Seismic response of mid-rise buildings on shallow and end-bearing pile foundations in soft soil

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Abstract

The seismic behaviour of structures built on soft soil is influenced by the soil properties, and the response is significantly different from the fixed-base condition owing to the interaction between the ground and the structure. In this study, in order to investigate the influence of the foundation type on the response of structures, considering soil–structure interaction, a series of experimental shaking table tests has been conducted for three different cases, namely, (i) a fixed-base structure representing the situation excluding the soil–structure interaction; (ii) a structure supported by a shallow foundation on soft soil; and (iii) a structure supported by an end-bearing pile foundation in soft soil. A laminar soil container has been designed and constructed to simulate the free-field soil response by minimising the boundary effects. Simulating the superstructure as a multi-storey frame during the shaking table tests makes the experimental data unique. A fully nonlinear three-dimensional numerical model employing FLAC3D has been adopted to perform a time history analysis and to simulate the performance of the structure considering the seismic soil–structure interaction. Hysteretic damping of the soil is implemented to represent the variation in the shear modulus reduction factor and the damping ratio of the soil with cyclic shear strain. Free-field boundary conditions have been assigned to the numerical model and appropriate interface elements, capable of modelling sliding and separation between the pile and the soil elements, is considered. A comparison of the numerical predictions and the experimental data shows a good agreement confirming the reliability of the numerical model.

Keywords: Soil–pile–structure interaction; Shaking table test; FLAC3D; Laminar soil container; Shallow foundation; End-bearing pile foundation

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1. Introduction

The problem of soil–pile–structure interaction in the seismic analysis and design of structures has become increasingly important, as the building of structures in locations with less favourable geotechnical conditions in seismically active regions is becoming inevitable. The influence of the underlying soil on the seismic response of a structure can be disregarded when the ground is stiff...
enough, and consequently, the structure can be analysed considering the fixed-base conditions. However, the same structure will behave differently when it is constructed on a soft soil deposit. Earthquake characteristics, the travel path, the local soil properties, and the soil–structure interaction are the factors affecting the seismic excitation experienced by structures. The results of the first three factors can be summarised as free-field ground motion. However, the foundation of a structure does not follow the deformation of the free-field motion due to its stiffness, and the dynamic response of the structure itself induces the deformation of the supporting soil (Kramer, 1996).

The dynamic equation of motion for the structure (Fig. 1) can be written as

$$[M][\ddot{u}] + [C][\dot{u}] + [K][u] = -[M][1]\ddot{u}_g$$

(1)

where $[M]$, $[C]$, and $[K]$ are the mass, the damping, and the stiffness matrices of the structure, respectively. In addition, $\{u\}$, $\dot{\{u\}}$, and $\ddot{\{u\}}$ are the relative nodal displacements, the velocities, and the accelerations of the structure with respect to ground, respectively. It is more appropriate to use the incremental form of Eq. (1) when plasticity and the nonlinear response are included; and therefore, the matrix $[K]$ should be the tangential stiffness matrix and $\ddot{u}_g$ is the earthquake-induced acceleration at the level of the bedrock. If the supporting soil is compliant, the foundation can translate and rotate. Characteristics of such a system with end-bearing pile foundations can be represented by a series of horizontal and vertical foundation springs, as shown in Fig. 1. Accordingly, due to the soil–pile–structure interaction, the natural frequency of the system decreases in comparison to the fixed-base condition. Moreover, the damping ratio of the system, including the soil–pile–structure interaction, is larger than the damping ratio of the fixed-base structure for typical soils and foundations. Consequently, the soil–pile–structure interaction tends to increase the overall displacement of the superstructure due to the translation and the rotation of the foundation (Han and Cathro, 1997). This effect is important for tall, slender structures or for closely spaced structures that can be subjected to pounding when relative displacements become large (Kramer, 1996). Moreover, an increase in the total deformation of the structure and, in turn, the secondary $P-\Delta$ effect, influences the total stability of the structure. This is supported by lessons learned from the failure of rigid body buildings in past earthquakes (e.g., Mendoza and Romo, 1989; Mizuno et al., 1996; Motosaka and Mitsuji, 2012).

The complexity of the Seismic Soil–Pile–Structure Interaction (SSPSI) problem and the unavailability of a standard and validated analysis techniques routinely result in disregarding or greatly simplifying the presence of pile foundations for structural design. The main challenge of the soil–structure interaction problem is that the two disciplines of structural and geotechnical engineering meet simultaneously. However, the analysis is usually conducted separately. A geotechnical engineer may idealise a complex multimode superstructure as a single degrees of freedom oscillator and, on the other hand, a structural engineer may ignore the SSPSI or represent the nonlinear soil–pile interaction with simple linear springs. In this way, the nonlinear system interaction between the superstructure and the substructure is artificially prevented (Meymand, 1998).

Over the past decades, several researchers (e.g., Carbonari et al., 1990; Gazetas, 1991; Hayashi and Takahashi, 2004; Hokmabadi et al., 2011; Shirato et al., 2008; Yamashita et al., 2011) have studied the seismic soil–pile–structure interaction (SSPSI) and the effects of this phenomenon on the response of various structures. The developed analytical methods for studying these soil–structure interaction may be classified into the following three groups: (i) Substructure Methods (or Winkler methods), in which a series of springs and dashpots are employed to represent the soil behaviour. The available substructure

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**Fig. 1.** Schematic modelling of the multi degree freedom structure considering: (a) fixed-base structure; (b) structure supported by shallow foundation; (c) structure supported by end-bearing pile foundation.
methods to model the dynamic behaviour of the soil can be categorised from a simple linear spring derived from an elastic half-space assumption (Gazetas, 1991) to the more sophisticated models in which the soil medium is divided into the inner zone, adjacent to the pile that accounts for the soil nonlinearity, and the outer zone, that allows for wave propagation away from the pile and considers the radiation damping in the soil medium (Mostafa and El Naggar, 2002). Winkler methods, due to their simplicity, have been frequently used in practice to represent the soil medium in SSI analyses accounting for the dynamic behaviour of the soil and possible uplift, gaping or sliding. However, as mentioned by many researchers (e.g., Allotey and El Naggar, 2008; Finn, 2005; Hokmabadi et al., 2012a), the idealisation of the soil continuum with discrete soil reactions and the avoidance of the shear transfer between the springs are the obvious missing fundamental mechanisms in the Winkler models. 

