AN EXPERIMENTAL INVESTIGATION OF TRANSIENT DYNAMICS OF PILE-SUPPORTED STRUCTURES IN LIQUEFIABLE SOILS

By

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ABSTRACT

Unsatisfactory performance of pile supported structures in liquefiable areas (ranging from tilting/settlement to complete collapse) is still observed after most major earthquakes. As a result, further research is required in this subject. This thesis therefore aims to study the response of pile supported structures during seismic liquefaction. The ground liquefies progressively in a top down fashion when the soil transform from solid material to liquid-like material. This is referred to as transient behaviour (from no-liquefaction to full liquefaction state) and is particularly focused in this work.

In practice, piles are usually analysed as laterally loaded beams using Beam on Nonlinear Winkler Foundation model where earthquake loading is applied in a pseudo-static way. Therefore, this study reviews methods of analysis of laterally loaded pile. Six different field case records were analysed using different approaches and the results were compared.

Large scale shake table experiments were also conducted consisting of four pile models (two single piles and two pile groups of 2×2) placed in a rigid soil container with energy absorbing boundaries. Redhill-110 sand was used and earthquake motions were applied to liquefy the soil. It was observed that the bending moment along the piles changed with the progression of liquefaction and the maximum bending moment occurred in the transient phase. It was also observed that the time taken to reach liquefaction may affect the amplification of the bending moment.

Design of piles requires soil parameters and as a result, a series of multi-stage soil element tests were carried out on four different types of sands; Redhill-110 sand, Japanese silica sand No. 8, Assam sand, and Ganga sand where the sands were first liquefied and then tests were carried out to obtain stress-strain of liquefied sand (post-liquefaction). The results showed that the post liquefaction behaviour of sand depends on the soil relative density. Furthermore, the results from the Redhill-110 sand were used to back analyse the shake table test results. Finally, a method has been proposed to incorporate transient behaviour of pile in liquefiable soils, based on an assessment of the estimated dynamics amplification factors in the shake table tests.

Keywords: Dynamic soil-pile interaction, Liquefaction, Shake table test, multi-stage soil element test, transient dynamics, dynamic amplification factors.
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Declaration

This thesis and the work to which it refers are the results of my own efforts. Any ideas, data, images or text resulting from the work of others (whether published or unpublished) are fully identified as such within the work and attributed to their originator in the text, bibliography or in footnotes. This thesis has not been submitted in whole or in part for any other academic degree or professional qualification. I agree that the University has the right to submit my work to the plagiarism detection service TurnitinUK for originality checks. Whether or not drafts have been so-assessed, the University reserves the right to require an electronic version of the final document (as submitted) for assessment as above.

Mehdi Rouholamin

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Site response analysis results

Appendix – B

Ground motion parameters

Appendix – C

Shake table test results
# Nomenclature

**Roman symbols**

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A$</td>
<td>Skempton pore water pressure parameter</td>
</tr>
<tr>
<td>$AI$</td>
<td>Arias Intensity</td>
</tr>
<tr>
<td>$a_{\text{max}}$</td>
<td>Maximum acceleration of earthquake</td>
</tr>
<tr>
<td>$B$</td>
<td>Skempton pore water pressure parameter</td>
</tr>
<tr>
<td>$B$</td>
<td>Width of pile</td>
</tr>
<tr>
<td>$c$</td>
<td>Soil cohesion</td>
</tr>
<tr>
<td>$C_{u}$</td>
<td>Undrained shear strength</td>
</tr>
<tr>
<td>$D$</td>
<td>Pile diameter</td>
</tr>
<tr>
<td>$D_f$</td>
<td>Depth of fixity</td>
</tr>
<tr>
<td>$D_l$</td>
<td>Depth of liquefaction</td>
</tr>
<tr>
<td>$D_i$</td>
<td>Internal diameter</td>
</tr>
<tr>
<td>$D_o$</td>
<td>External diameter</td>
</tr>
<tr>
<td>$D_{50}$</td>
<td>50% finer size</td>
</tr>
<tr>
<td>$D_r$</td>
<td>Relative density of soil</td>
</tr>
<tr>
<td>$E$</td>
<td>Young’s modulus</td>
</tr>
<tr>
<td>$E$</td>
<td>Energy of earthquake</td>
</tr>
<tr>
<td>$EI$</td>
<td>Modulus of rigidity</td>
</tr>
<tr>
<td>$e_c$</td>
<td>Critical void ratio</td>
</tr>
<tr>
<td>$e_{\text{max}}$</td>
<td>Maximum void ratio</td>
</tr>
<tr>
<td>$e_{\text{min}}$</td>
<td>Minimum void ratio</td>
</tr>
<tr>
<td>$f_n$</td>
<td>Natural frequency</td>
</tr>
<tr>
<td>$g$</td>
<td>Acceleration due to gravity</td>
</tr>
<tr>
<td>$G$</td>
<td>Shear modulus</td>
</tr>
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<td>$G_{\text{max}}$</td>
<td>Maximum shear modulus</td>
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<tr>
<td>$G_s$</td>
<td>Specific gravity</td>
</tr>
<tr>
<td>$G_0$</td>
<td>Initial shear modulus</td>
</tr>
<tr>
<td>$G_1$</td>
<td>Critical state shear modulus</td>
</tr>
<tr>
<td>$H$</td>
<td>Soil profile layer depth</td>
</tr>
<tr>
<td>$H$</td>
<td>Frequency response function</td>
</tr>
<tr>
<td>$H$</td>
<td>Base shear force</td>
</tr>
<tr>
<td>$I$</td>
<td>Moment of inertia</td>
</tr>
<tr>
<td>$k$</td>
<td>Stiffness</td>
</tr>
<tr>
<td>$K$</td>
<td>Bulk modulus</td>
</tr>
<tr>
<td>$K_0$</td>
<td>Lateral earth pressure coefficient</td>
</tr>
<tr>
<td>$k_i$</td>
<td>Terzaghi subgrade modulus</td>
</tr>
<tr>
<td>$k_n$</td>
<td>Normal stiffness of soil</td>
</tr>
<tr>
<td>$k_s$</td>
<td>Shear stiffness of soil</td>
</tr>
<tr>
<td>$L$</td>
<td>Pile embedded length</td>
</tr>
<tr>
<td>$m$</td>
<td>Mass</td>
</tr>
</tbody>
</table>
$m_b$  
Body wave magnitude

$M_c$  
Stress ratio

$M_{JMA}$  
Japanese Meteorological Agency magnitude

$M_k$  
Kawasumi magnitude

$M_L$  
Local magnitude of earthquake

$M_{\text{max-transient}}$  
Maximum bending moment in transient phase

$m_p$  
Reduction factor

$M_p$  
Plastic moment

$M_{\text{post-liq}}$  
Bending moment in post liquefaction

$M_{\text{pre-liq}}$  
Bending moment in pre liquefaction

$M_s$  
Scaling factor for strain

$M_s$  
Surface wave magnitude

$M_w$  
Moment magnitude of earthquake

$n_p$  
Modulus of subgrade reaction

$N_s$  
Scaling factor for stress

$N_{\text{SPT}}$  
SPT blowcount

$p$  
Lateral soil resistance

$p'$  
Mean effective stress

$P_{cr}$  
Critical load

$p'_{\text{ini}}$  
Initial overburden stress

$q$  
Deviator stress

$q_c$  
CPT resistance penetration

$R$  
Stiffness factor

$r_d$  
Stress reduction factor

$r_u$  
Excess pore water pressure ratio

$S_u$  
Residual strength

$S_{xx}$  
Auto spectral density

$S_{xy}$  
Cross spectral density

$t$  
Wall thickness

$T$  
Stiffness factor

$\tau$  
Time period

$T_G$  
Ground time period

$t_{\text{liq}}$  
Time to reach liquefaction

$T_p$  
Predominant period

$T_{\text{post-liq}}$  
Time period in post liquefaction

$T_{\text{pre-liq}}$  
Time period in pre liquefaction

$u$  
Pore water pressure

$V_{\text{liq}}$  
Speed of liquefaction

$V_s$  
Shear wave velocity

$y$  
Lateral soil-pile deflection
Depth of soil layer

Greek symbols

- $\beta$: Critical depth ratio
- $\Delta\sigma_1$: Maximum principal stress increment
- $\Delta\sigma_3$: Minimum principal stress increment
- $\Delta u$: Excess pore water pressure
- $\Delta z_{\text{min}}$: Smallest width of an adjoining zone
- $\varepsilon_a$: Axial strain
- $\gamma$: Shear strain
- $\gamma'$: Unit weight of soil
- $\gamma'$: Effective unit weight
- $\gamma_{\text{post-dilation}}$: Post dilation shear strain
- $\gamma_{\text{to}}$: Take-off shear strain
- $\mu$: Coefficient of friction
- $\eta_1$: Dynamic amplification factor
- $\eta_2$: Dynamic amplification factor
- $\rho$: Material density
- $\sigma$: Normal stress
- $\sigma_c'$: Effective confining stress
- $\sigma_v'$: Effective vertical stress
- $\sigma_y$: Total vertical stress
- $\sigma_y'$: Yield stress
- $\sigma_1'$: Maximum principle effective stress
- $\sigma_2'$: Intermediate principle effective stress
- $\sigma_3'$: Minimum principle effective stress
- $\tau$: Shear stress
- $\tau_{\text{max}}$: Maximum shear stress
- $\nu$: Poisson ratio
- $\varphi$: Friction angle of soil
- $\varphi'$: Effective angle of friction
- $\varphi_{\text{cs}}$: Critical angle of friction
- $\varphi_{\text{cv}}'$: Effective friction angle
- $\psi$: Dilation angle
- $\zeta_{\text{post-liquef}}$: Damping ratio in post liquefaction
- $\zeta_{\text{pre-liquef}}$: Damping ratio in pre liquefaction
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Chapter 1
Introduction

1.1 Introduction

Liquefaction is one of the dramatic phenomenon which may happen during an earthquake in loose to medium dense saturated sand formations. Consequence of liquefaction may result in collapse or severe damages to structures. Severity of the damage depends on a number of factors such as site conditions, earthquake characterises, and the type of structure on the site (Idriss and Boulanger, 2008). The potential of damage that may occur due to liquefaction was first observed following the two main earthquakes which were occurred in 1964 (i.e. the Niigata earthquake in Japan and the Alaska earthquake in the United States). Building failure due to liquefaction was also observed in many past earthquakes such as; San Fernando earthquake (United States, 1971), Kobe earthquake (Japan, 1995), Bhuj earthquake, (India 2001), Kocaeli earthquake (Turkey, 1999), and more recently Wenchuan earthquake (China, 2008), L’Aquila earthquake (Italy, 2009) and Tohoku earthquake (Japan, 2011). Figures 1.1 and 1.2 show two examples of failure due to liquefaction during Niigata earthquake. Figure
1.2 shows Showa Bridge which is a multi-span bridge, collapsed from its middle standing towards the sides like a domino effects. Similar pattern of multi-span bridge failure was also observed during Luzon earthquake (Philippine, 1999) and Maule earthquake (Chile 2010) (Figure 1.3).

Figure 1.1: Tilting of the building due to liquefaction (Niigata earthquake, Japan, 1964).

Figure 1.2: Collapse of Showa Bridge due to liquefaction (Niigata earthquake, Japan, 1964).
Figure 1.3: Bridge failure due to liquefaction during past earthquake: (a) Magsaysay Bridge (Luzon earthquake, Philippine 1999); (b) Puente Viejo (Bio-Bio) Bridge (Maule earthquake, Chile, 2010); and (c) Tubul Bridge (Maule earthquake, Chile, 2010).

There are different factors that affect the liquefaction phenomenon. These factors are: (1) earthquake characteristics such as Peak Ground Acceleration (PGA), duration of earthquake, and frequency content; (2) soil characteristics such as relative density, degree of saturation, and stress history; and (3) site topography (e.g. Level of water table, excess pore water pressure builds up, and patterns of deformation) (Idriss and Boulanger, 2008). The performance of a structure during seismic liquefaction is dependent on many of the factors as described above.

When the soil profile does not have enough resistance in a shallow deep to carry heavy superstructure load, pile foundation (deep foundation) may be used to transmit the loads from the superstructure to lower hard strata. Pile foundations are generally adopted for important and massive structures (e.g. power plants, bridges, dams, offshore structures, heavy oil tanks etc.). As pile foundations are normally surrounded by soil, the behaviour of soil under different loading condition (e.g. earthquake) can affect pile response. This phenomenon was observed during past earthquakes especially in liquefiable soils (i.e. soils which are prone to change their state from solid to liquid due to the sudden loading effect). The current understanding of pile failure identifies few other mechanisms which may control the pile behaviour and needs consideration during the structural design.

