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VARIATION OF MODAL PARAMETERS OF PILE-SUPPORTED STRUCTURES IN SEISMICALLY LIQUEFIABLE SOILS

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ABSTRACT

This paper presents experimental results that aimed to investigate the effects of soil liquefaction on the modal parameters (i.e. frequency and damping ratio) of pile-supported structures. The tests were carried out using the shaking table facility of the Bristol Laboratory for Advanced Dynamics Engineering (BLADE) at the University of Bristol (UK) whereby four pile-supported structures (two single piles and two pile groups) with and without superstructure mass were tested. The experimental investigation aimed to monitor the variation in natural frequency and damping of the four physical models at different degrees of excess pore water pressure generation and in full-liquefaction condition.

The experimental results showed that the natural frequency of pile-supported structures may decrease considerably owing to the loss of lateral support offered by the soil to the pile. On the other hand, the damping ratio of structure may increase to values in excess of 20%. These findings have important design consequences: (a) for low-period structures, substantial reduction of spectral acceleration is expected; (b) during and after liquefaction, the response of the system may be dictated by the interactions of multiple loadings, that is, horizontal, axial and overturning moment, which were negligible prior to liquefaction; and (c) with the onset of liquefaction due to increased flexibility of pile-supported structure, larger spectral displacement may be expected, which in turn may enhance P-delta effects and consequently amplification of overturning moment. Practical implications for pile design are discussed.

INTRODUCTION

Multi-storey buildings and bridges built on loose to medium dense sands are often supported on pile foundations. During earthquakes, if these sandy soils are saturated, they tend to develop excess pore water pressure, which in extreme cases may lead to the so-called liquefaction condition. Over the past years, the seismic design of pile-supported structures in liquefiable soils has been a constant source of attention to the earthquake engineering community. Many seismic design codes advise practising engineers to design pile foundations against bending due to inertia and kinematic forces induced by the deformation of the surrounding soil. In the presence of liquefaction phenomena, Eurocode 8 (EN 1998-5:2004, 2004) recommends that *"the side resistance of soil layers that are susceptible to liquefaction or to substantial strength degradation shall be ignored".* Similarly to the Eurocode 8, the Japanese Highway Code of practice JRA (JRA, 2002) suggests to design pile-supported structures considering two different loading conditions comprising: (a) kinematic loading exerted by the lateral pressure of the liquefied layer and any non-liquefied crust resting on the top of the liquefied deposit; (b) inertial force due to the oscillation of the superstructure. The code recommends engineers to check against bending failure considering inertia and kinematic forces separately.

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Although current design codes commonly use high factors of safety against both gravity and lateral loads, collapses of pile-supported structures induced by soil liquefation have been observed also in recent years (Yoshida and Hamada, 1990; Tokimatsu and Asaka, 1998; Bhattacharya et al., 2011a). The collapse of buildings designed employing large factors of safety against conventional loading conditions (i.e. inertia and kinematic loads) suggests that the actual mechanism of failure of pile-supported structures during liquefaction phenomena is not fully understood.

According to many building codes, the seismic demand can be conveniently represented by the elastic response spectrum. The assessment of the spectrum ordinate requires an accurate estimation of both natural period and damping ratio of the structure. The majority of the seismic codes recommend to compute the vibration period based on empirical formulae, which only consider the dimensions, type and material of the superstructure, whereas the foundation is considered to be rigid. For example, to assess the fundamental period, T_1, of the structure, the Eurocode 8 (EN 1998-1:2004, 2004) recommends equation (1)

$$T_1 = C_t H^{3/4} \tag{1}$$

Where T_1 is in seconds, H is the height of the building in m, the latter must be measured from the foundation or from the top of a rigid basement; C_t is equal to 0.085 for moment resistant space steel frames, 0.075 for moment resistant space concrete frames and for eccentrically braced steel frames, and 0.050 for all other structures. Equation 1 is quite similar to the well-known formula $T_1=0.1\times n$, where n is the number of stories of the building.