(ii) Elastic Continuum Methods, which are based on Mindlin (1936) closed-form solutions for the application of point loads to a semi-infinite elastic medium. Tajimi (1969) was the first person to use the elastic continuum theory to describe a dynamic soil–pile interaction. Poulos (e.g., Tabesh and Poulos, 2001) has been a major progenitor of using elastic solutions for the pile foundation response to axial and lateral loads, and presented a comprehensive set of analysis and design methods for pile foundations based on the elastic continuum theory. However, in elastic continuum methods, the accuracy of the solutions is based on an evaluation of the soil elastic parameters, and it is difficult to incorporate small strain and steady state problems. Thus, these methods are more appropriate for small strain and steady state problems. (iii) Numerical Methods: the extensive ability of powerful computers has significantly changed computational aspects making them useful for problems studying complex and complicated interactive behaviours. By exploiting these methods, it is possible to conduct time history analyses considering effects such as small strain, linear stress-strain behaviour of the soil and the superstructure, material and radiation damping, advance boundary conditions, and interface elements. Another advantage of employing numerical methods is the capability of performing SSI analysis on pile groups in a fully-coupled manner, without resorting to independent calculations of site, superstructure response, or the application of pile group interaction (Meymand, 1998). Consequently, numerical modelling predictions can capture different parameters involved in SSSI that are closer to reality (e.g., Dutta and Roy, 2002; Tabatabaiefar et al., 2013), and thus, have been adopted in this study.

It should be noted that the regulated procedures in the available codes, such as ATC (ATC-40, 1996), NEHEAP (BSSC, 2009), and ASCE (ASCE7-10, 2010), do not provide a procedure that can account for the different types of foundations in an elaborative manner. Accordingly, a simplified method representing the subsoil, through a series of springs and dashpots (impedance functions), and the superstructure, as an SDOF oscillator, has been adopted in the regulated codes. Moreover, a linear equivalent behaviour is adopted for the subsoil in the above-mentioned codes without capturing any soil nonlinearity directly, where soil stiffness and damping are assumed to be constant during the solution process.

1.1. Performance-based seismic design

Performance-based seismic design is a modern approach to earthquake-resistant design. Seismic performance (performance level) is described by considering the maximum allowable damage state (damage performance) for an identified seismic hazard (hazard level). Performance levels describe the state of structures after being subjected to certain hazard levels. And, based on FEMA273/274 (BSSC, 1997), they are classified as fully operational, operational, life safe, near collapse, or collapse. Overall, performance, ductility demand, and inter-storey drifts are the most commonly used damage parameters. The above-mentioned five qualitative levels are related to the corresponding quantitative maximum inter-storey drifts (as a percentage of story height) of < 0.2%, < 0.5%, < 1.5%, < 2.5%, and < 2.5%, respectively (BSSC, 1997). In addition, most of the force-based design codes employ an additional check in terms of limiting inter-storey drifts to ensure that particular deformation-based criteria are met. For example, ASCE (ASCE7-10, 2010) limits the allowable storey drift for structures according to the type and risk category of the structure. The Australian Earthquake Code (AS1170.4, 2007) indicates 1.5% as the maximum allowable storey drift. It is believed that the inter-storey drift is the most acceptable parameter for controlling displacement and resulting damage, and, in turn, the performance of the structure.

The aim of the present research is to evaluate and quantify the effect of foundation type (shallow and deep foundations) on the response of structures considering SSI, which is significantly important to the performance-based design of structures. Different types of foundations can alter the dynamic properties of the system, such as stiffness, damping, and natural frequency, which have been investigated in this study by conducting both experimental and numerical modelling. A three-dimensional explicit finite-difference program, FLAC3D (Itasca, 2009), is used to numerically model and examine the influence of the soil–structure interaction on the seismic response of a 15-storey moment-resisting building. The proposed numerical soil–structure model has been verified and validated against experimental shaking table test results.

2. Shaking table experimental tests

Model tests in geotechnical engineering offer the advantage of simulating complex systems under controlled conditions and providing the opportunity to better understand the fundamental mechanisms of these systems. Such tests are often used as calibration benchmarks for numerical or analytical methods, or to make quantitative predictions of the prototype response (Rayhani et al., 2008).

In previously conducted shaking table tests (e.g., Chau et al., 2009; Ishimura et al., 1992; Jakrapanyanun, 2002; Pitilakis et al., 2008; Meymand, 1998), the superstructure is simplified as a single degree of freedom oscillator in which the behaviour of the soil–structure system may not completely conform to reality and higher modes would not be captured. In the current model tests, unlike the previous efforts, a multi-storey frame is adopted for the superstructure which represents most of the dynamic properties of the
prototype structure, such as natural frequency of the first and higher modes, the number of stories, and density. Moreover, an advanced laminar soil container has been designed to simulate the free-field soil response by minimising the boundary effects. Consequently, in the current shaking table tests, by adopting the same soil properties, the same superstructure, the same input motions, and the same test setup, a clear comparison has been provided between the structural responses of the different types of foundations (i.e., shallow and deep foundations).

The experimental model tests have been carried out utilising the shaking table facilities located at the Structures Laboratory of the University of Technology Sydney (UTS). Table 1 summarises the specifications of the UTS shaking table.