1.2 Pile failure due to seismic liquefaction

Pile foundation response under earthquake strongly depends on the pile structure and the soil conditions which surrounding the pile. Based on the literature review, there are two main failure mechanisms of pile foundation during seismic liquefaction; bending mechanism and buckling instability. Figure 1.4 (a, b, c) shows the different stages of loading of a pile-
supported structure during a seismic liquefaction-induced event. Before the earthquake (Figure 1.4a), the axial loads are in equilibrium with the shaft and end-bearing resistance of the piles. As the shaking begins and before the build-up of the excess pore water pressure, piles are mostly loaded by inertia forces generated by the oscillation of the superstructure and the lateral load caused by the soil-pile kinematic interplay (Figure 1.4b). At this stage, the bending mechanism is expected to govern the internal stresses within the pile. Bending failure can occur due to inertia and lateral spreading. Lateral spreading observed in many past earthquakes (e.g., Nügata, 1964; Bhuj, 2001) and happens most likely when the saturated sand locates on a slope. Due to liquefaction the sandy soil layer would flow towards the downslope position which is called “lateral spreading.” This type of failure covers the majority of reported research about pile foundation failure (Hamada, 1992a, b, 2000; Tokimatsu et al., 1996, 1997, 1998; Ishihara, 1997; Finn and Thavaraj, 2001; Finn and Fujita, 2002; Abdoun and Dobry, 2002; Tazoh, 2007; Valsamis et al., 2010; Motamed et al., 2013; Tang et al., 2015; Chen et al., 2015; and Su et al., 2016). However, with the onset of liquefaction with pore water pressure build up (at full liquefaction, the excess pore water pressures reach the overburden vertical effective stress), the soil loses its strength and stiffness, and the pile acts as an unsupported column over the liquefied depth (Figure 1.4c). Piles that have high slenderness ratios will then be prone to buckling instability, which will also be amplified by imperfections, lateral forces and the dynamics of the earthquake. With regards to the second failure mechanism, Bhattacharya (2003), Bhattacharya et al (2004) and Bhattacharya et al. (2005) proposed a new theory of pile failure which is based on buckling instability theory. Specifically, the theory has been formulated by back-analysis of 15 case studies of pile foundation performance and verified by high quality experiments (Dynamic Centrifuge Tests) first by Bhattacharya (2003) and subsequently by Knappet and Madabhushi (2005) and Shanker et al. (2007).
Figure 1.4: Schematic of loading conditions acting on a typical pile-supported structure subjected to seismic induced liquefaction: (a) before earthquake; (b) before liquefaction; (c) at fully liquefaction; and (d) input motion time history and EPWPR (Excess Pore Water Pressure Ratio).

Figure 1.4d shows the time history of a real earthquake together with the site response analysis. From Figure 1.4d, it is clear that it takes time to reach full liquefaction. Also, there are other effects that need to be considered:

(a) At full liquefaction, the period of the structure will increase (i.e. the frequency will decrease) due to flexibility of the foundation;
(b) The transient phase from no–liquefaction to full liquefaction takes some time and the pile will experience bending moments which not only change along the depth but also change with time.

Excess pore water pressure is often increased in loose to medium dense sand layer which is essentially that most likely liquefaction occurs. It has been observed that liquefaction is a top-down phenomena (i.e. liquefaction occurs from a shallow surface to deeper surface due
to the less effective stress in shallow surface). The time taken to reach liquefaction (Figure 1.4d) however, depends on many site characteristics such as ground profile, soil type, and the type of motion. While simplified methods such as Eurocode 8 can express the depth of liquefaction, however, these methods cannot either imply the elongation of liquefaction (i.e. time taken to reach liquefaction) or even whether the soil liquefies fully or partially as well as the rate of liquefaction with the depth. The rate of liquefaction, however, can be presented by the parameter called “$r_u$”. This parameter is described as the ratio of the excess pore water pressure ($\Delta \mu$) and the initial effective confining stress ($\sigma_e'\sigma$). From what currently Eurocode 8 says in this regard, one may obtain the likelihood depth of liquefaction which intrinsically assumed that $r_u = 1$ for the soil (i.e. the ground till this depth is fully liquefied). However, in reality this may be a wrong assumption and in the other words $r_u$ may be 0.8 or 0.69 at the same depth. If the soil is not, for example, clean sand/sandy silt, these methods are not applicable.

Liquefaction phenomenon is dependent on the soil profile characteristics and the earthquake peak ground acceleration. During the liquefaction transient phase (i.e. from pre to full liquefaction), time period of structure increases and the structure becomes more flexible. As a consequence, the maximum bending moment occurs at this phase and the structure may fail or tilt. It seems that time taken to reach liquefaction (i.e. speed of liquefaction) can have an important role on pile foundation failure during seismic liquefaction. As pile and the surrounded soil are in an interaction, it seems that most likely pile failure may affect by the time taken to reach liquefaction. Based on this assumption, a real site with experience of liquefaction in the past earthquakes was chosen for simulation in the Cyclic1D software (nonlinear Finite Element program). The main purpose of using this software (Cyclic1D) is to be able to analyse the timeline of site response under earthquake process. It can also provide information on the time required to reach liquefaction (i.e. whether the soil/ground needed 20sec, 10sec, or 1sec to liquefy). The employed liquefaction model of Parra (1996) and Yang (2000) in Cyclic1D is developed based on the multi-yield-surface plasticity framework (e.g., Prevost 1985) (Cyclic1D, user’s manual, 2012). The Showa Bridge site was chosen due to the observed liquefaction caused by Niigata earthquake (Japan, 1964). Figure 1.5a,b illustrates the soil profile and the liquefaction profile of the site. As shown, the soil profile consists of four different layers which are laid on the engineering seismic base layer. The engineering seismic base layer is also known as seismic bedrock and earthquake motions are applied to this layer (Yoshida, 2015). The engineering seismic base
can be chosen by shear wave velocity as shown in Table 1.1. From the data provided in the table the value of the shear wave velocity increases as the importance of building increases. As Showa Bridge is a type of road bridge, therefore, based on the table, the shear wave velocity for the engineering seismic base layer would be 300~350m/s. The other shear wave velocities for the other layers were calculated based on Standard Penetration Test (SPT) N-value and using the proposed equation for sands by the Japanese Highway Code as given.

\[ V_s = 80N_{SPT}^{1/3} \]  

(1.1)

where \( N_{SPT} \) is the number of blow count in standard SPT test. The average value of the \( N_{SPT} \) for each soil type was considered to be used in the equation. The unit weight of the soil layers (\( \gamma \)) is estimated based on the N-value using Bowles (1996).

<table>
<thead>
<tr>
<th>Design specification</th>
<th>( V_s ) (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Port facility</td>
<td>300</td>
</tr>
<tr>
<td>Road bridge</td>
<td>300~350</td>
</tr>
<tr>
<td>Building and houses</td>
<td>400</td>
</tr>
<tr>
<td>Nuclear power plant</td>
<td>700</td>
</tr>
</tbody>
</table>
In order to perform site response analysis, different types of earthquakes have been chosen from the Pacific Earthquake Engineering Research centre (PEER) website. In order to choose earthquakes, the parameter of source site distance was considered as a constant parameter due to the diversity of the parameters. Therefore, earthquakes having the similar source-site distance around 80km caused by any type of fault movement have been selected. According to the author’s research the best matches of earthquakes were found, based on the considered distance assumption. Also similar Peak Ground Acceleration (PGA) of about 0.3g was considered to apply to the soil profile. Therefore, all of the above earthquakes were scaled to obtain 0.3g as PGA. Table 1.2 lists the applied earthquakes to analyse the considered soil profile in Cyclic1D software and the time taken to reach liquefaction at the first 10 meter obtained from the analyses.

Figure 1.5: (a) Considered soil profile in Cyclic1D, and the soil profile at the site, (b) soil liquefaction profile, (Hamada and O’Rourke, 1992).
### Table 1.2: The applied earthquakes on the considered soil profile in Cyclic1D software

<table>
<thead>
<tr>
<th>Earthquake Location &amp; year</th>
<th>Magnitude (Mw)</th>
<th>Source site distance (km)</th>
<th>PGA (g)</th>
<th>Scaled PGA (g)</th>
<th>Scaling factor</th>
<th>Type of fault</th>
<th>Time to reach liquefaction (sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kern County (Fig. A.1) United State (1952)</td>
<td>7.36</td>
<td>82.19</td>
<td>0.08</td>
<td>0.3</td>
<td>3.75</td>
<td>Reverse</td>
<td>~19</td>
</tr>
<tr>
<td>San Fernando (Fig. A.2) United State (1971)</td>
<td>6.61</td>
<td>89.72</td>
<td>0.02</td>
<td>0.3</td>
<td>15</td>
<td>Reverse</td>
<td>~8</td>
</tr>
<tr>
<td>Friuli (Fig. 1.6) Italy (1976)</td>
<td>6.5</td>
<td>80.41</td>
<td>0.05</td>
<td>0.3</td>
<td>6</td>
<td>Reverse</td>
<td>~10</td>
</tr>
<tr>
<td>Loma Prieta (Fig. A.3) Italy (1989)</td>
<td>6.93</td>
<td>83.45</td>
<td>0.25</td>
<td>0.3</td>
<td>1.2</td>
<td>Reverse oblique</td>
<td>~50</td>
</tr>
<tr>
<td>Christchurch (Fig. A.4) New Zealand (2011)</td>
<td>6.2</td>
<td>84.14</td>
<td>0.07</td>
<td>0.3</td>
<td>4.3</td>
<td>Reverse oblique</td>
<td>~20</td>
</tr>
<tr>
<td>Borah Peak (Fig. A.5) United State (1983)</td>
<td>6.88</td>
<td>82.6</td>
<td>0.02</td>
<td>0.3</td>
<td>15</td>
<td>Normal</td>
<td>~18</td>
</tr>
<tr>
<td>Kozani (Fig. A.6) Greece (1995)</td>
<td>6.4</td>
<td>79.38</td>
<td>0.02</td>
<td>0.3</td>
<td>15</td>
<td>Normal</td>
<td>~7</td>
</tr>
<tr>
<td>Dinar (Fig. A.7) Turkey (1995)</td>
<td>6.4</td>
<td>86.31</td>
<td>0.015</td>
<td>0.3</td>
<td>20</td>
<td>Normal</td>
<td>~30</td>
</tr>
<tr>
<td>Umbria marche (Fig. A.8) Italy (1997)</td>
<td>6.0</td>
<td>83.48</td>
<td>0.004</td>
<td>0.3</td>
<td>75</td>
<td>Normal</td>
<td>~30</td>
</tr>
<tr>
<td>L’Aquila (Fig. 1.7) Italy (2009)</td>
<td>6.3</td>
<td>89.89</td>
<td>0.008</td>
<td>0.3</td>
<td>37.5</td>
<td>Normal</td>
<td>~18</td>
</tr>
<tr>
<td>Southern California (Fig. A.9) United State (1952)</td>
<td>6.0</td>
<td>73.41</td>
<td>0.036</td>
<td>0.3</td>
<td>8.3</td>
<td>Strike slip</td>
<td>~8.5</td>
</tr>
<tr>
<td>Trinidad (Fig. 1.8) United State (1980)</td>
<td>7.2</td>
<td>76.26</td>
<td>0.06</td>
<td>0.3</td>
<td>5</td>
<td>Strike slip</td>
<td>~9.5</td>
</tr>
</tbody>
</table>
The excess pore water pressure caused by each earthquake was measured for 4 different levels of 5, 10, 15, and 20m of the soil profile. Figures 1.6 to 1.8 show the site response for all the cases based on EPWPR (Excess Pore Water Pressure Ratio, \( r_u \)) caused by the earthquakes. These plots represent the site response based on different type of fault rupture. In these figures subplot (a) shows the acceleration time history of the applied earthquake, and subplots (b) to (e) show the time history of EPWPR for different levels of the soil profile. As is shown from the results, the value of EPWPR decreased with depth. However, the increment pattern and duration of EPWPR is different for different earthquake. Also, for a particular site and soil profile, the time taken to reach liquefaction is dependent on the input motion. The rest of the site response analyses are presented in Appendix A.
Figure 1.6: Site response analysis of Friuli earthquake (1976): (a) time history of real earthquake; (b) excess pore water pressure at 20m depth; (c) excess pore water pressure at 15m depth; (d) excess pore water pressure at 10m depth; and (e) excess pore water pressure at 5m depth.
Figure 1.7: Site response analysis of L’Aquila earthquake (2009): (a) time history of real earthquake; (b) excess pore water pressure at 20m depth; (c) excess pore water pressure at 15m depth; (d) excess pore water pressure at 10m depth; and (e) excess pore water pressure at 5m depth.
In practice, pile foundations are usually analysed by Winkler method. In this approach, p-y curves are being employed to represent soil-pile interaction which can be obtained from empirical equation based on stress-strain of soil profile. In the case of liquefiable soil layer, the p-y curves are multiplied by a reduction factor to reduce the stiffness and strength. The liquefied p-y curves, however, can also be obtained from the soil element tests.

Figure 1.8: Site response analysis of Trinidad earthquake (1980): (a) time history of real earthquake; (b) excess pore water pressure at 20m depth; (c) excess pore water pressure at 15m depth; (d) excess pore water pressure at 10m depth; and (e) excess pore water pressure at 5m depth.
In order to estimate the pile response during liquefaction, it is important to have a closed estimate of the maximum bending moment which can occurred in transient phase (from pre to full liquefaction). The design method of pile foundation in seismic liquefaction is explained.