It can be noted that by using empirical equations such as the one given by equation 1, the period of vibration is estimated based only the characteristics of the superstructure but the foundation's flexibility is completely neglected. Although such an assumption may be conservative due to the beneficial effects of the SSI, i.e. de-amplification of spectral accelerations due to increase of damping and lengthening of the period, several studies (Goel and Chopra, 1998; Mylonakis and Gazetas, 2000; Khalil et al., 2007; Crowley and Pinho 2010) demonstrated that the effects induced by Soil-Structure Interaction (SSI), especially in the presence of liquefaction, can be un-conservative and led to higher spectral acceleration and displacements.

This paper presents results obtained from an experimental investigation which aimed to assess the effects of soil liquefaction on the natural frequency (or fundamental period) and damping ratio of typical pile-supported structures. The tests were carried out using the shaking table facility of the Bristol Laboratory for Advanced Dynamics Engineering (BLADE) at the University of Bristol (UK). The tests aimed to monitor the variation in natural frequency and damping of the four physical models at different degrees of excess pore water pressure generation and in full-liquefaction condition.

EXPERIMENTAL INVESTIGATION

The experimental investigation presented in this paper was conducted on the shaking table facility of the Bristol Laboratory for Advanced Dynamics Engineering (BLADE) at the University of Bristol (UK). The shaking table comprised of a 3×3 m cast aluminium platform driven by eight servo hydraulic actuators that allowed a full control of motion in all six-degrees of freedom. Each actuator had a dynamic capacity of 70 kN and maximum stroke of 300 mm, which permitted a maximum acceleration of 1.6 g and 1.2g in the horizontal and vertical direction respectively (considering a payload of 10 tonnes). The operational frequency range of the shaking table ranged from about 0 to 100 Hz. The soil container, shown in Figure 1, consisted of a rigid box with external dimensions of 2.4 m length, 2.4 m height and 1.2 m width. The soil container was secured to the shaking table by bolting the bottom plate of the container to the aluminium platform by using a set of steel wedges. The main limitations of using rigid containers were represented by the generation and reflection of P-waves from the end walls. In a typical soil layer, which may be idealised as semi-infinite extended deposit, the energy associated with the wave propagation diminishes gradually with distance. This dissipation may be related to the combined presence of hysteretic and radiation damping as well as the fact that energy is spreading to a larger volume of soil, the so-called geometric attenuation. To minimise the reflection

of P waves from the end-walls as well as to aid the dissipation of energy, sheets of absorbing materials were placed on both end-walls of the container (Bhattacharya et al., 2011b; Bhattacharya et al., 2012).

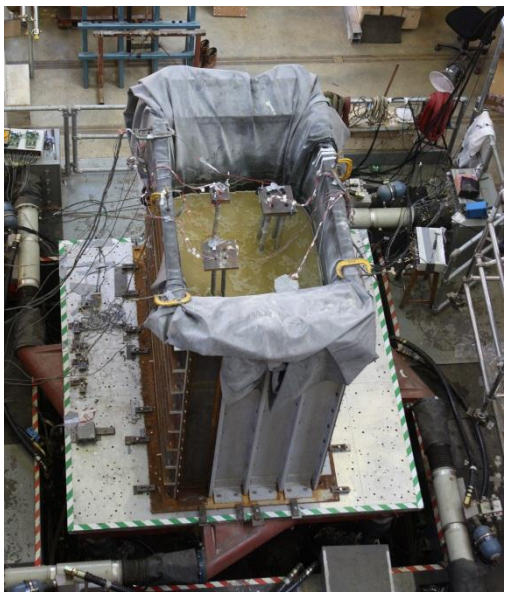

Figure 1. Experimental rig mounted on the shaking table.