### 2.1. Prototype characteristics and scaling factors

A 15-storey concrete moment-resisting building frame with a total height of 45 m and a width of 12 m, consisting of three spans and representing a conventional type of mid-rise moment-resisting buildings, is selected for this study, as shown in Fig. 2. The spacing between the frames is 4 m. The natural frequency of the prototype building is 0.384 Hz and its total mass is 953 t. The soil medium beneath the structure is a clayey soil with a shear wave velocity of 200 m/s and a density of 1470 kg/m³. The horizontal distance of the soil lateral boundaries and the bedrock depth were selected to be 60 m and 30 m, respectively. The building rests on a footing which is 1 m high and 15 m long. For the pile foundation (Fig. 2b), a 4 × 4 reinforced concrete pile group with equal spacing, a pile diameter of 1.2 m, and a length of 30 m, is considered. The piles are embedded into the bedrock representing a typical end-bearing pile foundation.

In order to achieve a reasonable scale model, a dynamic similarity between the model and the prototype should be applied, as described in the literature (e.g., Harris and Sabnis, 1999; Langhaar, 1951; Meymand, 1998). Dynamic similarity governs a condition where homologous parts of the model and the prototype experience homologous net forces. The scaling relations for the variables contributing to the primary modes of the system response are presented in Table 2.

Adopting an appropriate geometric scaling factor (\(\lambda\)) is one of the important steps in scale modelling on a shaking table. Although small-scale models could save on costs, the precision of the results could be substantially reduced considering the specifications of the UTS shaking table (Table 1), a scaling factor of 1:30 provides the largest achievable scale model with rational scales, the maximum payload, and an overturning moment that meets the facility limitations. Thus, a geometric scaling factor (\(\lambda\)) of 1:30 is

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**Table 1**

UTS shaking table specifications.

<table>
<thead>
<tr>
<th>Size of table</th>
<th>3 m × 3 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum payload</td>
<td>10 t</td>
</tr>
<tr>
<td>Overturning moment</td>
<td>100 kN-m</td>
</tr>
<tr>
<td>Maximum displacement</td>
<td>± 100 mm</td>
</tr>
<tr>
<td>Maximum velocity</td>
<td>± 550 mm/s</td>
</tr>
<tr>
<td>Maximum acceleration</td>
<td>± 2.5 g or 0.9 g (full load)</td>
</tr>
<tr>
<td>Testing frequency</td>
<td>0.1-100 Hz</td>
</tr>
</tbody>
</table>

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**Fig. 2.** (a) Prototype structure supported by shallow foundation; (b) prototype structure supported by end-bearing pile foundation.
adopted for the experimental shaking table tests on the scale model in this study. According to Table 2, apart from the geometric scaling which should be imposed on all the components, the required scaled natural frequency for the structural model, the required scaled shear wave velocity, and the density of the soil mix should be 2.11 Hz, 36 m/s, and 1470 kg/m³, respectively. Moreover, the required scaled natural frequency of the soil mix inside the soil container needs to be 10 Hz, which is used as a benchmark to design the laminar soil container.

2.2. Model components of shaking table tests

The developed soil–structure model for the shaking table tests possesses four main components, including the model structure, the model pile foundation, the laminar soil container, and the soil mix. Details and characteristics of these components are explained below.

2.2.1. Model structure

Employing a geometric scaling factor of 1:30, the height, length, and width of the structural model were determined to be 1.50 m, 0.40 m, and 0.40 m, respectively. According to the scaling relationship shown in Table 2, the required natural frequency of the structural model is 2.11 Hz. In addition, the density of the model and the prototype should be equal. Thus, a total mass of 106 kg is obtained for the model structure.

In order to simulate the prototype structure more accurately on the shaking table, the model structure has been designed employing SAP2000 (CSI, 2010) software considering the required characteristics of the model structure. The 3D numerical model consists of thirteen horizontal steel plates as the floors and four vertical steel plates as the columns. Steel plate grade 250, according to Australian standards (AS/NZS3678, 2011), with a minimum yield stress of 280 MPa and a minimum tensile strength of 410 MPa, has been adopted in the design. The thickness of the steel plates was determined in the design process after several cycles of trial and error in order to fit the required natural frequency and the mass of the model structure. The finalised base plate is a 500 mm/C2 500 mm/C2 10 mm steel plate, while 400 mm/C2 400 mm/C2 5 mm plates are used for the floors and four 500 mm/C2 400 mm/C2 2 mm steel plates are used for the columns. The connections between the columns and the floors are provided by stainless steel metal screws with a diameter of 2.5 mm and a length of 15 mm. After the numerical modelling and the design, the structural model was constructed in house. The completed structural model is shown in Fig. 3.

2.2.2. Pile foundation

Similar to the model structure, the model pile should be subjected to the competing scale model criteria. In order to achieve a successful model pile design, the principal governing factors of the pile response, such as slenderness ratio L/d, the moment curvature relationship, flexural stiffness EI, the relative soil/pile stiffness, the yielding behaviour/mechanism, and the natural frequency of vibration should be addressed (Meymand, 1998). By adopting geometric similarity, the overall pile slenderness and relative contact surface area would be preserved in the model. This also guarantees that the pile group’s relative spacing and the consequent group interaction would be replicated at the model scale. Thus, by considering the geometric scaling factor (λ) of 1:30 in this study, the model piles should have a diameter of 40 mm with an L/d ratio of 25.

The moment–curvature relation criterion represents the pile response to the lateral loading which is a function of the flexural rigidity and yielding behaviour. Since, in the present study, piles
are intended to respond in the elastic range (this assumption is confirmed numerically), this criterion is achieved by scaling the flexural rigidity ($EI$) of the piles according to Table 2 ($\lambda = 1/30$) in addition to ensuring that the yielding point of the model pile is equal to or greater than the scaled prototype. Furthermore, by scaling the stiffness of the soil and the pile consistently, the relative soil/pile stiffness parameter will inevitably be satisfied. Therefore, the soil–pile interaction should then be accurately reproduced in the model.