### 1.3 Aims of this research

Failures of pile foundations during seismic liquefaction are still being observed. Therefore, soil-pile interaction in seismic areas needs to be further investigated. The response of the pile under seismic liquefaction can be different, based on the time taken to reach liquefaction. Therefore, the response of pile foundation during shaking was investigated in terms of considering the effect of speed of liquefaction and pile foundation time period change during liquefaction. To better understand and clarify this, a series of scaled model tests using shake table were carried out. The main objectives of the experiments were:

1) to determine the effect of seismic liquefaction on pile foundation time period;
2) to understand how bending moment does change during an earthquake;
3) to examine the effect of time taken to reach liquefaction on pile foundation response.

As pile design method require soil parameters, a series of advanced soil element tests were carried out to investigate the response of different sand under cyclic loading using Cyclic Triaxial test. The main objectives of this series of experiment were:

4) to investigate the cyclic behaviour of different sands;
5) to examine the post liquefaction response of liquefied sands;
6) to obtain p-y curves in order to back analysis of the shake table test.

Moreover, a method to design pile foundation in seismic liquefaction was proposed based on the understanding developed in this research by using Winkler approach. In this approach, Winkler springs were employed in order to represent the soil-pile interaction. Winkler springs are defined by p-y curves which can be obtained from the soil element test.

### 1.4 Overview of this research

This research is written in 9 chapters. After the introduction and background of the subject (i.e. Chapter 1) the rest of the chapters are as follow;
Chapter 2 reviews the literature relating to fault movement, earthquake characteristics, liquefaction, and theory of pile failure mechanism during seismic liquefaction based on lateral spreading and buckling theories. Different methods of laterally loaded pile analysis are also being presented in this chapter. Finally, Beams on Non-Linear Winkler Foundation (also known as “p-y spring” approach) is also expressed and followed by methods of obtaining p-y curves.

Chapter 3 considers six different case studies of laterally loaded pile foundations embedded in different types of soil including uniform and layered soils. These case studies are analysed using different methods; Winkler, Broms, and continuum approach (FLAC\textsuperscript{3D}). The obtained results from these analyses are also compared with the results from the field test.

Chapter 4 presents the shake table experiment. In this chapter the shake table facility at the University of Bristol is described. Also, the methodology of shake table test is explained in terms of test set-up and the material used.

Chapter 5 expresses the results and analysis of the shake table experiment. The response of pile foundation during liquefaction was investigated in this chapter.

Chapter 6 presents the advanced soil element test method. This chapter describes the Cyclic Triaxial apparatus at the University of Surrey. Sample preparation and material used are also explained.

Chapter 7 discusses the results of the soil element test. The response of four different types of sands are monitored under multi-stage soil element test. The post liquefaction response of the sands is investigated in more details. Back analysis of shake table is considered using Winkler method. The proposed p-y curves obtained from the soil element test are used to assign to the Winkler springs.

Chapter 8 proposes a design methodology of pile foundations in seismic liquefaction. The method develops a new design criteria for such pile based on the observations from the shake table experiments. This chapter is also presented an example to show the method.

Chapter 9 summaries the investigated subjects and the obtained output results as well as the potential future of conducting such research topics.
Chapter 2

Literature review

2.1 Introduction

Performance of pile foundations in liquefiable soils during earthquake is still considered as a problem. Since 1964 when the two massive earthquakes occurred in Niigata, (Japan) and Alaska, (US), research on liquefaction started and this is considered as an important factor that affects the foundation behaviour. Previous research shows that the performance of pile foundations depends on factors such as earthquake characteristics (magnitude, duration and frequency), ground profile, time taken to reach full liquefaction (i.e. speed of liquefaction). This chapter reviews the literature related to relevant factors to pile foundation failure during seismic liquefaction such as earthquake and ground characteristics as well as different theories of pile failure in liquefiable soils. Methods of analysing laterally loaded pile are also covered at the end of the chapter.
2.2 Factors affecting pile foundation failure in seismic liquefaction

2.2.1 Earthquake motion

Earthquake is a catastrophic event which can have devastating consequences including collapse of critical infrastructures, buildings and bridges. The performance of structures and infrastructures during earthquakes depends on various factors: earthquake magnitude, duration of earthquake, frequency content and peak ground acceleration. Each of these parameters can affect the performance and hence, engineers need to characterize earthquakes. Some of the important earthquake parameters are defined in Appendix B.

As earthquake occurs as a result of fault movement, different types of fault ruptures are discussed below. Basically, fault is a fracture in a volume of rock. An earthquake occurs when a fault is ruptured. Earthquake effects can be different based on fault movement type. There are different types of fault rupture and the main ones are shown in Figure 2.1. Basically, there are two directions of fault movement; Strike and dip directions. When the earth crust is under tension stress the rupture may occur in dip direction and caused normal fault (Figure 2.1 (b)). Examples of this type of fault movement are Irpinia earthquake (Italy, 1980), Kozani earthquake, (Greece, 1995), Dinar earthquake, (Turkey, 1995), and L’Aquila earthquake, (Italy, 2009). At the same direction (i.e. dip direction), in the compression stress, the earth crust may move upward from the rock fracture and caused reverse fault ((Figure 2.1 (c) and (d)). Examples of this type of fault movement are San Fernando earthquake, (US, 1971), Kern County earthquake, (US, 1952), Friuli earthquake, (Italy, 1976), and Tabas earthquake (Iran, 1978). When the earth crust is under shear stress, the strike-slip fault may occur (e.g. Southern calif, (US, 1952) and Trinidad, (US, 1980)). Figure 2.1 (e) illustrates this type of fault movement. In reality the earth crust may face a combination of rupture direction which can cause oblique fault. For example, Mammoth lakes earthquake, (Greece, 1980) and Corinth earthquake, (Greece, 1981) occurred due to the normal oblique fault, and Christchurch earthquake, (New Zealand, 2011) due to the reverse oblique fault.
Figure 2.1: Fault rupture types: (a) before earthquake; (b) normal fault; (c) reverse fault; (d) thrust fault; and (e) Strike-slip fault.

2.2.1.1 Effect of earthquake motion on liquefied zone

In the past strong earthquakes which occurred in seismically liquefiable areas (e.g. Japan, New Zealand, China, India, Chile, Turkey, and so on) severe damages occurred due to liquefaction. As mentioned earlier, when earthquake occurs in a zone having saturated sand profile, the excess pore water pressure increases dramatically or gradually (based on type of earthquake and soil profile). As a result of pore water pressure generation, the soil effective stress decreases. Therefore, soil loses its shear stiffness and strength and as a consequence large deformation occurs. Figure 2.2 shows two photos of the tilted building and damaged road due to the large deformation caused by liquefaction during Kocaeli earthquake (Turkey, 1999) and Tohoku earthquake (Japan, 2011).
In order to explain the liquefaction phenomenon it might be better to note that the maximum shear modulus and ground time period can be calculated by Equations 2.1 and 2.2 as follows;

\[ G_{\text{max}} = \rho V_s^2 \]  

(2.1)

where, \( G_{\text{max}} \) is the maximum shear modulus in (kPa), \( V_s \) is the shear wave velocity of soil layer in (m/s), and \( \rho \) is soil density in (kN/m\(^3\)).
\[ T_G = \frac{4H}{V_s} \]  \hspace{1cm} (2.2)

where, \( T_G \) is the ground time period in second, and \( H \) is the soil profile layer in meter.

During liquefaction, when the soil effective stress (stress between the soil particles) moves towards zero, the maximum shear modulus \( (G_{\text{max}}) \) decreases. As a consequence, based on Equation 2.1 when the maximum shear modulus decreases, shear wave velocity reduces as well. As a result, by decreasing the shear wave velocity, the ground time period increases according to Equation 2.2. As a consequence, the ground may experience a large deformation due to liquefaction and may increase the probability of matching the predominant time period of earthquake and the time period of the ground and causes resonance phenomenon. This event might be the main reason of structure failure during seismic liquefaction (Towhata, 2008). The flexibility of ground due to liquefaction may depend on the time taken to reach liquefaction due to the fact that liquefaction can happen either slowly or quickly (i.e. speed of liquefaction).

2.2.1.2 Time taken to reach full liquefaction

Time taken to reach liquefaction (liquefaction speed) can play an important role in failure of building during earthquake. Based on the author’s research, the effect of liquefaction speed on structure failure during earthquake has not been considered significantly in the literature. As this parameter can be obtained from the Finite Element (FE) method and site response analysis, therefore, in order to understand the effect of this parameter, Cyclic 1D software (based on FE model) was employed to analyse a real site. The soil profile of Showa Bridge site (with the experience of liquefaction in the past earthquakes) was modelled in the software. The liquefaction model of Parra (1996) and Yang (2000) which is based on the multi-yield-surface plasticity framework (e.g., Prevost 1985) was used to model the liquefiable layers (Cyclic1D, user’s manual, 2012). The soil profile was subjected to different input motions which were chosen based on different types of fault movement. The results of these analyses are presented in Appendix A. From the results, the value of EPWPR (Excess Pore Water Pressure Ratio) decreased with depth. However, the increment pattern and duration of EPWPR is different for different earthquake. Also, for a particular site and soil profile (i.e. Showa Bridge site), the time taken to reach liquefaction is dependent on the input motion.
2.2.2 Soil characteristics

2.2.2.1 Undrained response of sandy soils

When an earthquake happens in a site having loose to medium dense saturated sandy soil strata, the soil layer tends to compact. As a consequence, pore water pressure within the soil layer might increase dramatically followed by loss of the soil effective stress. As a result, the structure of the soil changes from solid to liquid which is so-called liquefaction. Due to the fact that an earthquake is a sudden event which can happen during seconds, therefore there is no enough time for the generated excess pore water to dissipate. It may normally take a few minutes/hours (based on soil profile) for the generated pore water to dissipate in sandy deposit with several meters in thickness. This time is much longer than earthquake duration which is normally between 10-20 seconds (Towhata, 2008). This event represents the undrained response of soil and the liquid soil (solid suspension of sand particles and water) cannot support embedded structures such as pile foundations. Therefore, it might be important to characterise the undrained behaviour of sandy soils.

A sandy soil profile like any soil profile consists of two parts of sand particles and voids which consist of water and air. The strength of soil profile comes from contact force between sand particles which is called “effective stress”. The soil failure is based on the effective stress. Soils like other materials can be failed due to overloading. Basically, there are two different failure criteria for materials; cohesion and friction. These types of failure happen when the Mohr circle reaches an envelope given by Equations 2.3 and 2.4 (Atkinson (2007).

\[ \tau' = c' \]  

(2.3)

where \( c' \) is soil cohesion and \( \tau' \) is shear stress.

\[ \tau' = \sigma' \mu = \sigma' \tan \phi' \]  

(2.4)

where \( \mu \) is coefficient of friction, \( \phi' \) is soil friction angle and \( \sigma' \) is normal stress.

The Mohr–Coulomb criterion is the third criterion of failure and is the summation of cohesion and friction failure criteria. Soil fails when the Mohr circle reaches a line given by the Equation 2.5.
\[ \tau' = c' + \sigma' \tan \phi' \] (2.5)

More details of these failure criteria can be found in Atkinson (2007). Figure 2.3 shows these types of failure criteria.

The undrained behaviour of sandy soil can be different based on the initial state of the soil in terms of void ratio and stress condition. The response of sandy soil under undrained loading is dependent on some factors such as soil bulk/particle density. The soil responses are like compression and dilation for loose/medium and dense sands respectively. When sandy soil is subjected to shear stress, the soil deformation caused by the applied shear stress is developed by a volume change which is called dilatancy. Bishop (1950) expresses the
effects of dilatancy on shear strength of sand. Figure 2.4 illustrates the effective stress between sand particles as well as sand response under undrained loading. Figure 2.5 illustrates the volume and stress-strain response of cohesionless soils.

Figure 2.4: Shear deformation of different types of sand density: (a) normal state; (b) compression state for loose sand; and (c) dilation state for dense sand (redrawn from Budhu, 2011 and Towhata, 2008).
Figure 2.5: Stress-strain and volume response of cohesionless soils.

The undrained response of sand such as stress-strain behaviour is very much dependent on various parameters such as stress history of soil, soil relative density \((D_r)\), and confining effective stress \((\sigma'_c)\). The critical state concept (Schofield and Wroth, 1968) evaluates, in order to consider the combination of the soil relative density and the confining effective stress. The Critical State Line (CSL) implies the failure state of soil. As shown in Figure 2.6, CSL can be considered in stress path graph (i.e. \(p'\)-\(q\)) with the slope of \(M\) which is dependent on the critical friction angle. The effective friction angle \((\phi'_{cv})\) is related to slope of the \(p'\)-\(q\) graph which is \(M_c\). Equation 2.6 expresses this relationship;

\[
M_c = \left(\frac{q}{p'}\right) = \frac{6 \times \sin \phi'_{cv}}{3 - \sin \phi'_{cv}}
\]

(2.6)

where, \(q\) is deviator stress in (kPa) and \(p'\) is mean effective stress in (kPa).
CSL can be also shown in the environment of mean effective stress versus void ratio graph (i.e. e-ln $p'$) with the slope of $\lambda$ (Budhu, 2011). In Figure 2.6, $p'$ and $q$ are the mean effective and deviator stress respectively and $M_c$ and $M_e$ are the ratio of $(\frac{q}{p'})$ in the compression and extension tests, respectively.