The tests were carried out at normal gravity on four physical models, which consisted of two single piles and two pile groups (see Figure 2). Model *"Single Pile – 1"* and *"Single Pile – 2"* corresponded to the single pile models with outer diameter of 25.4 mm and 41.27 mm respectively. Model *"Pile Group – 1"* and *"Pile Group – 2"*corresponded to the pile groups, which were composed of four piles having outer diameter of 25.4 mm and 41.27 mm respectively. Aluminium alloy (type L114-T4 6082-T4) was chosen as the material for all piles. The dimensions and mechanical properties of the models are listed in Table 1. The pile spacing ratio (i.e. ratio between centre-to-centre distance and outer diameter) was 3 for both pile group configurations. This value was chosen according to the current practice, which adopts a spacing of 3 to 4 diameters (Fleming et al., 2008).

Table 1 Dimensions and mechanical properties of the physical models

Model ID	Outer Diameter [mm]	Thickness [mm]	Length [m]	EI [Nm2]	Pile cap dimension [mm]	Pile-cap weight [kg]
Single Pile 1	25.4	0.711	2	294	100×100×25.4	1.9
Single Pile 2	41.275	0.711	2	1305	150×150×25.4*	8.44
Pile Group 1	25.4	0.711	2	294	260×260×25.4	13.08
Pile Group 2	41.275	0.711	2	1305	260×260×25.4*	22.72

**Two plates are used for the pile cap.*

A schematic of the instrumentation layout used is shown in Figure 3. In order to estimate the dynamic response of the four models, each pile-cap was instrumented with an accelerometer. The input motion applied through the shaking table was also recorded by an accelerometer located on the table. Finally the ground response was measured by means accelerometers embedded in the soil deposit as illustrated in Figure 3. Excess pore water pressure were monitored by means of pore water pressure transducers (PPTs), which were placed at four depths as shown in Figure 3.