Previous researchers (e.g., Bao et al., 2012; Chau et al., 2009; Tao et al., 1998) have used different types of materials like aluminium tubes, steel bars, and reinforced concrete to build model piles. Considering the selected scaling factor in this study ($\lambda = 1/30$) and, in turn, the required stiffness and yielding stress for the model piles, a commercial Polyethylene high pressure pipe with a Standard Dimension Ratio (SDR) of 7.4, according to the Australian Standard (AS/NZS4130, 2009), is the selected candidate which falls in the range of acceptable criteria with a 5% deviation from the target value for $E_I$. Moreover, Polyethylene pipes can tolerate large deformation prior to the yielding point without any brittle failure. Characteristics of the model pile used in this study are summarised in Table 3.

### 2.2.3. Soil mix and earthquake records

In this study, a synthetic clay mixture was designed to provide the soil medium for the shaking table testing. In order to develop the synthetic clay mixture, Q38 kaolinite clay, Active-bond 23 Bentonite, class F fly ash, lime, and water were used as the components of the soil mixture. Bender element tests were performed to measure the shear wave velocity over the curing age. To carry out the reported bender element tests (Fatahi et al., 2013), the soil specimens were placed between bender elements, and the shear wave velocity of each soil specimen was obtained by measuring the time required for the wave to travel between two bender elements using a computer running GDS bender element control software. The adopted system has a data acquisition speed of 2,000,000 samples per second, 16 bit as the resolution of data acquisition, and a connection to the control box through a USB link. In this study, the propagated shear wave type has been sine waves with an amplitude of 10 V and a period of 1 s. Fig. 4 shows the schematic graphical signal processing to measure the shear wave travel time in the bender element tests.

Several mixtures have been examined, and finally the desired soil mix of 60% Q38 kaolinite clay, 20% Active-bond 23 Bentonite, 20% class F fly ash and lime, and 120% water (% of the dry mix) produced the required scaled shear wave velocity of 36 m/s on the second day of its curing age. Table 4 summarises the soil mixture properties on the second day of curing which has been adopted in this study. Accordingly, the soil density on the second day was determined to be 1450 kg/m³, being almost equal to the prototype soil density (1470 kg/m³) as required. Therefore, the designed soil mixture possesses the required dynamic similarity characteristics.

Each test model was subjected to two near field shaking events, Kobe, 1995 and Northridge, 1994, and two far field earthquakes, El Centro, 1940 and Hachinohe, 1968. The characteristics of the these earthquakes suggested by the International Association for Structural Control and Monitoring for benchmark seismic studies (Karamodin and Kazemi, 2010) are summarised in Table 5. As with the other components of the model, the imposed earthquake excitations should be scaled as well. Refer to Table 2, although the model earthquake magnitude remains the same as the prototype, the time intervals of the original records should be reduced by a factor of 5.48 ($\lambda^{1/30}$), which means that the scaled earthquakes contain higher frequencies and shorter durations. The scaled acceleration records of the four adopted earthquakes are illustrated in Fig. 4a to d.

### 2.2.4. Laminar soil container

A soil container is required to hold the soil in place during the shaking table test and to provide confinement. The ideal soil container should simulate the free-field soil response by minimising the boundary effects. Since the seismic behaviour of the soil container affects the interaction between the soil and the structure, the performance of the soil container is of key importance for conducting seismic soil–structure interaction model tests successfully (Pitilakis et al., 2008). A well-designed laminar soil container, as determined by many researchers (e.g., Chau et al., 2009; Taylor, 1997), has the advantage over other types in that the lateral motion of the entire depth of the laminar soil container follows a sinusoidal shape representing the authentic conditions of the free-field ground motion. Therefore, in order to perform rigorous and reliable experimental shaking table tests, a laminar soil container has been employed in this study.

![Fig. 4. Schematic graphical signal processing to measure the shear wave travel time between the sender and receiver bender elements.](image-url)
The distance from the model structure to the lamina is twice the width of the structure \( (b=400 \text{ mm}) \) on each side of the shaking direction, giving an overall length of the laminar soil container equal to \( 5B + 100 \text{ mm extra for construction purposes} \) (2100 mm). Since the employed shaking table facility can apply excitation in just one direction, the overall width of the laminar soil container, in the direction perpendicular to the shaking direction, is considered equal to \( 3B + 10 \text{ mm extra for construction purposes} \) (1300 mm) following the research done by Rayhani and El Naggar (2008). As with the model structure, the laminar soil container is initially designed employing a 3D numerical model. The key parameter in the design of the soil container is the natural frequency of the container itself which should be close to the natural frequency of the soil deposit inside the container (approximately 10 Hz for this study) in order to minimise the interaction between the soil and container during the shaking table tests.

The employed laminar soil container consists of a rectangular laminar box made up of aluminium rectangular hollow section frames separated by rubber layers. The aluminium frames provide lateral confinement of the soil, while the rubber layers allow the container to deform in a shear beam manner. The soil container was fixed and secured on the shaking table using eight M38 bolts passing through the provided holes. Then, the internal surface of the soil container was covered and sealed with two layers of black plastic sheeting. Several researchers (e.g., Gohl and Finn, 1987; Valsangkar et al., 1991) successfully employed absorbing layers, such as Polystyrene foam, in order to absorb the energy and to reduce the wave reflection from the boundaries during the experimental simulations. This layer was attached to both end

<table>
<thead>
<tr>
<th>Soil properties</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mass density (kg/m³)</td>
<td>1450</td>
</tr>
<tr>
<td>Shear wave velocity (m/s)</td>
<td>36</td>
</tr>
<tr>
<td>Maximum Shear modulus, ( G_{max} ) (kPa)</td>
<td>1776</td>
</tr>
<tr>
<td>Undrained shear strength, ( S_u ) (kPa)</td>
<td>3.1</td>
</tr>
<tr>
<td>Plastic index, ( PI ) (%)</td>
<td>42</td>
</tr>
</tbody>
</table>

Table 4
Required properties of the soil mix on the second day of curing adopted in the 3D numerical model.