The critical state line expresses the condition of sand when sand shears with no further change in its stress and volume (Idriss and Boulanger, 2008). There is another concept which is called “steady state” that was introduced by Poulos (1981). Other researchers such as Vaid et al. (1990), Chu (1995), and Benahmed (2001) suggest that steady state can be employed instead of critical state line. Idriss and Boulanger (2008) imply that these two concepts are essentially synonymous.

![Critical state line](image_url)

Figure 2.6: Critical state line in (a) $p'$-$q$ and (b) log $p'$-$e$ (redrawn from Budhu, 2011).
The undrained shear of sandy soil can be either under monotonic or cyclic loading. Both of these types of loading are useful to understand soil behaviour in static and dynamic states. Each of these loading is explained in following paragraphs.

2.2.2.1 Undrained monotonic loading

Soil samples can be monotonically sheared either under stress or strain control. In undrained monotonic loading of saturated sand, the void ratio of the sample remains unchanged. Therefore, the volumetric strain is near zero. Although the local changes in void ratio might occur, but the void ratio of the sample would be constant (Idriss and Boulanger, 2008).

The undrained behaviour of the sample can be explained by using stress path graph (i.e. $p^\prime$-$q$) which consists of two axes of mean effective stress ($p^\prime$) and deviator stress ($q$). These stresses are defined as given in Equations 2.7 and 2.8:

\[
p^\prime = \frac{\sigma_1^\prime + \sigma_2^\prime + \sigma_3^\prime}{3} \tag{2.7}
\]

\[
q = \sigma_1^\prime - \sigma_3^\prime \tag{2.8}
\]

where, $\sigma_1^\prime$ and $\sigma_3^\prime$ are the maximum and minimum principal effective stress, respectively. In triaxial test it is assumed that the intermediate and the minimum principal stress are the same (i.e. $\sigma_2^\prime = \sigma_3^\prime$).

There has been much research on undrained monotonic behaviour of sands on different parameters of sandy soil such as relative density and confining effective stress. Figure 2.7 shows some results of experiments carried out on Toyoura sand by Ishihara (1993). He presented that the behaviour of saturated sand is dependent on relative density and confining effective stress. As data in Figure 2.7 shows, at the beginning of the undrained monotonic loading, sand tended to compress followed by dilative behaviour. Ishihara et al. (1975), implied the point where the behaviour of sand transfers from contractive to dilative response as a “transformation point”.
2.2.2.1.2 Undrained cyclic loading

Under undrained cyclic loading in saturated sand, loose soil sample tends to contract. Therefore, due to the increases of the pore water pressure \((u)\) and decreases in effective stress \((\sigma')\), normal stress might transfer from the sand particles to the pore water. The plastic volumetric strain caused by the undrained cyclic loading is balanced by an elastic rebound of the sample skeleton due to the reduction of effective stress (Idriss and Boulanger, 2008). As the total stress is the weight of the soil and remains constant, when the pore water pressure starts to increase, the effective stress commences to decrease as the effective stress is calculated by subtracting total stress to pore water pressure (Equation 2.9).

\[
\sigma' = \sigma - u
\] (2.9)

Figure 2.7: Undrained monotonic response of Toyoura sand (Ishihara, 1993).
where, $\sigma$ and $u$ are the total stress and pore water pressure in (kPa) respectively.

In this situation, sand loses its strength and stiffness and becomes softer and softer. As a consequence, sand becomes liquid like material which is called “Liquefaction”.

2.2.2.1.3 Liquefaction

Since 1964 when the two major strong earthquakes happened (i.e. Niigata and Alaska earthquakes), liquefaction has become a significant subject to consider as a remarkable reason for buildings failure. Liquefaction happens when the pore water pressure builds up during earthquake and eventually reaches the initial effective confining stress. As a consequence, large axial strain of about 5% in double amplitude occurs. This phenomenon is called “initial liquefaction” or “liquefaction” (Ishihara, 1996). Towhata, (2008) explains the liquefaction mechanism based on e-log $p'$ graphs drawn by Seed (1979). As shown in Figure 2.8 which illustrates liquefaction mechanism, point A represents the initial state of saturated sand. During shearing, soil tends to compress and its volume reduces and therefore, it moves towards point B. As explained earlier, due to the earthquake total time period, there is no enough time for the generated excess pore water pressure to dissipate. As a consequence, it represents the undrained behaviour of sandy soil. Therefore, the void ratio (i.e. the volume) remains unchanged. By unloading the effective stress the volume contraction (AB) can be removed by swelling (BC). When earthquake happens soil moves straight from point A to C. In the case of negative dilatancy extension, point B might locate further below and the effective stress could be smaller at point C.

![Figure 2.8: Liquefaction mechanism (redrawn from Towhata, 2008).](image-url)
Figure 2.9 shows a typical response of sandy soil during undrained cyclic triaxial test carried out on Gioia Tauro sand by Ghionna and Porcino (2003).

Loose sand under undrained cyclic loading presents softening behaviour. In the case of medium to dense sand, softening behaviour is also seen due to the pore water pressure builds up. However, large deformation may not occur in medium to dense sand because these sands may not lose their strength completely. Hence, in medium to dense sand the initial liquefaction may occur when the pore water pressure is completely generated or
consider the development 5\% of double amplitude axial strain (Ishihara 1993, 1996). Therefore, the onset of liquefaction depends on the soil relative density, \( D_r \). For loose to medium sand, the onset of liquefaction occurs when the condition of zero effective stress was achieved, i.e. the “initial liquefaction” as proposed by Seed and Lee (1966). While for dense sand, the onset of liquefaction is defined as the development of 5\% double amplitude of axial strain (Ishihara 1993, 1996).

There are also two main groups of liquefaction; flow liquefaction and cyclic mobility. Flow liquefaction occurs when the static shear stress is greater than soil shear strength required for liquefied state. Therefore, static shear stress can produce large deformation caused by flow liquefaction. On the other hand, cyclic mobility happens when the static shear stress is less than soil shear strength required for liquefied state. As a consequence, deformation of soil caused by cyclic mobility is produced incrementally by earthquake. The deformation caused by cyclic mobility can be generated either by cyclic or static shear stress (Kramer, 1996).

2.2.2.1.4 Evaluation of liquefaction

As has been observed from the past earthquakes, liquefaction can cause significant damage on structures (e.g. Japan, 2011; New Zealand, 2011; Chile, 2010; Italy, 2009; China, 2008; etc.). Therefore, it is important to evaluate liquefaction phenomenon, in such a seismic areas having saturated sandy profile. There are some factors which might be considered in evaluation of liquefaction. One factor to consider can be having research on fields where liquefaction was observed during past earthquake which can show the potential of soil profile to liquefy. These investigations can be useful to identify a particular area or more general site for the future likely earthquake (Kramer, 1996). Another factor of interest might be the grain size distribution of sand. It is believed that fine sand with uniform grain size has a high potential to liquefy. In comparison, silty material and gravel are less likely to liquefy. In fine loose sand the permeability is low due to the small grain size. Therefore, during earthquake, pore water pressure generates quickly. The initial state of the soil can be another factor which needs to be considered to evaluate liquefaction. This factor could be considered based on the term of “Critical Void Ratio” (Casagrande, 1936). Basically, he carried out a series of triaxial tests on loose and dense sand at different confining stress. At large strain, he found that all the samples reaches the same density and shearing could be continued with constant shearing resistance. The void ratio regarding to the constant density is called Critical Void Ratio (CVR) also known as \( e_c \). Based on these tests, Casagrande drawn CVR line which
is defined a boundary between loose and dense material (Figure 2.10). The material above the line is prone to liquefy during undrained loading.

Figure 2.10: CVR line represents the boundary between (a) the loose and dense sand and (b) the susceptible and non-susceptible of liquefaction (redrawn from Kramer, 1996).

Steady state might be also another factor that can be considered to evaluate liquefaction which was explained in section 2.2.2.1.

There are also some in-situ methods to evaluate liquefaction such as Standard Penetration Test (SPT), Cone Penetration Test (CPT), and in-situ shear wave velocity. In SPT, the sum of the number of blows required for the second and third 150mm of penetration reports as blowcount value \( (N_1)_60 \). Seed and Idriss (1971, 1981, 1982), Seed et al. (1977, 1983), Seed (1979), present the liquefaction evaluation based on \( (N_1)_60 \). CPT which was initially developed in the 1950s in Holland (also known as Dutch Cone Test) is one of the most used site investigation tool. The test consists of pushing an instrumented cone, with the tip facing down, into the ground with a controlled rate (typically 1.5-2.5 cm/s). The resistance to penetration \( (q_c) \) is continuously measured. Seed et al. (1983) and Seed and De Alba (1986), suggested the model which the SPT N-value can be converted to equivalent CPT tip resistance. An empirical method was presented by Stark and Olson (1995) based on tip resistance for liquefaction evaluation.
Measuring shear wave velocity \( (V_s) \) is another in-situ method to evaluate liquefaction. The shear wave velocity can be measured either by drilling a borehole or surface wave measurement (Tokimatsu et al. 1991), and a seismic downhole survey method. Shear wave velocity can be calculated by Equation (2.1) which was already presented in section 2.2.1.1.

The potential of liquefaction can be assessed by calculating the factor of safety against liquefaction (Seed and Idriss, 1971). This factor is the ratio between Cyclic Resistance Ratio (CRR) and Cyclic Stress Ratio (CSR) as given in Equation (2.10). A soil profile accounts as a liquefiable soil if the FOS<1 and soil profile could be safe against liquefaction in the case of FOS>1.

\[
FOS_{\text{lilquefaction}} = \frac{\text{CRR}}{\text{CSR}}
\]  

(2.10)

The cyclic stress ratio for a particular earthquake can be calculated by Equation 2.11.

\[
\text{CSR} = \frac{\tau}{\sigma'_v}
\]  

(2.11)

where, \( \tau \) is the shear stress caused by earthquake in (kPa) and \( \sigma'_v \) is the effective vertical stress in (kPa). As earthquake is an irregular time history, therefore, this irregular time history can be converted to a regular time history (Seed and Idriss, 1982). This equivalent regular time history is equal to 65\% of the maximum shear stress induced by irregular time history. Figure 2.11 illustrates this conversion schematically. They defined the equivalent number of cycles based on earthquake magnitude which are listed in Table 2.1. Seed et al. (1983, 1985) presented the equation for CSR (Equation 2.12) based on the simplified equation suggested by Seed and Idriss (1971).

\[
\text{CSR} = 0.65 \frac{a_{\text{max}}}{g} \frac{\sigma_{sv}}{\sigma'_{sv}} r_j
\]  

(2.12)

where, \( a_{\text{max}} \) is the maximum acceleration of the earthquake (m/s\(^2\)), \( g \) is the acceleration due to gravity (9.81 m/s\(^2\)), \( \sigma_{sv} \) and \( \sigma'_{sv} \) are the total and effective vertical stress in (kPa) respectively, and \( r_j \) is the stress reduction factor which is used for soil flexibility as a function of depth. Youd and Noble (1996) suggested the relationships between \( r_j \) and depth (z) based on different depth:
\[ r_d = 1 - 0.00765z \quad \text{for} \quad z \leq 9.15 \text{ m}; \]
\[ r_d = 1.174 - 0.0267z \quad \text{for} \quad 9.15 < z \leq 23 \text{ m}; \]
\[ r_d = 0.744 - 0.008z \quad \text{for} \quad 23 < z \leq 30 \text{ m}. \]

In the case of soil element test such as Triaxial test, the cyclic stress ratio can be obtained by Equation 2.13.

\[ CSR = \frac{\tau}{2\sigma_v'} \quad (2.13) \]

![Figure 2.11: Schematic equivalent number of cycles; (a) earthquake time history, (b) equivalent number of cycles (Seed and Idriss, 1982).](image)

Table 2.1: Equivalent number of stress cycles (Seed and Idriss, 1982).

<table>
<thead>
<tr>
<th>Earthquake magnitude (Richter scale)</th>
<th>Equivalent number of cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td>8.5</td>
<td>26</td>
</tr>
<tr>
<td>7.5</td>
<td>15</td>
</tr>
<tr>
<td>6.75</td>
<td>10</td>
</tr>
<tr>
<td>6</td>
<td>5-6</td>
</tr>
<tr>
<td>5.25</td>
<td>2-3</td>
</tr>
</tbody>
</table>

The cyclic resistance ratio can be obtained based on SPT N-value for clean and silty sand. As shown in Figure 2.12, there are two separate empirical graphs (for clean sand and silty sand) which show liquefaction potential based for an earthquake with the surface wave magnitude (\(M_s\)) of 7.5 (Eurocode 8, 2003). These graphs are based on CSR and corrected SPT N-value which expresses CRR. For any earthquakes with a magnitude greater than 7.5,
CSR is required to be multiplied by a factor called CM which is listed in Table 2.2 (Eurocode 8, 2003).

