/r=1.04$), representative of nearly resonance conditions for a low slenderness system with average relative mass ratio.

Figure 6a compares the settlement accumulation with time for an SFS and an equivalent FS system. Contrary to the case of dry soil, the final settlements of the SFS system are about 30% smaller than those for the FS system. Following the greater immersion of the FS system footing into the liquefied soil, its corresponding deviatoric strains are also greater (Figure 6f). Note that, following the onset of liquefaction at about $t=2.6$ sec, the liquefied soil deposit acts as a natural seismic isolator and therefore
horizontal vibrations of the foundation are de-amplified (Figure 6b). For this reason, settlement accumulation in both models (SFS and FS) will be compared separately for the following two time windows: i) before the onset of liquefaction (i.e. \( t<2.6\) sec) and ii) after it (i.e. \( t>2.6\) sec). Observe that, during the first time window, the FS system accumulates settlements faster than the SFS system, while settlements are accumulated with the same – and in both cases significantly lower than the initial– rate during the second period (Figure 6d). Similar to what had been observed for dry soil conditions, base rotation is only present in the SFS system (Figure 6c). The magnitude of rotations is reduced by about 50% relative to the case of dry soil conditions, as excess pore pressure build up and liquefaction soften the foundation soil and de-amplify the seismic ground motion.

The de-amplification of structural vibrations results in reduced deviatoric stresses under the footing edges for the SFS system, which now become comparable to the deviatoric stresses of the FS system (Figure 6e). Consequently, rocking-induced soil failure does not aggravate the performance of the SFS system. Thus, the severe bearing capacity degradation due to liquefaction becomes now the governing mechanism of settlement accumulation. The phase lag between the structure and the foundation vibrations counteracts soil inertia forces (and failure wedge sliding), explaining the less overall settlement of the SFS system.

The influence of structural inertial characteristics to seismic settlements is investigated with the aid of Figures 8a and b, following the same framework as the one employed in the previous chapter. Figure 8a shows the effect of Period ratio \( T/T_e \) on i) the normalized absolute accelerations \( S/a_g \) at the structural level and ii) the phase angle \( \phi \) between the vibration of the mass and the vibration of the free field. Figure 8b shows the influence of Period ratio \( T/T_e \), Slenderness ratio \( h/r \), and Relative mass ratio \( \gamma \) on settlement ratios \( \rho_{SFS}/\rho_{FS} \). At first, the following observations can be made with regard to the reference case, i.e. \( h/r=1.04 \) and \( \gamma=2.04 \):

- **Rigid** SFS systems \( (T/T_e =0) \) settle equally with FS systems \( (\rho_{SFS}/\rho_{FS} \approx 1.0) \). This observation verifies that rocking-induced settlements are absent, so that SFS systems do not develop additional deviatoric stresses under their footing edges. In addition, as the structure moves as a rigid block, inertial forces at the foundation base are equal for both systems \( (S/a_g \rightarrow1.0) \) and act in phase with ground displacements, thus explaining why \( \rho_{SFS}/\rho_{FS} \approx 1.0 \).

- **Flexible** SFS systems \( (T/T_e > 1.40) \) settle approximately. 10% less than FS systems \( (\rho_{SFS}/\rho_{FS} \approx 0.9) \). This is because structural inertial forces, even though they are small \( (S/a_g \leq1.0) \), act out-of-phase with foundation displacements, thus inhibiting the failure wedge formation and sliding below the foundation.

- **At resonance** \( (T/T_e \approx 1.00) \), the reduction of SFS becomes maximum \( (\rho_{SFS}/\rho_{FS} \approx 0.75) \). This is because mass acceleration –and inertial forces– are amplified \( (S/a_g \approx 2) \) while they also experience considerable phase lag \( (\phi \approx 90\ deg) \) relative to the foundation vibration. As a result, structural inertial force inhibits wedge sliding to the greatest extent.

The effect of Relative mass ratio \( \gamma \) on settlement ratios is observed in Figure 8, for two levels of \( \gamma \), i.e. \( \gamma=1.24 \) (light blue line) and \( \gamma=2.04 \) (reference case–black line), with all other parameters kept constant. It is seen that for smaller values of relative mass density, structural mass and, therefore, structural inertial force is not significant enough in order to inhibit wedge sliding in the underlying soil. The effect of Slenderness ratio \( h/r \) is also observed in Figure 8, for \( h/r=1.04 \) (reference case–black line) and \( h/r=1.88 \) (orange line). In general, slenderness ratios do not affect settlement ratios, indicating that the magnitude of inertial force is more crucial than the magnitude of the applied moment on the foundation.
5. CONCLUSIONS

The aim of this study was to explore the effects of soil-structure interaction on seismic building settlements for dry as well as liquefiable subsoil. For this purpose, representative cases of SFS systems on both types of subsoil are examined comparatively with FS systems in order to assess qualitatively as well as quantitatively soil-structure interaction effects. The seismic response of the SFS and the FS systems was analyzed with the aid of parametric numerical analyses, focusing on the main (soil, excitation, structure) parameters. In summary, it is concluded that:

(a) Soil-structure interaction effects proved significant for dry soils conditions, leading to increased seismic settlements. In that case, seismic settlements are mainly controlled by rocking-induced shear stresses and strains in the foundation soil. For a given seismic excitation, the vibration characteristics of the structure determine spectral accelerations, thus controlling the magnitude of foundation rocking and the associated seismic settlements. The maximum increase in seismic settlements is anticipated at structure-excitation resonance conditions.

(b) On the other hand, soil-structure interaction effects were minor for liquefiable soil conditions, leading even to somewhat reduced seismic settlements. This is mainly because soil liquefaction attenuates seismic ground and structure (spectral) accelerations, thus de-activating the foundation rocking mechanism, which was identified under dry soil conditions. Furthermore, out-of-phase structural inertial force inhibits wedge sliding in the subsoil so that settlements of the SFS system are reduced relative to those of the plain FS system.

(c) Except from the period ratio $T_s/T_e$, the slenderness ratio $h/r$ and the mass ratio $\gamma$ of the structure seem also to contribute to soil-structure interaction effects, and deserve a more thorough investigation.

7. REFERENCES