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>Country</th>
<th>Year</th>
<th>PGA (g)</th>
<th>Mw (R)</th>
<th>Duration (s)</th>
<th>Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Northridge</td>
<td>USA</td>
<td>1994</td>
<td>0.843</td>
<td>6.7</td>
<td>30.0</td>
<td>Near field</td>
</tr>
<tr>
<td>Kobe</td>
<td>Japan</td>
<td>1995</td>
<td>0.833</td>
<td>6.8</td>
<td>56.0</td>
<td>Near field</td>
</tr>
<tr>
<td>El Centro</td>
<td>USA</td>
<td>1940</td>
<td>0.349</td>
<td>6.9</td>
<td>56.5</td>
<td>Far field</td>
</tr>
<tr>
<td>Hachinohe</td>
<td>Japan</td>
<td>1968</td>
<td>0.229</td>
<td>7.5</td>
<td>56.5</td>
<td>Far field</td>
</tr>
</tbody>
</table>

Table 5
Utilised earthquake base motions.

Fig. 5. Scaled earthquake records: (a) Northridge earthquake; (b) Kobe earthquake; (c) El Centro earthquake; (d) Hachinohe earthquake.
walls which are perpendicular to the shaking direction. Zeng and Schofield (1996) explained some uncertainties in adopting different types of absorbing layers for physical modellings. It can be concluded that by allocating a sufficient distance between the soil boundaries and the structural model, the influence of the mechanical properties of the absorbing layer, such as friction and stiffness, is negligible in the final response of the model. Accordingly, 25-mm-thick absorbing layers of Polystyrene foam sheets have been installed at the end walls of the soil container. In addition, a layer of well-graded gravel was glued to the bottom of the soil container to avoid any possible torsion of the structure during seismic excitations could be monitored.

Initially, a Sine Sweep test was performed on the structural model to determine the natural frequency of the model. Sine Sweep tests involve a logarithmic frequency sweep holding a specified acceleration constant at the base of the structure. For the current Sine Sweep test, the frequency of the shaking table increased from 0.1 Hz to 50 Hz. The first resonance between the shaking table and the structural model frequencies showed the fundamental natural frequency of the model. This test was repeated three times to ensure that the determined natural frequency was adequately accurate. The natural frequency of the constructed structural model obtained from the Sine Sweep test results was 2.19 Hz, which is in very good agreement with the desired natural frequency of the structural model (2.11 Hz). Therefore, the constructed structural model, with a natural frequency of 2.19 Hz and a total mass of 104 kg, possesses the required characteristics to meet the dynamic similarity criteria. The estimated value of the structural damping ratio of the constructed structural model was determined to be equal to 1.1%, obtained from the free vibration and displacement records of the structural model using the Taylor series expansion (Craig and Kurdila, 2006).

After ensuring the adequacy of the structural model characteristics, shaking table tests were performed by applying the scaled earthquake acceleration records of Kobe, 1995 (Fig. 2a), Northridge, 1994 (Fig. 2b), El Centro, 1940 (Fig. 2c), and Hachinohe, 1968 (Fig. 2d) to the fixed base structural model; the results in terms of the maximum lateral deflections are presented in Fig. 8. To determine the lateral deflections, the movement of the shaking table was subtracted from the storey movements. Therefore, all the records are relative to the base movements. It should be noted that the presented data are based on the lateral deformation of each storey when the maximum deflection at the top level occurs. This approach provides a more reasonable pattern of structural deformation than the approach for which the maximum absolute storey deformation is recorded irrespective of the occurrence time (Hokmabadi et al., 2012b). Fig. 7 illustrates a sample of the time history deformation records used to obtain the lateral deformation reported in Fig. 8.

The second case of the shaking table tests was to study the effect of the soil–structure interaction under shallow foundation cases. After securing the laminar soil container on the shaking table, 2 m³ of the designed soil mixture (60% Q38 kaolinite clay, 20% Active-bond 23 Bentonite, 20% class F fly ash and lime, and water), 120% of the dry mix was produced and placed into the laminar soil container. As explained in Section 2.2.4, the desired soil mixture acquires the required stiffness, and consequently, the shear wave velocity after two days of curing. As a result, the time frame for the testing process was very tight and time sensitive. Therefore, soil mixing and placement needed
Maximum lateral deformation (mm)

Fig. 7. Sample experimental time-history displacement results for the fixed base model under the influence of El Centro earthquake.

to be carried out in one day in order to produce a homogenous soil mixture and, after two days of curing, the final tests were to be performed.

During the soil mixing process, ten cylindrical soil samples, of $D = 50$ mm and $h = 100$ mm, were taken from the soil mixture for quality control of the mixture. The entire mixing process and the filling of the laminar soil container were completed in one day. Then, the soil mixture inside the container was left to cure for two days, while the surface of the soil container was covered and sealed. On the second day, the structure model was lifted up and placed on the designated location. In order to prevent any excessive settlement or failure underneath the base plate during installation, the model piles were driven into the soil through a 150-cm-tall wooden template to ensure the location and the verticality of the template was constructed with special cut outs to accommodate a few millimetres of extra room for the piles with internal strain gages for the purpose of preventing any possible damage to the strain gages during installation.