![Diagram showing the relationship between cyclic stress ratio and $N_{(60)}$ for $M_s = 7.5$; (a) for clean sands, (b) for silty sands (modified from Eurocode 8, 2003).](image)

<table>
<thead>
<tr>
<th>$M_s$</th>
<th>5.5</th>
<th>6.0</th>
<th>6.5</th>
<th>7.0</th>
<th>8.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>CM</td>
<td>2.86</td>
<td>2.2</td>
<td>1.69</td>
<td>1.30</td>
<td>0.67</td>
</tr>
</tbody>
</table>

**2.2.2.1.5 Post liquefaction**

Liquefiable soil after earthquake may face a massive deformation. After earthquake, soil particles in liquefiable soil starts to re-change their position due to the excess pore water pressure dissipation. This pore water pressure dissipation may cause a massive deformation (i.e. settlement) after earthquake. Hamada et al. (1987) and Seed and Harder (1990), carried out some research about the response of liquefiable soil after earthquake. After earthquake,
the liquefied soil is placed underneath the weight of soil from the upper level or superstructure. The response of the liquefied soil under this load could be dilative (Thomas, 1992; Vaid and Thomas, 1995). As a result, the observed hardening response at large strains can be explained with the dilative response of soil under undrained monotonic shearing. Thomas (1992), Vaid and Sivathayalan (1997) and Koester (1999), have been investigated the post liquefaction behaviour of sand. Based on Thomas (1992), the stress-strain behaviour of sand in post liquefaction is divided into three regions as illustrates in Figure 2.13. According to data in this figure, the first region would start after liquefaction in zero effective stress and consequently shear stiffness. Due to undrained monotonic load the shear stiffness gradually increases by increasing strain. The dramatic parabolic increases in shear stiffness might happen in the second region by increasing the strain. In the third region stress-strain curve is linear which represents the constant shear stiffness.

![Figure 2.13: Post liquefaction stress strain curve proposed by Thomas (1992).](image)

The different shape of post liquefaction stress strain has been presented by different researcher. The two linear stress behaviour of post liquefaction stress-strain curve was proposed by Yasuda et al. (1995). Recently, Dash (2010), proposed a simple post liquefaction stress strain curve as shown in Figure 2.14. He introduced four key parameters to present the post liquefiable stress- strain curve. These parameters are take-off shear strain, initial shear modulus, critical state shear modulus, and maximum shear stress. Each of these
parameters are explained as follow. More information of these parameters can be found in Dash (2010).

**Take-off shear strain** ($\gamma_{to}$): During undrained monotonic loading, the shear strain equivalent to 1kPa of shear stress is called take-off strain.

**Initial shear modulus** ($G_1$): The secant shear modulus in the first section of the post liquefaction stress strain curve is called initial shear modulus.

**Critical state shear modulus** ($G_2$): The tangent shear modulus is relevant to when the soil sheares following the critical state line (the third section of the curve). The tangent shear modulus is approximately constant.

**Maximum shear stress** ($\tau_{max}$): The maximum shear stress can be calculated theoretically. As can be seen in Figure 2.14, there are three states for $\tau_{max}$: (i) possible minimum excess pore water pressure, (ii) minimum non negative pore water pressure during post liquefaction shearing, and (iii) residual strength of soil which is obtained from the back analysis of case studies.

![Figure 2.14: Post liquefaction stress strain curve proposed by Dash (2010).](image)
The maximum shear stress can be theoretically calculated based on three different conditions (Dash, 2010): 

(1) The maximum theoretical maximum shear stress as given by Equation 2.14;

\[
\tau_{\text{max}(1)} = \frac{M_c (p'_{\text{im}} + 100kPa)}{2} \tag{2.14}
\]

where, \(p'_{\text{im}}\) is the initial overburden pressure and \(M_c\) is the stress ratio and based on critical angle of friction \(\phi_{cs}\) as given;

\[
M_c = \frac{6 \sin \phi_{cs}}{3 - \sin \phi_{cs}} \tag{2.15}
\]

(2) The maximum shear stress in condition of non-negative excess pore water pressure can be calculated by Equation 2.16;

\[
\tau_{\text{max}(2)} = \frac{M_c p'_{\text{im}}}{2} \tag{2.16}
\]

(3) The maximum shear stress can be calculated by the residual strength \(s_u\) as given in Equation 2.17. The shear stress can be mobilized when a soil sample was monotonically strained in a very large strain which is called as the residual strength of soil.

\[
\tau_{\text{max}(3)} = s_u \tag{2.17}
\]

Based on the Dash (2010) discussion, \(\tau_{\text{max}(1)}\) is the maximum shear stress which is referred to the possible minimum excess pore water pressure (i.e. absolute vacuum condition). This condition is less likely to happen in real field. The value of \(\tau_{\text{max}(3)}\) comes from back analysis of liquefied soil from the past case studies of flow failure. Therefore, the dilative response has not been considered and is more likely to underestimate the soil strength. This condition can be considered for shallow depth soils with no impermeable top layer such as slope failure during earthquake. \(\tau_{\text{max}(2)}\) can be considered in the deeper depth of strata where the dilative behaviour is more likely to happen due to the undrained boundary condition.

There have been some studies carried out on post liquefaction behaviour and the factors which can affect the post liquefaction behaviour. Based on these research studies, the post liquefaction response can be different by considering various amount of confining effective stress, soil relative density, maximum pre-cycles strain level, and finally type of loading.
Thomas (1992), implies that by increasing the soil relative density, the post liquefaction stiffness increases. Vaid and Thomas (1995), expressed that the by increasing the relative density, the axial strain decreases where the stress strain curve becomes linear. The effect of confining effective stress is another parameter which has been considered on the post liquefaction behaviour. As Thomas (1992), and Vaid and Sivathayalan (1997) showed by increasing the confining effective stress, the stiffness in the linear section of stress strain curve increases. Maximum pre-cycles strain level is another parameter which Thomas (1992) explained its effect on post liquefaction response. Based on his research, this parameter could have some effects on the first section of the stress strain curve (Figure 2.13). The strain in the first region of the curve increases by increasing the maximum pre-cycles strain level. This behaviour represents the weak behaviour of the soil. Thomas (1992) has also discussed about the effect of type of loading on post liquefaction response. He carried out a series of triaxial tests either in extension and compression. Based on his results, the soil showed stronger post liquefaction behaviour in compression triaxial tests than the extension tests. Also Vaid and Sivathayalan (1997) investigated the same parameter by comparing the results between simple shear test and compression triaxial test. They presented that the soil showed stronger post liquefaction response in compression triaxial test than the simple shear test.

2.3 Theories of pile foundation failure in liquefiable soils

For many decades, pile foundations are generally adopted for important and massive structures such as power plants, bridges, dams, offshore structures, and heavy oil tanks, to transmit the loads from the super structure to lower hard strata. The design of pile foundations is very much dependent on the behaviour under lateral loads rising due to variety of sources such as wind forces, wave impact, earthquake shaking, slope failure, and so on. It has been observed that many pile foundations failed during past earthquakes especially in liquefiable soils. There have been many investigative studies carried out on different case studies to understand the failure mechanism of pile foundations. This section reviews the literature on pile foundations failure in liquefiable areas.

2.3.1 Pile foundations response under seismic liquefaction

Even though pile foundations are the embedded structures inside soil, during earthquake incident, these elements (i.e. pile and soil) act together as one interactive system. Therefore, the interaction between pile and soil might be considered to understand the response of pile foundations under earthquake. Pile foundation responses under earthquake strongly depends
on pile structure and soil conditions which surrounding the pile. Based on literature there are two main failure mechanism of pile foundation during seismic liquefaction; bending mechanism and buckling instability. Bending failure can occur due to inertia force and lateral spreading. The movement of the superstructure during earthquake causes inertia force which can also induce bending moments in the pile foundations. JRA (1996, 2002) considers the effect of inertia force from the superstructure on the pile foundations. Lateral spreading observed in many past earthquakes (e.g. Niigata, 1964; Bhuj, 2001). This phenomenon occurs when a saturated sand layer locates on a slope formation. Due to liquefaction the sandy soil layer could flow towards the downslope position which is called “lateral spreading”. Figure 2.15 schematically illustrates pile foundation failure due to lateral spreading. This mechanism is based on kinematic bending failure process. This type of failure covers the majority of reported research about pile foundation failure (Hamada, 1992a,b, 2000; Tokimatsu et al., 1996, 1997, 1998; Ishihara, 1997; Finn and Thavaraj, 2001; Finn and Fujita, 2002; Abdoun and Dobry, 2002; Tazoh, 2007; Motamed and Towhata, 2010; Wang and Orense, 2014; Tang et al., 2015; Chen et al. 2015; and Su et al. 2016).

Figure 2.15: Bending failure mechanism of pile foundation under liquefaction (redrawn from Bhattacharya and Madabhushi, 2008).
Ishihara (1997) and Tokimatsu et al. (1998) presented two theories of pile failure based on bending mechanism. In 1997, Ishihara implied two concepts of pile failure called “top-down effect” and “bottom-up effect.” These concepts are explained further as follow.

**Top-down effect**

During earthquake, the shear force coming from the inertia force of the superstructure could transfer to the top of the pile foundation and then to the soil surrounding the pile (i.e. top-down effect). The significant movement of ground can then cause bending moment in the pile foundation. If the bending moment exceed the bending capacity, the pile might be potentially failed.

**Bottom-up effect**

When the ground where pile foundation locates is on slope formation, during liquefaction, it might move horizontally and causes lateral force. This lateral force will apply to the pile foundation. As a result, the pile foundation could move towards the slope direction. Ishihara (1997) assumed that the earthquake motion at such condition has already passed the peak. However, the shaking may still have lesser intensity. As a consequence, the inertia force of superstructure might be relatively small. Therefore, the location of the maximum bending moment may be at the lower position rather than the pile head (i.e. bottom-up effect).

Tokimatsu et al. (1998), explains another theory of pile failure. Based on his theory, the inertia force from the superstructure may increase before the development of pore water pressure. By increasing the pore water pressure, kinematic forces (generated by the liquefied soil) apply to the pile. At the end of shaking process, this kinematic force, with a dominate role, affects on the pile foundation which can be significant, if permanent movement happens in the lateral spreading phenomenon.

Showa Bridge is one of the well-known case studies and has been investigated for liquefaction studies for several decades. For many years it was believed that lateral spreading is the major cause of pile foundations failure of Showa Bridge during earthquakes. However, Bhattacharya (2003), Bhattacharya et al. (2004, 2005), and Bhattacharya and Madabhushi (2008), suggested an alternative failure mechanism of pile foundations for the bridge during earthquake, called “buckling instability”. They indicated that on lateral spreading mechanism, the axial load pressure applied on the top of the pile foundations has been ignored. However, during seismic liquefaction this axial load can play an important role on identifying type of
pile failure. As soil liquefies, it loses its strength and stiffness and becomes liquid state and cannot support the pile foundations anymore. The pile foundations embedded in such a soil destabilized condition becomes like a cantilever. Therefore, if the applied axial load exceeds the critical load of pile, pile may fail due to buckling instability. Figure 2.16c shows buckling failure of pile foundations schematically.

2.3.2 Effect of dynamics on pile response

Pile foundations can be failed due to shearing, bending, buckling, and dynamic process effects. In the first three failure mechanisms it is assumed that the loads are pseudo-static in nature. However, during an earthquake, an additional stresses may be generated in the pile due to the dynamic processes and properties of building and soil. Dynamic properties of structures and soils can be changed as a function of input motion characteristics such as magnitude, duration and time period. Therefore, pile foundations will experience additional dynamic forces due to earthquake and change in the dynamic property of the structure and surrounded soil. Thus, this dynamic failure of piles should not be ignored in the design.
process. Current codes of practice for pile foundations (e.g. JRA and Eurocode) are focused, however, only on the bending failure where the lateral loads induce bending stress in the pile. Recently, Bhattacharya et al. (2009) and Lombardi and Bhattacharya (2014), focused on the natural frequency of structures during liquefaction and were able to show that the time period of structure increases during liquefaction and the structure becomes more flexible. The research study on dynamic failure of pile foundations are very few and more research studies are needed to understand the effect of dynamics on pile behaviour.

2.4 Methods of analysing laterally loaded piles

Studies on pile foundation may broadly be divided into numerical studies and experimental studies (such as full-scale and small-scale tests). The numerical studies of laterally loaded piles, however, can majorly be classified in the following methodologies; the elastic continuum approach (Poulos 1971, Pise 1982), finite element approach (Randolph 1981), elastic subgrade reaction approach (Hetenyi, 1946; Reese and Matlock, 1956; Davisson and Gill, 1963), and \( p-y \) curve approach (Matlock, 1970; Reese and Welch, 1975). Poulos and Davis (1980) studied laterally loaded piles using experimental and finite difference method of numerical analyses. Yang and Jeremic (2002) also studied pile and pile groups under pure lateral loads by numerical methods of analyses. Papadopoulou and Comodromous (2010), used finite difference analysis to examine the response of pile under lateral loading. Three-dimensional continuum approach has been developed to consider soil medium as continuum. Though much research has been considered based on continuum approach (e.g. Bentley & El Naggar, 2000; Wu and Finn, 1997; Trochanis et al., 1991; Sarkar and Maheshwari, 2012) but still up to date empirical approaches (Winkler approach or Broms’ method) are the most common methods used for analysis and design of pile foundations under lateral loads specially in practical point of view.