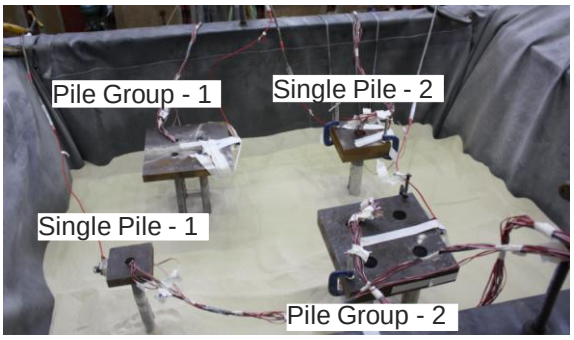

Figure 2. Physical models representing single and pile-group foundations

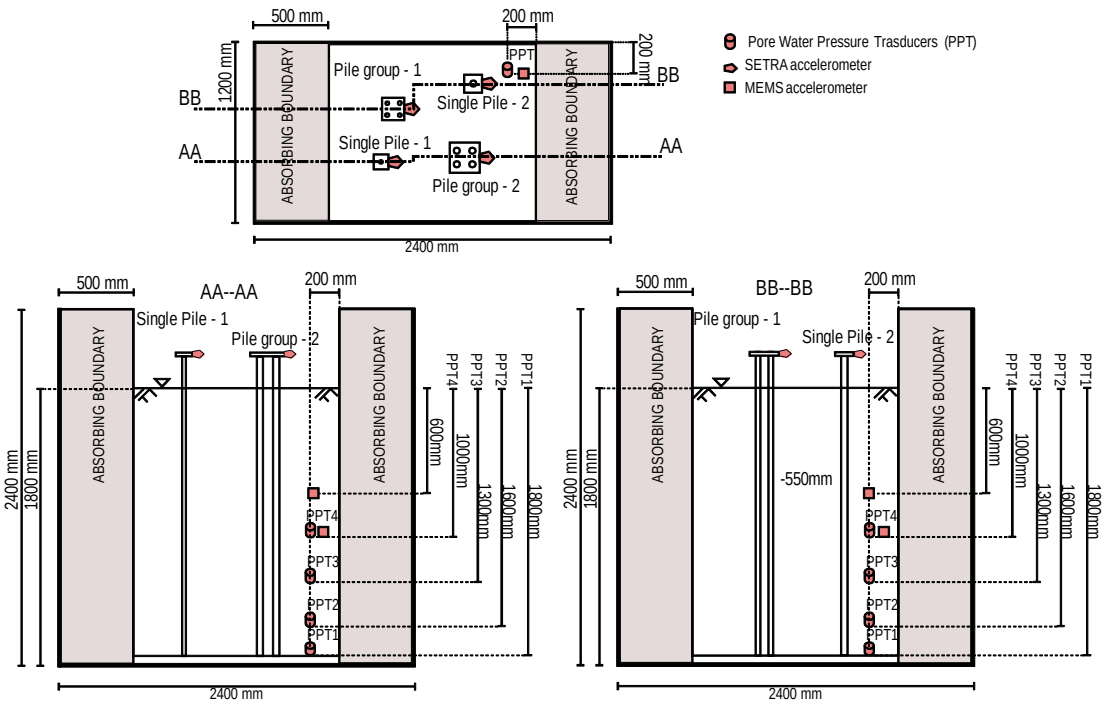

Figure 3. Schematic of instrumentation layout

The soil container was subjected to broadband random white noise having 0-100Hz bandwidth frequency, which was applied by the shaking table for a total duration of 300s. Specifically, three different acceleration amplitudes, respectively, 0.02g, 0.04g and 0.15g were applied without stopping the shaking, for a total duration of 100s each. The different acceleration levels aimed to invstigate the dynamic behaviour of the models in three different conditions, namely: (i) before liquefaction; (ii) during the transition to liquefaction; (iii) at full liquefaction.

The assessment of frequencies and damping ratios were performed dividing each phase into 15 “blocks” having a duration of 20s each. Considering that the longer the record the higher the frequency resolution is, the length of each block was so determined to obtain an acceptable frequency resolution of the FRF and to ensure that all the frequencies of interest were sufficiently excited. These two considerations may be achieved by choosing a length of the block much larger than the period of the lowest mode of interest. On the other hand, the length of the block cannot be very long, due to the fact that the system under investigation is highly non-linear and therefore its modal parameters are changing with time. In particular, the effect of having blocks of longer duration may cause widening of the spectrum, which may lead to an overestimation of the damping and erroneous assessment of the natural frequency. Subsequently the FRF was evaluated for each block considering the acceleration response recorded by the accelerometer mounted on the shaking table as input, and the response

measured by the accelerometer placed on the pile cap as output. To estimate the reliability of all measurements, the associated coherence function was estimated for each FRF. More details about the techniques used for the assessment of the modal parameters can be found in Lombardi and Bhattacharya (2014) and Lombardi (2014).

The damping ratio of the four models was estimated from the width of the resonant peak in the FRF using the "Half-Power Bandwidth Method". However, in order to achieve accuracy before the assessment of damping, the measured FRF was matched with a theoretical FRF of a single degree of freedom system using a curve fitting technique. The matching was carried out in the vicinity of the resonance peak by minimizing the squared error between the real response and the fitting function. The value of damping was estimated using the Half-Power Bandwidth Method, which was applied on the fitted FRF. Figure 4 shows a flow chart summarising the different modal analysis procedures employed for the assessment of natural frequencies and damping ratios of the four physical models.