After the installation of the model piles, the template was removed and the steel plate (simulating the foundation) with prefabricated holes was fitted over the group. Sixteen M12 bolts were used to provide a fixed connection between the pile heads and the steel plate. The required nuts were fixed to the pile tops with strong glue and steel rings before the test, and the strength and the capability of this connection technique was examined successfully. Then, the model structure was suspended from an overhead crane and connected to the steel plate similar to the fixed-base and shallow foundation cases.

Consequently, all the components of the system, including the container, soil, piles, and superstructure were installed. The same arrangement of displacement transducers and accelerometers was used on the structure and the steel plate (simulating the foundation). Moreover, fifteen strain gauges were installed on the piles and four 3D accelerometers were embedded inside the soil body. Since the influence of the soil–structure interaction on the response of the superstructure is the main objective of this research, just the data obtained from the instrumentation on the structure itself, not the soil or pile sensors, are of main interest, and thus, are reported in this paper. Similar shaking events, including the Sine Sweep test and four scaled earthquake records (Fig. 5), were applied to the end-bearing pile foundation and to investigate the influence of the soil–structure interaction on the seismic response of the structure itself, not the soil or pile sensors, are of main interest, just the data obtained from the instrumentation on the structure itself, not the soil or pile sensors, are of main interest, and thus, are reported in this paper. Similar shaking events, including the Sine Sweep test and four scaled earthquake records, have been applied to the end-bearing pile foundation system. The natural frequency of the soil–structure model from the performed Sine Sweep test was measured to be 1.93 Hz. The results of the shaking table tests under the influence of the four scaled earthquake acceleration records, in terms of the maximum lateral deflections of various stories of the structure, are presented and compared in Fig. 8. The final setup of the tests,
Fig. 8. Recorded maximum lateral deflection of the structure from the shaking table tests for the fixed base, shallow foundation, and end-bearing pile foundation cases under the influence of: (a) Northridge earthquake; (b) Kobe earthquake; (c) El Centro earthquake; (d) Hachinohe earthquake.
including the displacement transducers and the accelerometers at different levels of the structural model for the end-bearing pile foundation system on the shaking table, are presented in Fig. 6. A discussion on the experimental results is presented in Section 4.

3. Development of 3D numerical model

A three-dimensional explicit finite-difference based program, FLAC3D, Fast Lagrangian Analysis of Continua, version 4.0 (Itasca, 2009) has been employed following the experience of other researchers (e.g., Comodromos and Papadopoulou, 2012; Ghee and Guo, 2010; Rayhani et al., 2008; Tamura et al., 2012) to develop a numerical model for the shaking table tests and to simulate the response under seismic loading. This program can simulate the behaviour of different types of structures and materials by elements which can be adjusted to fit the geometry of the model. Each element behaves according to a prescribed constitutive model in response to the applied forces or boundary restraints. The program offers a wide range of capabilities for solving complex problems in mechanics, such as inelastic analyses, including the plastic moment and the simulation of hinges for structural systems.

Three cases including the fixed-base condition, the structure supported by the shallow foundation, and the structure supported by the end-bearing pile foundation have been modelled separately and the results are compared. The dimensions of the numerical models were chosen to be similar to the experimental tests. The reason for choosing a soil deposit thickness of 30 m for both the experimental and the numerical models is that most of the amplification occurred within the first 30 m of the soil, which is in agreement with most modern seismic codes calculating local site effects based on the properties of the top 30 m of the soil profile (Rayhani and El Naggar, 2008).

Experience gained from the parametric studies helped to finalise the adopted mesh size and the maximum unbalanced force at the grid points to optimize the accuracy and the computation speed, simultaneously. For the end-bearing pile foundation model, the generated mesh comprised of 10,868 zones and 16,356 grid points. Fast computation facilities at the University of Technology Sydney were employed to conduct a time history analysis, and the computation took approximately 20 h for a single analysis. The numerical grid and the model components in FLAC3D are shown in Fig. 9.

Adjusting the boundary conditions for the static analysis, in which the system is only under gravity loads, the bottom face of the model is fixed in all directions, while the side boundaries are fixed in horizontal directions. During the dynamic time history analysis, in order to avoid the reflection of outward propagating waves back into the model, quiet (viscous) boundaries comprising independent dashpots in the normal and shear directions are placed at the lateral boundaries of the soil medium. Viscous damping on the boundaries is a function of soil density and the velocity of the propagated wave. Referring to Lysmer and Kuhlemeyer (1969), the employed viscous normal and shear tractions are as follows:

$$t_n = -\rho C_p V_n$$
$$t_s = -\rho C_s V_s$$

where $V_n$ and $V_s$ are the normal and shear components of the velocity at the boundaries, respectively, $\rho$ is the mass density, and $C_p$ and $C_s$ are the velocities of the p-wave and the s-wave, respectively. In the developed numerical analysis procedure, the above-mentioned viscous terms are implemented as boundary loads at every time step. Alternatively, these viscous terms can be introduced directly into the equations of motion of the grid points lying on the boundary. Employing the viscous dashpots, the
lateral boundaries of the main grid are coupled to the free-field grids at the sides of the model, as shown in Fig. 2, to simulate the free-field motion which would exist in the absence of the structure and pile foundation. Rigid boundary conditions, adopted to simulate the bedrock in the seismic soil—structure interaction analysis, as suggested by other researchers (e.g., Dutta and Roy, 2002; Kocak and Mengi, 2000; Spyrakos et al., 2009), Lu et al. (2005) emphasised the influence of the gravity load on the contact state of the soil—structure interface mentioning that a significant error in the analysis may occur if gravity is not taken into account in the dynamic analysis.