2.4.1 Winkler method (Non-liquefiable and liquefiable soil)

In this method of analysis, the pile is idealised as an elastic beam supported by a series of discrete non-linear springs. The stiffness of each spring is non-interactive and non-linear represented by the lateral soil resistance \( (p) \) which is non-linear with the lateral deflection of pile \( (y) \). Analysing the soil-pile system with soil represented by the nonlinear soil springs \( (p-y \) curves), the design outputs (pile deflection, rotation, bending moment, shear and soil reaction) are obtained very conveniently. This analysis approach is often termed as
“Displacement based analyses” and as such it can be applied to the serviceability limit state design of piles under lateral loading. The fundamental works from which the p-y curve method was later developed include the Winkler soil idealisation, and the subgrade reaction approach by Terzaghi (1955) and later by Reese and Matlock (1956). Reese accredits McClelland and Focht (1958) as originally deriving the concept of p-y curves. API (2000) has postulated the formulas to obtain the p-y curves for clays as well as sands based on the modulus of subgrade reaction. The lateral springs resist lateral loads and displacements. Using the Winkler idealisation the stiffness of a particular spring is the applied force on the spring with respect to the spring deflection it causes. The spring stiffness coefficient is synonymous with the coefficient for the soil stiffness; the coefficient of subgrade reaction. A series of p-y curves can be produced for a series of springs along the pile length (Figure 2.17). In Winkler method (Beam on Non-linear Winkler Foundation) of analysis of piles, the pile-soil interactions are represented by a set of nonlinear soil springs: p-y springs (commonly known as curves incorporate the lateral pile-soil interaction), t-z springs (models the shaft resistance, i.e., pile-soil friction) and q-z spring (models the end-bearing interaction). Figure 2.17 shows a simple model of a pile which can be analysed using any standard structural software and can incorporate advanced features such as P-delta effects, non-linearity in the material of the pile. For any load or displacement applied to the pile either at the pile head (represents inertia load from the superstructure) or along the pile, the required analysis outputs are pile deflection, rotation, bending moment, shear and soil reaction. However, undoubtedly the critical inputs for a realistic analysis are the springs which represent the interactions. p-y springs are generally constructed using a set of scaling rules as prescribed by codes of practice and necessary input parameters are obtained from stress-strain of the soil. The Winkler idealisation does not represent a continuum and, therefore, the p-y curve for a particular depth is independent from shear stresses above or below that depth, and from the shape or stiffness of the pile.
Obtaining p-y curve

p-y data is essentially the load deflection response of a soil and it can be seen that p-y curves are strongly comparable with soil test stress-strain curves.

The most rigorous method of deriving accurate p-y curves for a soil profile is to actually test full-scale instrumented piles. The instrumented pile is lined with strain gauges and the measured strain data can be used to find the bending moment at those points where the gauges are situated. The variation of bending moment with depth is integrated twice to give the deflection \( (y) \), and differentiated twice to give lateral force per unit length \( (p) \). A number of case studies were conducted with instrumented piles and standard methods for generating predicted p-y curves from commonly found parameters were derived. The parameters required to construct predicted p-y curves using the recommended methods are: for soft clay and stiff clay conditions; unit weight, undrained shear strength, and \( \varepsilon_{50} \) (strain which occurs at 50% of the failure stress in an undrained laboratory compression test).
Meanwhile for sands condition; unit weight and the effective angle of fiction ($\phi'$) are adequate.

In practice, p-y curves are normally obtained from codes of practice, see for example API (2000) and the input required is the stress strain of the soil. Figure 2.18 shows a typical stress-strain of sand and a typical p-y curve for sand. Similarly, Figure 2.19 shows stress-strain of a typical clay soil along with p-y curves for clay. An interesting feature may be observed that the shape of the p-y curve for sand and clay is similar to their stress-strain behaviour and more detailed of reasoning behind these similarities is being explored by Bouzid et al (2013).

Figure 2.18: Typical stress-strain curve for Quartz sand (Wichtmann, 2005) and API p-y curve.

Figure 2.19: Typical stress-strain curve for Ariake clay (Chai et al., 2007) and API p-y curve.
**Current p-y curves for liquefied soil**

There are no standard p-y curves for liquefied soils and often a reduction factor is normally used to obtain empirical p-y curve for a specific liquefied soil from its non-liquefied counterpart. In this method, both the stiffness and strength of a non-liquefied soil is multiplied by a factor known as "p-multiplier, \( p_m \)" and Figure 2.20 shows the shapes and it is more reasonable to name this "empirically obtained p-y curves". From Figure 2.20, it may be noted that the empirically based p-y curve for liquefiable soils does have an initial stiffness which is denoted by \( k_2 \) shown in the diagram.

Different values are also proposed for obtaining p-multipliers based on \( N_{SPT} \) (see, for example Table 2.3 and Figure 2.21). Table 2.3 is based on Brandenberg (2005) whereby p-multiplier for liquefiable soils is obtained for a corresponding Standard Penetration Test (SPT) which in turn can be linked to relative density of a sandy soil. Figure 2.21a on the other hand is based on excess pore water pressure ratio (degree of liquefaction) and Figure 2.21b is a collation of other proposed p-multiplier.

![Figure 2.20: Types of p-y curves.](image-url)
Table 2.3: Suggested value of \( m_p \) to obtain liquefiable soils p-y curve (Brandenberg, 2005)

<table>
<thead>
<tr>
<th>((N)_{80})</th>
<th>(\text{p-multiplier (m}_p)</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;8</td>
<td>0.0 to 0.1</td>
</tr>
<tr>
<td>8-16</td>
<td>0.1 to 20</td>
</tr>
<tr>
<td>16-24</td>
<td>0.2 to 0.3</td>
</tr>
<tr>
<td>&gt;24</td>
<td>0.3 to 0.5</td>
</tr>
</tbody>
</table>

Figure 2.21: p-multiplier suggested by (a) Dobry et al. 1995 and (b) modified from Brandenberg et al., 2007.

Figure 2.22 shows a typical stress-strain curve for liquefied Nanjing sand obtained from hollow cylinder apparatus (Pan et al, 2011). It may be noted that there is a zone of zero-stiffness at small strains and after a threshold strain (which is later termed as take-off strain) there is strain hardening behaviour of the soil. Similar observations have also been reported by other researchers using cyclic triaxial apparatus on other types of sands (Vaid and Thomas, 1995: Yasuda et al, 1995).

Naturally, one may expect to see the shape of the p-y curve for liquefied soil to follow the stress-strain curve. However, comparing Figure 2.20 and Figure 2.22, it is clear that the shape of stress-strain curve for liquefied sands is different from the empirically obtained p-y curves. This observation, however, calls for further research.
2.4.2 Broms’ method

This proposed method by Broms (1964a, 1964b) is actually based on the previous work of Brinch Hansen (1961) and Matlock and Reese (1960). In this method, the pile is assumed to be rigid and thus, a solution is found by use of the equations of static for the distribution of ultimate resistance of the soil that puts the pile in equilibrium. This method can be used to compute the ultimate loading of a pile of particular dimensions. Broms has categorized piles as short and long pile with the criteria based on stiffness factors which has different expressions for normally consolidated clays, sands and stiff over consolidated clays respectively. Tables 2.4 and 2.5 summarise different empirical equations suggested by Broms (1964a, 1964b).

Cohesive soil

According to Broms’ model of piles in cohesive soil, the soil’s reaction is assumed to be equal to zero to a depth of $1.5D$, then $9C_u$ for below this depth, where, $C_u$ is the undrained shear strength of the soil and $D$ is the diameter of the pile. However, it is understandable that there will be some soil reaction from ground level. Therefore, based on the work of Brinch-Hansen (1961), it was empirically deduced that at ground level, there is a soil reaction of $2C_u$ which increases to a value of $8 - 12C_u$ at a depth of $3D$ beyond which the reaction remains constant. The value of constant reaction is determined by calculating the ultimate
lateral resistance as a function of the shape at the cross sectional area and the roughness of the pile.

**Free-headed short pile**

The infinitely stiff nature of short piles means failure under lateral loading will be a result of the soil yielding along the total length of the pile, and the pile rotates as a unit around a point located at some depth below the ground surface (as can be seen in the deflected shape in Table 2.4). The maximum positive moment occurs at the level where the shear force along the pile equals zero and this is labelled $f$ in Table 2.4.

**Free-headed long pile**

Long piles are more flexible and therefore more likely to deflect. As a result, failure will generally occur due to a plastic hinge forming like the one demonstrated in deflected shape of Table 2.4 at the depth where the moment developed due to lateral loading equals or exceeds the moment of resistance of the pile section. It has been assumed that the lateral deflections are large enough to develop the full passive resistance of the soil down to the depth corresponding to the location of zero bending moment.

**Fixed-headed short pile**

In this case due to the stiffness of the pile section, failure is most likely to take place in the soil, mainly due to the applied lateral load equalling or exceeding the ultimate lateral resistance of the soil, causing the pile to move as a unit through the soil (as illustrated in Table 2.4). The depth of the maximum moment will be the same as a free headed pile but the value of the maximum moment is assumed as half the value of the free headed version.

**Fixed-headed intermediate piles**

For a restrained intermediate pile, failure happens when the maximum moment due to lateral loading of the pile equals the ultimate moment resistance of the pile section and the pile rotates around a point located at some depth below the ground surface. The maximum moment could be the maximum negative (restraining) moment at the head of the pile or maximum positive moment located at a depth below the ground surface determined from the requirement that the shear force along the pile section equals zero. It is assumed that at ultimate state, the maximum positive and negative moments are equal and like is the equivalent of half the value of a similar free-headed pile.
**Fixed-headed long pile**

In the case of long restrained piles, failure takes place when two plastic hinges form along the length of the pile. Hinges are most prone to directly below the pile cap where the negative moment is at maximum and the section where the positive moment is greatest, most likely below the ground surface where shear equals zero. The maximum bending moment also needs to be greater than or equal to the ultimate moment resistance of the pile section. Table 2.4 demonstrates the likely deflected shape at failure and the assumed soil reaction.

<table>
<thead>
<tr>
<th>Table 2.4: Suggested empirical equation by Broms (1964a) in cohesive soil</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Pile type</strong></td>
</tr>
<tr>
<td>----------------</td>
</tr>
<tr>
<td>Free headed short pile</td>
</tr>
<tr>
<td>Free headed long pile</td>
</tr>
<tr>
<td>Fixed headed short pile</td>
</tr>
<tr>
<td>Fixed headed intermediate pile</td>
</tr>
<tr>
<td>Fixed headed long pile</td>
</tr>
</tbody>
</table>
Cohesionless soil

Laterally loaded piles in cohesionless soil are assumed to display soil reactions reflective of passive resistance. Based on this observation, the soil reaction calculated is dependent on the Rankine’s coefficient of passive resistance of the soil and a safety factor of three is being considered.

Free-headed short pile

For a short pile like shown in Table 2.5, the nature of failure will depend on the depth of embedment and the degree of end restraint. For a free-headed pile, failure takes place when the soil yields and the pile rotates as a unit around a point located below the ground surface.

Free-headed long pile

For a free-headed long pile as shown in Table 2.5, failure occurs when a plastic hinge forms at some distance from ground surface where the bending moment along the pile length (as a result of the applied lateral load) equals or exceed the ultimate or yield resistance of the pile section. Like piles in cohesive soil, it is assumed that the passive lateral earth pressure develops from the ground surface beyond the maximum moment and down to the level where moment equals zero.

Fixed-headed short pile

Short restrained pile fails quite similarly to a short restrained pile in cohesive soils and takes place when the load applied to the pile is equal to the ultimate lateral resistance of the pile resulting in movement as a unit through the soil. The behaviour of this pile under lateral loading is like a short pile in cohesive soil except for the soil reaction and the resulting bending moment value (Table 2.5).

Fixed-headed intermediate pile

The failure pattern of the intermediate pile as can be found in Table 2.5 is quite similar to that of an intermediate pile in cohesive soil. However, the resulting soil reaction is different and subsequently the bending moment also differs.

Fixed-headed long pile

Like the short and intermediate piles mentioned above, the failure pattern of the long restrained pile is quite similar to a long restrained pile in cohesive soils and the soil reaction
is different and depends on the effective density of the soil as well as its other contributing properties (Table 2.5).