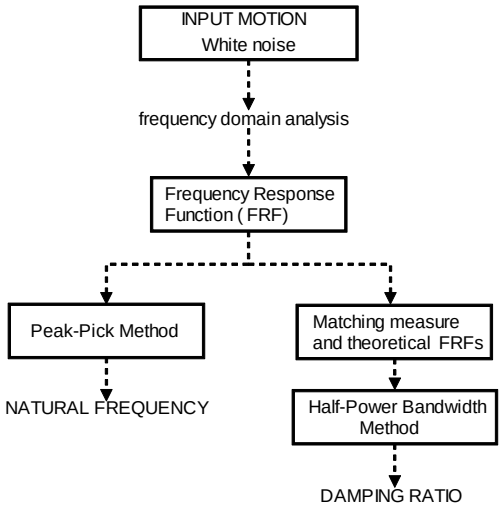

Figure 4. Flow chart illustrating the modal analysis procedures used for the assessment of the modal parameters

EXPERIMENTAL RESULTS

Figure 5 plots the variation of natural frequencies and damping ratios of the models during the excess pore water pressure development. The latter is commonly expressed in terms of the ratio between the excess pore water pressure Δu, and the effective overburden stress σ'_{v0}. This is commonly referred to as excess pore water ratio r_u. The change in frequency is represented in Figure 5 by the ratio f/f_{inital} where f is the average value of natural frequency estimated considering a block of 20 seconds, and f_{inital} is the frequency of the system measured in the first 20s of the input, which is assumed as the value of frequency in the initial condition.

From Figure 5 it may be observed that, in the first phase (i.e. 0-100s), the natural frequencies of the models slightly increased with the shaking, and no significant pore water pressure built up in the soil (r_u<0.1). The small increase in frequency was caused by the densification of the soil due to the small amplitude shaking (i.e. acceleration level ~0.02g). In the second phase (i.e. 0-200s), as r_u increased (a maximum of r_u =0.35 was measured at a depth of 1m), the natural frequencies reduced by 20%-25% for models *Single Pile – 2* and *Pile Groups – 1 and 2* (i.e. f/f_{inital} ~0.80-0.75), although for the *Single Pile – 1* this change was considerably lower, probably due to the fact this model tilted slightly during the saturation of the soil. During the last phase (200-300s), the amplitude of the input motion was incremented to 0.15g, and the soil started to liquefy from top to bottom, this can be observed from excess pore water pressure ratios plotted in Figure 5. Full liquefaction was reached at about 250s up to a maximum depth of 1.6m. When this length was compared with the pile's length (2m), a ratio 1.6/2.0 = 0.8 was obtained (i.e. 80% of the pile was fully unsupported). At this stage, the frequency reduced to 20% for *Single Pile – 1* (i.e. f/f_{inital} ~0.80), 50% for *Pile Group – 1* (i.e. f/f_{inital} ~0.50) and 60% for *Single Pile – 2* and *Pile Group – 2* (i.e. f/f_{inital} ~0.40).

The variation of the damping ratios of the four physical models is plotted in Figure 5. It can be recognised that the initial damping ratios of the systems were different, specifically, 5% for *Single Pile – 1*, around 8% for *Single Pile – 1 and Pile Group – 1* and just above 10% for *Pile Group – 2*. In the first phase of loading (0-100 s), the damping ratios remained approximately constant for all models. In Phase 2 (100-200 s), as the acceleration amplitude increased, the damping slightly increased and reached a value of about 10%. At full liquefaction (i.e. after 250 s), the models experienced a significant increase in damping ratio up to values above 20%.