Solid elements are used to model the soil deposits, and the Mohr–Coulomb failure criterion is adopted. In addition, the built-in tangent modulus function developed by Hardin and Drnevich (1972) is adopted to implement the hysteretic damping of the soil, representing the variation in shear modulus reduction factor, and the damping ratio \( D \) with the cyclic shear strain of the soil. This model is defined as follows:

\[
M_s = 1/(1 + (\gamma/\gamma_{ref}))
\]

where \( M_s \) is the secant modulus \((G/G_{max})\), \( \gamma \) is the cyclic shear strain, and \( \gamma_{ref} \) is Hardin/Drnevich constant. In this study, \( \gamma_{ref}=0.234 \), representing the backbone curves suggested by Sun et al. (1988) for fine grained soils, are adopted as illustrated in Fig. 10.

The cylindrical specimens taken from the soil mixture during the mixing process, as described in Section 2.3, were used to obtain the soil parameters. Common soil tests, such as bender element and density tests, were conducted on these specimens on the second day of curing age. The results are in good conformity with the recommended relationship by Rayhani and El Naggar (2008) and Itasca Consulting Group (Itasca, 2009) for the isotropic soil medium, as follows:

\[
k_s = k_n = 10 \left[ \frac{K + (4/3)G}{\Delta z_{min}} \right]
\]

where \( K \) and \( G \) are the bulk and shear modulus of the soil, and \( \Delta z_{min} \) is the smallest width of an adjoining zone in the normal direction. This is a simplifying assumption that has been used to ensure that the interface stiffness has minimal influence on the system compliance.

Finally, a fully nonlinear time history analysis is conducted under the influence of the scaled earthquake records (Fig. 5), and the results in terms of the maximum inelastic lateral

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![Fig. 10. Adopted fitting curve for fine grained soil in this study (after Sun et al., 1988): (a) relations between \( G/G_{max} \) versus cyclic shear strain; (b) relations between damping ratio versus cyclic shear strain.](image-url)
deflections determined for the three mentioned cases are presented in Fig. 12.

4. Results and discussion

The results of the shaking table tests and the 3D numerical predictions for the maximum lateral displacements of the fixed-base, shallow foundations, and end-bearing pile foundations are summarised and compared in Fig. 13. An evaluation of the predicted and observed values of the maximum lateral displacements indicates that the trend and the values of the 3D numerical predictions are in good agreement and consistent with the experimental shaking table test results. Therefore, the developed 3D numerical model can replicate the behaviour of the soil–pile–structure system with acceptable accuracy and is a rational and appropriate tool for further studies of the soil–pile–structure interaction effects.

The observed disparity between FLAC3D predictions and experimental measurements in the lower levels can be due to the nature of the numerical methods, the models employed to simulate the complicated dynamic behaviour of the cohesive soil, the assumption of an ideal connection between the structural elements, and unavoidable experimental uncertainties. Fig. 14 presents the time history acceleration records at the bedrock and the free-field soil surface under the influence of the 1940 El Centro earthquake. In this case, the peak ground acceleration for the amplified record is 0.653 g, while the same variable on the bedrock is 0.349 g illustrating the amplification of seismic waves as they propagated in the soft soil deposit from the bedrock to the surface. The natural frequency of the system decreases due to the soil–structure interaction (2.19 Hz for the fixed-base condition, 1.93 Hz for the end-bearing pile foundation, and 1.60 Hz for the shallow foundation case). Therefore, such decreases in the natural frequency (increase in the natural period) alter the response of the building frames under the seismic excitation considerably. This is due to the fact that the natural period of the system lies in the long period region of the response spectrum curve. Therefore,
Fig. 12. 3D numerical predictions of the maximum lateral deformation under the influence of: (a) Northridge earthquake; (b) Kobe earthquake; (c) El Centro earthquake; (d) Hachinohe earthquake.
Fig. 13. Shaking table experimental measurements of the maximum lateral deformations versus 3D numerical predictions for: (a) Northridge earthquake; (b) Kobe earthquake; (c) El Centro earthquake; (d) Hachinohe earthquake.
although the structural demand or the base shear of the structures decreases according to the acceleration response spectrum (Fig. 16b), the total displacement of a system tends to increase. The pile foundations reduce the lateral displacements in comparison to the shallow foundation case since the presence of stiff pile elements in the soft soil increases the equivalent stiffness of the ground and influences dynamic properties of the whole system, such as the natural frequency and damping.

The rocking component plays an important role in the lateral deformation of the superstructure. According to Kramer (1996), the relative lateral structural displacements under the influence of soil–structure interaction consist of a rocking component and a distortion component. In this study, considering the maximum vertical displacement of the foundation, the rocking angles summarised in Table 6, and the maximum lateral displacements reported in Fig. 16, it is noted that for the end-bearing pile foundation cases approximately 20% of the maximum lateral deflections were due to the rocking component, while 80% took place due to the distortion component. These values for the shallow foundation cases are 37% and 63%, respectively. For example, under the influence of the El Centro (1940) earthquake, the maximum lateral deflection at the top of the fixed base model was measured to be 13.63 mm due to the distortion component, while the maximum lateral deflection at the top of the structure supported by the end-bearing pile foundation was 16.12 mm with 3.72 mm of that value being due to the rocking component and 12.4 mm took place due to the distortion component. In the end-bearing pile foundation cases, rocking occurs due to the axial deformation of the pile elements. The area replacement ratio of the pile group is 8% in this study and, as a result, piles attract significant axial forces. However, the rocking of the structure in the shallow foundation case, without pile elements, is clearly much more than the case with pile foundations resulting in further amplification of the lateral deformations.

The corresponding inter-storey drifts of the average values of the 3D numerical model (Fig. 17) have been calculated using the following equation based on the Australian standard (AS1170.4, 2007)

\[
\text{Drift} = \frac{(d_{i+1} - d_i)}{h}
\]

Fig. 14. Time-history acceleration records at top of the 15-storey model structure under the influence of El Centro earthquake for: (a) fixed base structure; (b) structure supported by shallow foundation; (c) structure supported by end-bearing pile foundation.

Fig. 15. Average values of maximum lateral deflections based on shaking table experimental measurements versus 3D numerical predictions.