**Table 2.5: Suggested empirical equation by Broms (1964b) in cohesionless soil**

<table>
<thead>
<tr>
<th>Cohesionless soil (Broms, 1964b)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Pile type</strong></td>
</tr>
<tr>
<td>Free headed short pile</td>
</tr>
<tr>
<td>Free headed long pile</td>
</tr>
<tr>
<td>Fixed headed short pile</td>
</tr>
<tr>
<td>Fixed headed intermediate pile</td>
</tr>
<tr>
<td>Fixed headed long pile</td>
</tr>
</tbody>
</table>
2.4.3 Continuum method

Laterally loaded pile can be analysed by modelling the pile and the soil as a continuum model system. Laterally loaded piles may be analysed by two widely accepted continuum approaches:

- Finite Element Method (FEM) approach
- Finite Difference Method (FDM) approach

These approaches are widely used to investigate the interaction between the pile and its surrounding soil. Modelling pile and soil as a continuum system developed as close-form solution based on linear elasticity. The lateral pile deflection with depth can be analysed by using method proposed by Basu et al. (2009). One of the first research that used the method was suggested by Poulos (1971). In this method, the soil is modelled as an elastic continuum and the pile as a strip that applied pressure on the continuum. Today, by improving and developing new software, the most continuum model method being used are the FEM and FDM methods. Three dimensional interactions between pile and the soil and also linear and non-linear behaviour of soils can be analysed by these approaches (Busu et al., 2008). Numerical methods (e.g., 2D and 3D finite element, finite element with Fourier analysis, and finite difference) are required to analyse soil as a continuum (Basu et al., 2009). There has been much research on continuum approach which were carried out using different numerical techniques such as Poulos, 1971a, 1971b; Banerjee & Davis, 1978; Randolph, 1981; Budhu & Davies, 1988; Brown et al., 1989; Verruijt & Kooijman, 1989; Trochanis et al., 1991; Bransby, 1999; Ng & Zhang, 2001; Klar & Frydman, 2002. Despite the fact that continuum method is one of the more accurate method of analysis, it is less popular method due to the time and expertise required.

2.5 Conclusions

This chapter provides a thorough review of the past research on pile foundation failure in seismic liquefiable areas. The current understanding of pile failure is based on bending and buckling mechanisms where dynamics of the whole problem is ignored due to complexity. Therefore, the effect of earthquake motion on pile response as the soil liquefies progressively needs to be studied to apprehend whether or not ignoring dynamics is conservative. With soil liquefaction, the dynamics characteristics (Time period, $T$, and damping ratio, $\zeta$) of the
whole structure will change. The effect of time taken to reach full liquefaction and its impact on pile response has not been studied. This is therefore the focus of this study.
Chapter 3
Laterally loaded pile analysis

3.1 Introduction

Pile foundations have generally been adopted for important and massive structures (e.g. power plants, bridges, dams, offshore structures, heavy oil tanks etc.) to transmit the loads from the superstructure to lower hard strata. The design of pile foundations is very much dependent on the behaviour of the pile under lateral loads rising due to variety of source parameters such as wind forces, wave impact, earthquake shaking, slope failure etc. In the case of lateral loaded piles, the main design issue of concern is with the excessive lateral deflection; the yielding or complete failure of the pile through the development of one or more hinges or yielding of the soil causing the pile to move as a unit through the soil. The main consideration in recent performance based design is to control deformations (displacements or settlements of the foundations).

Recently, empirical approaches (such as Winkler approach or Broms’ method) are most popular methods for analysis and design of pile foundations under lateral loads.
Sometimes, these empirical approaches may yield very dangerous design in terms of
dynamic behaviour of soil-pile system. Ideally, full-scale testing shall be carried out before
taking up design of pile under lateral loading. While the use of full scale/small scale testing
would help the design process immensely, its use is limited due to its time consuming and
cost-effectiveness. Though many numerical and experimental investigations have been
carried out on this issue, comparison between the field-test results with the numerical
analyses are rare in the literature.

3.2 Objective of the analysis

Up to now, lateral load carrying piles have been analysed and designed based on the
empirical discrete approaches. In order to compare the various number of numerical
methods, six different case studies of laterally loaded pile are analysed by using two well-
known empirical methods (i.e. Winkler and Broms’ approach), and also with continuum
method using FLAC\textsuperscript{3D} software. The numerical analyses results were compared with each
other as well as with the field test results.

3.3 Methods of laterally loaded pile analysis

Laterally loaded pile can be analysed by using different numerical methods. These
methods can be classified into three groups; i) advanced method (continuum approach), ii)
standard method (Winkler approach), and iii) simplified method (Broms’ method). Today,
many complicated models (e.g., pile-soil interaction, interaction between tunnels and pile,
slope stability, and so on) can be modelled and analysed either with finite element or finite
difference methods. In this study, however, finite difference method of analysis was
adopted. Software package FLAC\textsuperscript{3D} (Fast Lagrangian Analysis of Continua in 3
Dimensions, Itasca Consulting Group, Inc. 2014) was used for modelling and analysing the
soil-pile system considering the soil medium as continuum.

Despite the fact that continuum approach is able to analyse many problems, however,
Winkler and Broms’ methods are more popular in practical sense. This may be because of
the more efficient steps and time needed to perform analysis by the continuum method.
The three methods are explained briefly in follow paragraphs.
3.3.1 Advanced approach (Continuum method) - FLAC\textsuperscript{3D} software

FLAC\textsuperscript{3D} is a geotechnical software based on Finite Difference Method (FDM) and this software is used to analyse six laterally loaded pile case studies. This section provides further detail of FLAC\textsuperscript{3D} modelling.

The FLAC\textsuperscript{3D} analysis consists of soil mesh generation, pile mesh generation and installation, boundary conditions, gravity and lateral loading. Any type of model can be created in FLAC\textsuperscript{3D} by generating meshes. There are varied number of pre-defined mesh shapes in the software to generate a zone. In this study, radially graded mesh around cylindrical-shaped tunnel was used to generate the soil medium and cylindrical-shaped mesh was then employed to model pile foundation geometry. A typical shape of these meshes is shown in Figure 3.1.

Once the zone has been generated (Figure 3.2) and the soil material has been defined, the zone was then analysed to obtain an equilibrium stress-strain under gravitational load. In this stage the maximum unbalanced force (i.e. the nodal force vector) would be decreased. In the numerical analysis, the maximum unbalanced force might never reach zero. However, the model is considered to be in equilibrium once the maximum unbalanced force is smaller than the total applied forces in the model (Itasca, 2014). Figure 3.3 shows the obtained unbalanced force from the analysis.
The next step is to generate pile mesh which was then created separately (Figure 3.4a) and attached to the soil (Figure 3.4b). Next, the model was re-analysed to obtain an equilibrium stress-strain after installing pile. In this analyse soil material would be replaced by pile material. As soil and pile are defined with different materials, a layer of interface was created between the soil and pile by using cylindrical-shaped tunnel mesh (Figure 3.1b). The interfaces were then installed between the pile wall and at the pile tip and the soil. Interface properties were defined by the following properties; shear stiffness ($k_s$), normal stiffness ($k_n$), cohesion ($c$) and friction ($\varphi$). The normal and shear stiffness are assumed to be ten times of the stiffest neighbouring element (Itasca 2014). The value of normal stiffness can be calculated by Equation (3.1).
\[ k_n = \left[ \frac{K + \frac{3}{4} G}{\Delta z_{\text{min}}} \right] \times 10 \]  

(3.1)

where, \( K \) and \( G \) are bulk and shear modulus of the soil medium, respectively, and \( \Delta z_{\text{min}} \) is the smallest width of an adjoining zone in the normal direction.

Figure 3.4: soil-pile model in FLAC\(^{3D}\) (a) pile is created separately (b) pile is installed inside the soil.

Since the lateral extent of the soil medium largely influences the lateral load–deflection behaviour of the soil-pile system, the boundary should be far away to avoid errors resulting by the implication of boundaries. This implies an important finding that the boundaries shall be infinitely away from the pile. This enhances the computational cost. Therefore, the lateral boundaries have been extended based on some trial analyses. A distance of 30D (D
is pile diameter) is adopted for the lateral extent of the boundary and distance equal to 7D (D is pile diameter) was selected for the bottom boundary. As shown in Figure 3.5, only one half of the model was modelled due to the symmetry of the model.

![Figure 3.5: A plan view of the model](image)

The Mohr-Coulomb failure criterion as implemented in FLAC$^3D$ has also been adopted for the study. The simplest form of Mohr’s envelope, is the linear relation between shear stress ($\tau$) and normal stress ($\sigma$).

$$\tau = c + \sigma \tan \phi$$

(3.2)

The constants $c$ and $\phi$ are the cohesion and angle of internal friction, respectively. According to this criterion, material will fail for all states of stress for which the largest of the Mohr circles is just tangent to the envelope. The concept of Mohr circle can be used to express the criterion in terms of principal stresses as expressed.

$$\left(\frac{\sigma_1 - \sigma_3}{2}\right) = \left(\frac{\sigma_1 + \sigma_3}{2}\right) \sin \phi + c \cos \phi$$

(3.3)

where $\sigma_1$ and $\sigma_3$ are major and minor principal stresses, respectively.

The yield surface of Mohr-Coulomb failure criterion represents an irregular hexagonal pyramid in the stress space as shown in Figure 3.6. In addition to the yield functions, plastic potential functions are defined for the Mohr-Coulomb model. The plastic potential
functions contain a third plasticity parameter, the dilation angle $\psi$. This parameter is required to model positive plastic volumetric strain increments (dilatancy) as actually observed for dense soils.

The parameters might be considered as inputs for Mohr-Coulomb model in FLAC$^{3D}$ are; bulk modulus ($K$), shear modulus ($G$), friction angle ($\phi$), cohesion of soil ($c$), and dilation angle ($\psi$). It is to be noted that the elastic parameters constants, $K$ (Bulk modulus) and $G$ (Shear modulus), are used in FLAC$^{3D}$ rather than Young’s modulus ($E$), and Poisson’s ratio ($\nu$), because it is believed that bulk and shear modulus correspond to more-fundamental aspects of material behaviour than young’s modulus and Poisson’s ratio (Itasca, 2014). These parameters were calculated from the following equations.

\[
K = \frac{E}{3(1-2\nu)} \quad (3.4)
\]

\[
G = \frac{E}{2(1+\nu)} \quad (3.5)
\]

By having the unit weight ($\gamma$) and the frictional angle ($\phi$) or cohesion of soil ($c$), the elasticity modulus of soil can be estimated by some correlations presented in Bowles (1996). Another parameter that should be considered is dilation angle ($\psi$) which controls plastic volumetric strain that develops during plastic shearing. This parameter is also

Figure 3.6: Mohr-Coulomb yield surface in principal stress space (for $c=0$).
assumed constant during plastic yielding. The dilation angle was therefore obtained from Rowe equation, as follows:

\[ \sin\psi = \frac{(\sin\phi - \sin\phi_{cv})}{(1 - \sin\phi\sin\phi_{cv})} \]  

(3.6)

where, \( \phi_{cv} \) is the ultimate friction angle and can be obtained from soil mechanics references such as Das (2008). According to the definition of dilation angle, this parameter has a significant role in plasticity analysis. To understand the effect of the dilation angle on, case study 1 was analysed in FLAC\textsuperscript{3D} (Figure 3.8). The dilation angle was calculated from Equation 3.6 and was used in the analysis using the software. Based on the applied lateral load, the pile lateral deflection and maximum bending moment were obtained. As can be seen in Table 3.1, dilation angle does not seem to have an important role in low amplitude of loading. In lateral load close to the ultimate loading, dilation angle shows low effects in the outcome results. Therefore, this parameter can be ignored due to the fact that the applied lateral loads in the case studies were in low amplitudes (i.e. elastic range).

<table>
<thead>
<tr>
<th>Lateral load (kN)</th>
<th>Lateral deflection (mm)</th>
<th>Bending Moment (kN.m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \phi = 39^\circ ) ( \psi = 0^\circ )</td>
<td>( \phi = 39^\circ ) ( \psi = 6^\circ )</td>
<td>( \phi = 33^\circ ) ( \psi = 0^\circ )</td>
</tr>
<tr>
<td>60</td>
<td>2.3</td>
<td>2.3</td>
</tr>
<tr>
<td>100</td>
<td>5.3</td>
<td>5.1</td>
</tr>
<tr>
<td>140</td>
<td>9</td>
<td>8.9</td>
</tr>
<tr>
<td>180</td>
<td>14</td>
<td>15</td>
</tr>
<tr>
<td>220</td>
<td>20</td>
<td>23</td>
</tr>
<tr>
<td>267</td>
<td>28</td>
<td>33</td>
</tr>
</tbody>
</table>

All the investigated case studies were modelled based on the methodology explained above in FLAC\textsuperscript{3D}. To simplify the procedure of the modelling, Figure 3.7 schematically illustrates the model of pile soil interaction and the parameters required for modelling in FLAC\textsuperscript{3D}.
Figure 3.7: Schematic view of the case studies investigated (D=pile diameter, t=wall thickness, L=pile length, EI=modulus of rigidity, K=bulk modulus, G=shear modulus, c=cohesion, $\phi =$ friction angle, $\gamma =$ soil unit weight, $k_n =$normal stiffness, and $k_s =$shear stiffness)

### 3.3.2 Standard approach (Winkler method) - Alp software

Winkler method, as a standard method of analysis is widely used in practice. This method was already explained in Chapter 2. There are many software which are based on Winkler approach. Alp (Analysis of Laterally Loaded Piles) is one of these available software programs. In this program, pile is modelled as a series of elastic beam elements and soil is modelled as a series of non-interactive, non-linear Winkler springs. In this program, there are also three options to model soil; elastic-plastic behaviour, specifying p-y curve, and generating p-y curve. This program is able to predict shear forces, bending moment, lateral deflection, pressure caused by a pile subjected to lateral loads, moments, and soil displacements (Oasys, Alp user’s manual, 2013).

### 3.3.3 Simplified approach (Broms’ method)

Broms’ method is a famous simplified method which can be employed to analyse laterally loaded pile. This method has already been explained in Chapter 2.