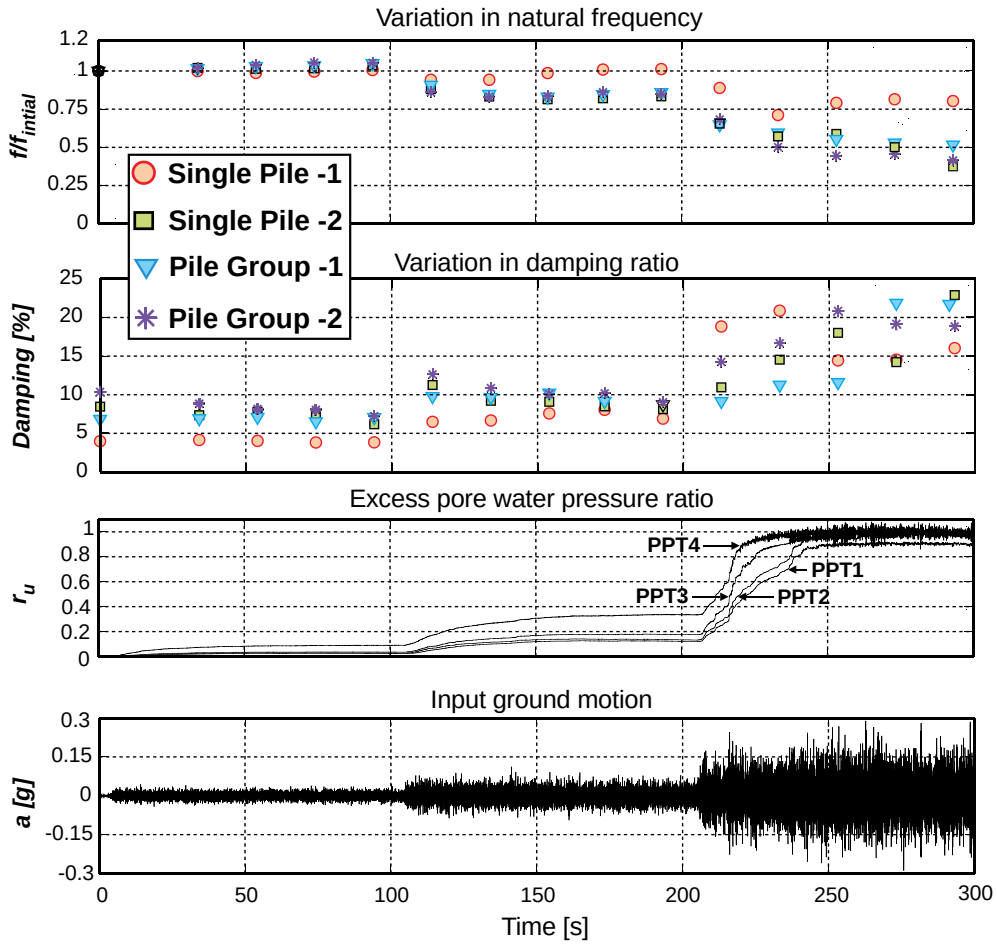

Figure 5. Variation of natural frequency and damping ratios of the four physical models subjected to a white noise input

EFFECTS OF EXCESS PORE PRESSURE ON RESPONSE SPECTRA

The response spectrum of an earthquake represents a practical tool for determining the seismic input to be considered for designing structures in earthquake-prone areas. However in this research, response spectra are computed in order to better understand the influence of soil-structure interaction on the seismic response of the models prior to and after soil liquefaction. This was achieved by evaluating the response spectra from the time histories recorded in the soil which can be considered representative of the free-field response. Furthermore, to better identify the effects induced by the soil softening, the response spectra from data recorded on the table have been also computed and compared with the spectra associated with the free-field. The structural damping ratio used in the calculation was calculated from the damping ratios of the structures estimated during the free vibration tests before the pluviation of the soil (Lombardi and Bhattacharya, 2014). Therefore, it is important to highlight that this damping reflected only the response of the structure and neglected the additional damping introduced by the flexibility of the foundation and SSI effects. Based on this consideration a damping ratio of 3% was employed in all analyses. Figure 6 depicts the computed acceleration

response spectra estimated from the accelerometer at 1 m depth. The spectra determined from the acceleration time histories recorded on the table have also been plotted in dotted lines for comparison. It should be noted that the spectral values plotted in Figure 6 are not pseudo quantities, however, these can be obtained by multiplying the spectral acceleration and velocity by the angular frequency and square angular frequency respectively.