Fig. 16. (a) Bedrock record and the amplified free field soil surface record under the influence of El Centro earthquake; (b) acceleration response spectrum.
where \( d_{i+1} \) is the deflection at the \((i+1)\) level, \( d_i \) is the deflection at the \(i\) level, and \( h \) is the storey height. In the performance-based design, which is a modern approach to earthquake-resistant design, the seismic performance (performance level) is described by considering the maximum allowable damage state (damage performance) for an identified seismic hazard (hazard level). Inter-storey drifts are the most commonly used damage parameters, and based on FEMA (BSSC, 1997), as discussed in Section 1.1, a maximum inter-storey drift of 1.5% is defined as the border between life safe and near collapse levels. According to Fig. 17, seismic soil–structure interaction tends to increase the inter-storey drifts of the superstructure. The inter-storey drifts of the structure supported by the end-bearing pile foundation are more than the fixed-base conditions excluding soil–structure interaction. However, the structure supported by the end-bearing pile foundation experience more than the fixed-base conditions excluding soil–structure interaction. For example, the maximum recorded inter-storey drift of the fixed-base structure is measured to be 1.48%, while the corresponding value for the pile foundation and shallow foundation cases are 1.7% and 2.25%, respectively. In other words, the effects of soil–pile structure interaction (pile foundation) and soil–structure interaction (shallow foundation) induce increases of 15% and 52% in the recorded inter-storey drifts, respectively. As a result, soil–structure interaction may affect the performance level of a structure and shift the performance level of the structure from life safe zone to near collapse or even collapse levels.

The soil nonlinearity during an earthquake plays an important role in the dynamic response of soil–structure systems. The amplification ratio in the soil medium and its natural frequency change during the shaking excitations in accordance with the developed shear strain level in the soil elements. The generated shear strain at a particular point in the soil medium changes during the excitation. For instance, the developed shear stress and shear strain for the case of the end-bearing pile foundation, recorded at the soil surface below the foundation under the influence of the El Centro earthquake, are presented in Fig. 18.

In the above-mentioned case, point A experiences a shear strain level of up to \(4 \times 10^{-2}\%\) during the excitation. Comparing Figs. 10 and 18, it is evident that the actual secant modulus ratio \((G/G_{\text{max}})\) varies from 1 to 0.7, while the damping ratio varies from 0% to 5% during the applied earthquake in this particular point. Soil elements in different locations experience different shear strains during the earthquake. This highlights the advantage of fully nonlinear models over the equivalent linear methods in capturing the cyclic nonlinear behaviour of the soil more accurately, while in the equivalent linear methods, the strain-dependent modulus and damping functions are only taken into account in an average sense, in order to approximate some effects of the soil nonlinearity (e.g., Fatahi and Tabatabaieifar, 2013; Kramer, 1996).

<table>
<thead>
<tr>
<th>Scaled earthquake acceleration record</th>
<th>Maximum vertical displacement</th>
<th>Rocking angle of the foundation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Fixed base</td>
<td>Shallow foundation</td>
</tr>
<tr>
<td>Northridge</td>
<td>0</td>
<td>2.54</td>
</tr>
<tr>
<td>Kobe</td>
<td>0</td>
<td>1.32</td>
</tr>
<tr>
<td>El Centro</td>
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<td>1.98</td>
</tr>
<tr>
<td>Hachinohe</td>
<td>0</td>
<td>1.47</td>
</tr>
</tbody>
</table>

Fig. 17. Average 3D numerical inter-storey drifts for: (a) fixed-base structure; (b) structure supported by shallow foundation; (c) structure supported by end-bearing pile foundation.

Fig. 18. Developed shear stress versus shear strain in the soil medium at Point A for the case of end-bearing pile foundation record at the soil surface below the foundation under the influence of El Centro earthquake.
5. Conclusions

Employing end-bearing pile foundations is a common practice for transferring structural loads through soft soil to the underlying bedrock or the stiffer layers in order to increase the bearing capacity and to reduce the settlement of the superstructure. In the seismic design of structures supported by end-bearing piles, structural engineers often ignore or simplify the soil–pile–structure interaction and design the structure under the fixed-based condition. In order to assess the accuracy of this assumption and to investigate the effects of soil–pile structure interaction on the seismic response of buildings, a series of shaking table experimental tests has been conducted. A laminar soil container has been designed to simulate the free-shaking table experimental tests has been conducted. A laminar interaction on the seismic response of buildings, a series of fi

soil bearing piles, structural engineers often ignore or simplify the structure. In the seismic design of structures supported by end-bearing capacity and to reduce the settlement of the superstructure in comparison with the fi

foundations, end-bearing pile foundations increase the lateral displacements in comparison to the shallow foundation case due to the rocking fixed-base assumption, and reduce the lateral displacements in comparison with the fi

investigations, it is observed that the shear strain effects of the structures sitting on the end-bearing pile foundations is amplified in comparison to the fixed-base model (17% based on the experimental measurements and 16% based on the 3D numerical predictions). This amplification for the structure sitting on the shallow foundation is even more (55% based on the experimental measurements and 56% based on the 3D numerical predictions). The results comparing different types of foundations, end-bearing pile foundations increase the lateral displacements of the superstructure in comparison with the fixed-base assumption, and reduce the lateral displacements in comparison to the shallow foundation case due to the rocking components.

Consequently, the seismic soil–pile–structure interaction affects the performance level of the structure sitting in soft soil by increasing the inter-storey drifts which may shift the performance level of the structure from life safe to near collapse or even collapse levels. Therefore, the choice of the foundation type is dominant and should be included in investigations of the influence of SSI on the superstructure response during shaking excitations; conventional design procedures excluding soil–structure interaction are not adequate to guarantee the structural safety of moment-resisting buildings resting on soft soil.

References