The results showed that before liquefaction the highest spectral peak determined from the accelerometer located at 1 m depth, i.e. 1.34 g, occurred at 0.07 second. It can also be observed that the response spectra of the free-field was very similar in magnitude and shapes with the one obtained from the shaking table, which considered the response at the bedrock. After soil liquefaction, the free-field response spectra exhibited a significant attenuation of the spectral acceleration in the 0.1 s to 0.9 s period range. The de-amplification factor calculated with respect to the response spectra computed on the table after liquefaction (depicted in black dotted line) was by a factor of 1.4. Finally, it can be observed that the response spectra shifted towards long periods, which in turn slightly amplified spectral acceleration beyond 0.9 s.

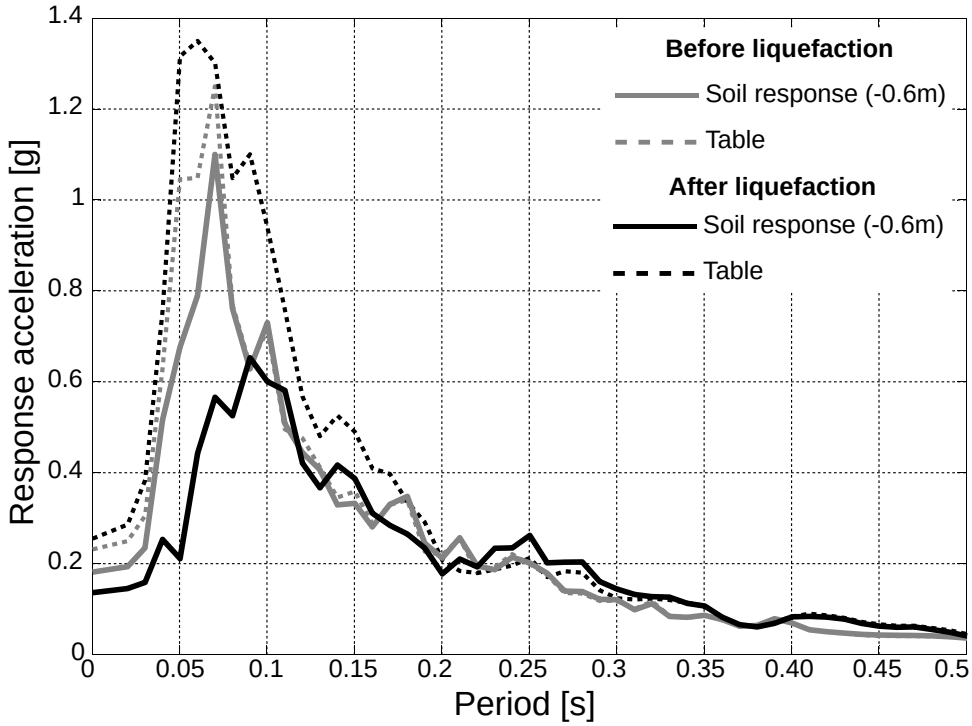

Figure 6. Computed acceleration response spectra estimated before and at full liquefaction obtained considering an average damping ratio of the models of 3%

To confirm the de-amplification of the spectral accelerations, the acceleration time histories measured by the accelerometers located on the pile heads of the four structures are plotted in Figure 7. The measurements clearly demonstrated that with the onset of liquefaction the acceleration response of the models was considerably reduced.

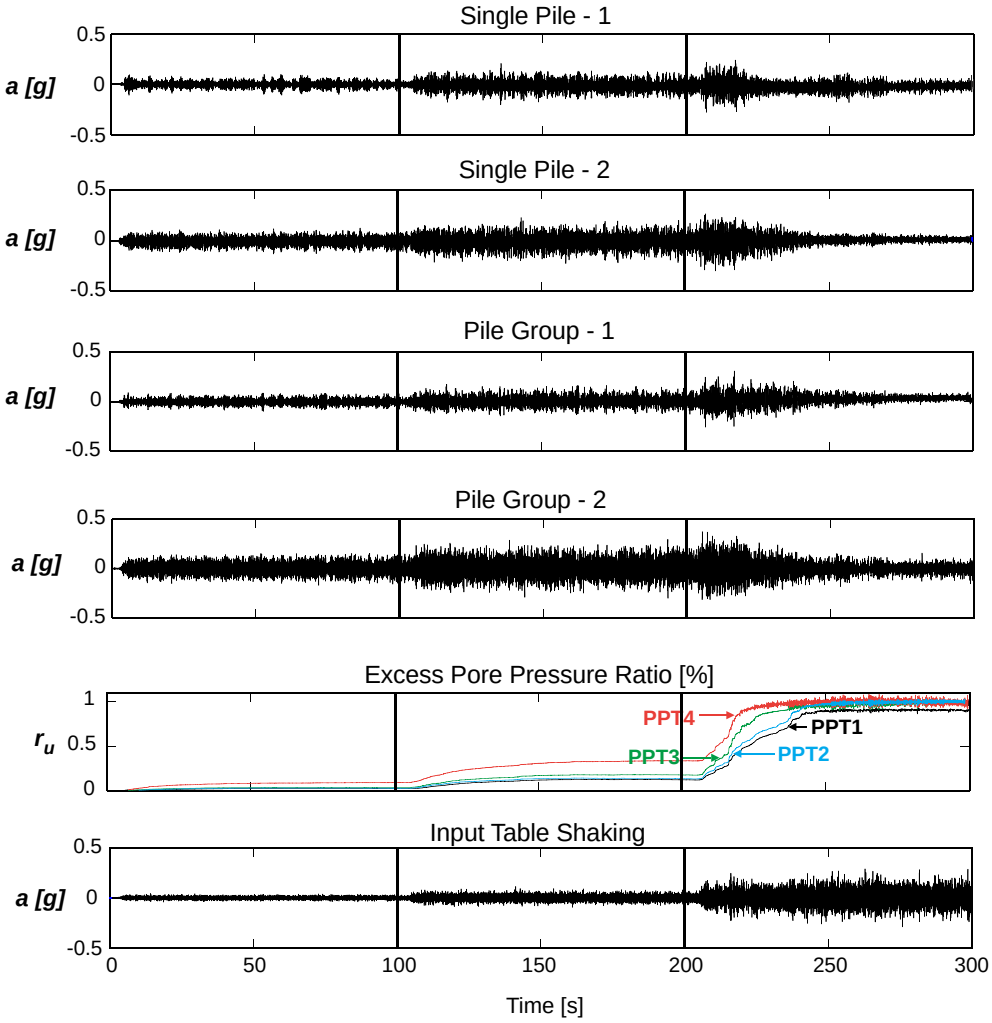

Figure 7. Acceleration time histories recorded on the pile caps of the four models

CONCLUSIONS

The large scale tests presented herein provided valuable data for investigating the dynamic behaviour of the four physical models that consisted of two single piles and two 2×2 pile groups. The results focused on the change in natural frequency and damping ratios of the models due to generation of excess pore water pressure. It is important to note that the full liquefaction condition (i.e. r_u=1.0) was reached at a maximum depth of 1.6m, thus, the results must be interpreted bearing in mind that the ratio between the total depth of liquefaction (1.6m) and the length of the pile (2m) was 0.8.

The experimental results showed that the natural frequency of pile-supported structures may decrease considerably owing to the loss of lateral support offered by the soil to the pile. On the other hand, the damping ratio of structure may increase to values in excess of 20%. These findings have important design consequences: (a) for low-period structures, substantial reduction of spectral acceleration is expected; (b) during and after liquefaction, the response of the system may be dictated by the interactions of multiple loadings, that is, horizontal, axial and overturning moment, which were negligible prior to liquefaction; and (c) with the onset of liquefaction due to increased flexibility of pile-supported structure, larger spectral displacement may be expected, which in turn may enhance P-delta effects and consequently amplification of overturning moment.

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