### 3.4 Pile rigidity influence investigation

It is estimated two responses for piles under lateral loading (Tomlinson and Woodward 2008):
1) Short rigid pile

2) Long flexible pile

For laterally loaded pile analysis, it might be necessary to know that a pile belongs to which categories. This can be obtained by considering stiffness factor. According to Tomlinson and Woodward (2008), the equations for obtaining the stiffness factors are as follows:

For normally-consolidated clay and granular soils:

\[ T = 5 \frac{EI}{n_h} \]  

(3.7)

For stiff over-consolidated clay:

\[ R = 4 \frac{EI}{KB} \]  

(3.8)

where, \( T \) and \( R \) are stiffness factor, \( E \) and \( I \) are pile properties (elasticity modulus and inertia moment), \( n_h \) is modulus of subgrade reaction that depends on type of soil, and \( B \) is the width of the pile. \( K = k_i / 1.5 \) where, \( k_i \) is Terzaghi’s subgrade modulus. The parameters of \( n_h \) and \( k_i \) can be obtained from Tomlinson and Woodward (2008). By calculating \( T \) or \( R \), according to the pile embedment length \( (L) \), the response of pile is defined in Table 3.2. As can be seen from Table 3.3, all the piles in the investigated cases studies act as a long flexible member.

<table>
<thead>
<tr>
<th>Type of pile</th>
<th>Sands and normally consolidated clay</th>
<th>Stiff clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>Short rigid</td>
<td>( L \leq 2T )</td>
<td>( L \leq 2R )</td>
</tr>
<tr>
<td>Long flexible</td>
<td>( L \geq 4T )</td>
<td>( L \geq 3.5R )</td>
</tr>
</tbody>
</table>
Table 3.3: Nature of the piles in the case studies

<table>
<thead>
<tr>
<th>Case study (Ref. Table 3.4)</th>
<th>Length of Pile (m)</th>
<th>T</th>
<th>R</th>
<th>4T</th>
<th>3.5R</th>
<th>Type of Pile</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>21</td>
<td>2</td>
<td>-</td>
<td>8</td>
<td>-</td>
<td>Flexible</td>
</tr>
<tr>
<td>2</td>
<td>15</td>
<td>-</td>
<td>2</td>
<td>-</td>
<td>7</td>
<td>Flexible</td>
</tr>
<tr>
<td>3</td>
<td>12.8</td>
<td>3</td>
<td>2</td>
<td>12</td>
<td>-</td>
<td>Flexible</td>
</tr>
<tr>
<td>4</td>
<td>11.6</td>
<td>1.8</td>
<td>-</td>
<td>7.2</td>
<td>-</td>
<td>Flexible</td>
</tr>
<tr>
<td>5</td>
<td>13</td>
<td>1.8</td>
<td>-</td>
<td>7.2</td>
<td>-</td>
<td>Flexible</td>
</tr>
<tr>
<td>6</td>
<td>25.6</td>
<td>4.8</td>
<td>-</td>
<td>19.15</td>
<td>-</td>
<td>Flexible</td>
</tr>
</tbody>
</table>

All case studies were modelled and analysed by using different numerical methods. The results obtained from the numerical analyses were then compared with the field test results for each of the case studies. Moreover, the results from the past research studies were also compared with the results from the present studies. A summary of the investigated case studies is presented in Table 3.4.

Table 3.4: Summary of the analysis of the case studies

<table>
<thead>
<tr>
<th>Pile load test</th>
<th>Pile details</th>
<th>Soil profile</th>
<th>Nature of the pile (Ref. Table 3.3)</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case study 1 (Mustang Island, Texas US)</td>
<td>21</td>
<td>0.61</td>
<td>Sand</td>
<td>Cox et al. (1974)</td>
</tr>
<tr>
<td>Case study 2 (Manor, Texas, US)</td>
<td>15</td>
<td>0.61</td>
<td>Stiff clay</td>
<td>Reese and Welch (1975)</td>
</tr>
<tr>
<td>Case study 3 (Sabine River, Texas, US)</td>
<td>12.8</td>
<td>0.32</td>
<td>Soft clay</td>
<td>Matlock (1970)</td>
</tr>
<tr>
<td>Case study 4 (Salt Lake City International Airport)</td>
<td>11.6</td>
<td>0.324</td>
<td>Layered</td>
<td>Synder (2004)</td>
</tr>
<tr>
<td>Case study 5 (Salt Lake City International Airport)</td>
<td>13</td>
<td>0.324</td>
<td>Layered</td>
<td>Walsh (2005)</td>
</tr>
<tr>
<td>Case study 6 (Incheon Bridge, Korea)</td>
<td>25.6</td>
<td>1.016</td>
<td>Layered</td>
<td>Kim et al. (2009)</td>
</tr>
</tbody>
</table>

3.5 Laterally loaded pile tests

3.5.1 Uniform soils

3.5.1.1 Case study 1

Cox et al. (1974) carried out field investigation on piles intended for offshore industry. The test site was in Mustang Island, Texas and the soil was predominantly sands. Two
large, hollow, open ended, circular driven displacement piles were tested; one under static loading and another one under cyclic loading. The tests were performed using a manually operated hydraulic equipment to apply the lateral load. The applied load was measured by using strain gauges with accuracy of 0.25%. The outputs for bending moments, lateral loads and displacements were recorded using a high-speed 20-channel digital-data acquisition system with the accuracy of 0.1%. More details of the test may be found in Cox et al. (1974).

Properties of pile

The test pile contained the following properties.

- Outer diameter \((D) = 0.61\)m
- Wall thickness \((t) = 0.0095\)m
- Length \((L) = 21\)m below ground and 3 m above the ground surface
- Modulus of rigidity of the pile section \((EI) = 170\)MN.m².

The plastic moment of the pile section can be calculated by Equation 3.9 as follow.

\[
M_p = \left( \frac{D_o^3 - D_i^3}{6} \right) \times \sigma_y
\]

where, \(M_p\) is plastic moment of the section, \(D_o\) and \(D_i\) are the external and internal diameter, respectively, and \(\sigma_y\) is the yield stress which can be assumed 400MPa for steel piles. Based on this equation, the plastic moment of the pile is predicted using the following calculation;

\[
M_p = \left( \frac{0.61^3 - 0.5913^3}{6} \right) \times 400000 = 1370kN.m
\]

Soil profile characteristics

The soil profile was predominantly sand. In the analysis, soil profile is idealised as sand layer of uniform properties. Elastic properties (bulk modulus, shear modulus, and Poisson’s ratio) have been determined from the data available in the test report by following empirical relations given in Bowles (1996). Water table was considered at the ground surface. Table 3.5 lists the sand properties for the considered sand layer.
Table 3.5: Material properties for case study 1

<table>
<thead>
<tr>
<th>Bulk modulus, $K$ (MPa)</th>
<th>Shear modulus, $G$ (MPa)</th>
<th>Unit weight, $\gamma$ (kN/m$^3$)</th>
<th>Internal friction angle, $\phi$ (degree)</th>
<th>Normal stiffness $k_n$ (Pa/m)</th>
<th>Shear stiffness $k_s$ (Pa/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>72</td>
<td>24</td>
<td>20.2</td>
<td>39$^\circ$</td>
<td>$2.1 \times 10^8$</td>
<td>$2.1 \times 10^8$</td>
</tr>
</tbody>
</table>

Results

Static lateral loading was applied at the free pile head (0.3m above the ground surface) and the analyses were carried out using FLAC$^{3D}$ (continuum method), Alp (Winkler method), and Broms’ method (Figure 3.8). Results obtained from these analyses were compared with the field test results along with the results obtained from different approach in the following sections. The results were presented in three parts of lateral load-deflection behaviour, lateral load-bending moment, and bending moment profile along the pile according to the particular applied lateral load.

Figure 3.8: Idealised of pile and soil in case study 1.

Lateral load-deflection behaviour

It can be seen from Figure 3.9 that the results obtained by Winkler approach were in close proximity with continuum approach (i.e. FEM$^{3D}$ and FLAC$^{3D}$) and with the observed values from the field test. As it appeared, at the higher amplitudes of loading, the deviation of the load-deflection pattern of Winkler approach increased from that of the observed field values.
Lateral load–bending moment behaviour

The design maximum bending moment has been computed for the pile section by different approaches and the results were compared with the field values (Figure 3.10). It can be observed that the bending moments obtained from continuum approaches such as FEM$^{3D}$ and FLAC$^{3D}$ and Winkler methods were in close agreement with the field test results. The moments obtained from the Broms’ approach significantly deviated from the observed values. This shows that the pile section designed by Broms’ method of analysis may yield very conservative but also can be considered as an uneconomic design. The plastic moment of the pile was computed around 1370kN.m and shown in Figure 3.10.

The bending moment profile obtained from different approaches for the maximum applied lateral load of 267 kN (as also applied during field investigation) were compared in Figure 3.11. Here also, it may be observed that Winkler and continuum approaches matched closely with the field values. Therefore, it can be concluded from the analyses that Winkler and continuum approaches yielded good agreement of results with the observed field values for sandy soil.

Figure 3.9: Comparison of lateral load-deflection behaviour for sandy soil.
Figure 3.10: Comparison of maximum bending moment for sandy soil ($M_p = $ Plastic moment capacity of pile section).

Figure 3.11: Comparison of bending moment profile for lateral load of 267 kN for sandy soil.
3.5.1.2 Case study 2

Reese and Welch (1975) conducted the full-scaled field test on pile behaviour for offshore industry. The test site was located in Manor Texas. This test also consisted of two large, hollow, open ended, circular, driven displacement piles subjected to static and cyclic loading. The strain gages were installed to measure the bending moment. The outputs for bending moments, lateral loads and displacements were recorded. More details of the test may be found in Reese and Welch (1975).

Properties of pile

The test pile contained the following properties.

- Outer diameter \((D) = 0.61\text{m}\)
- Wall thickness \((t) = 0.0095\text{m}\)
- Length \((L) = 15\text{m below ground and 3 m above the ground surface}\)
- Modulus of rigidity of the pile section \((EI) = 170\text{MN.m}^2\).
- Plastic moment \((M_p) = 1370\text{kN.m}\)

Soil profile characteristics

The soil profile of the field test was strongly over-consolidated stiff clay materials. In the analysis, soil profile was idealised as stiff clay layer of uniform properties. As in the previous case study, elastic properties (bulk modulus, shear modulus, and Poisson’s ratio) have been determined from the data available in the test report by following empirical relations given in Bowles (1996). Water table was considered at the ground surface. Table 3.6 represents the material properties for the stiff clay considered in the analysis.

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Unit weight, (\gamma) (kN/m(^3))</th>
<th>Bulk modulus, (K) (MPa)</th>
<th>Shear modulus, (G) (MPa)</th>
<th>Undrained shear strength, (c_u) (kPa)</th>
<th>Normal stiffness, (k_n) (Pa/m)</th>
<th>Shear stiffness, (k_s) (Pa/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0 - 0.9</td>
<td>18.1</td>
<td>230</td>
<td>5</td>
<td>25-70</td>
<td>4.7×10^8</td>
<td>4.7×10^8</td>
</tr>
<tr>
<td>0.9 - 1.52</td>
<td>18.2</td>
<td>580</td>
<td>12</td>
<td>70-163</td>
<td>1.2×10^9</td>
<td>1.2×10^9</td>
</tr>
<tr>
<td>1.52 - 4.11</td>
<td>19.4</td>
<td>1200</td>
<td>25</td>
<td>163-333</td>
<td>2.5×10^7</td>
<td>2.5×10^7</td>
</tr>
<tr>
<td>4.11 - 9.41</td>
<td>20.3</td>
<td>1600</td>
<td>34</td>
<td>333</td>
<td>3.3×10^7</td>
<td>3.3×10^7</td>
</tr>
<tr>
<td>&gt; 9.41</td>
<td>20.8</td>
<td>3500</td>
<td>72</td>
<td>333-1100</td>
<td>7.2×10^9</td>
<td>7.2×10^9</td>
</tr>
</tbody>
</table>
Results

Analyses have been carried out in different numerical methods with static lateral loading applied at the free pile head which has 0.3m projection over the ground surface (Figure 3.12). Results obtained from the different methods were validated against the field test results.

![Figure 3.12: Idealised of pile and soil in case study 2.](image)

**Lateral load–deflection behaviour**

As is shown in Figure 3.13, the results obtained by FLAC\textsuperscript{3D} and Winkler approaches were in less agreement with the field test values. However, the continuum method results showed much closer agreement with the observed values from the field test. At the higher amplitudes of loading, the deviation of the load-deflection pattern of Winkler approach increased from that of the observed field values. From these results, it may be concluded that continuum approach was in fairly good agreement with field test results. Another point that can be obtained from this figure was pile stiffness. As can be seen numerical methods illustrated less stiffness than measured values.
CHAPTER – 3
LATERALLY LOADED PILE ANALYSIS

Figure 3.13: Comparison of lateral load-deflection behaviour for stiff clay soil.

Lateral load–bending moment behaviour

Maximum bending moment in the pile section by different approaches were compared with the field values in Figure 3.14. It is evident that all methods of Broms, continuum and Winkler approaches yielded good comparison. The bending moment profile obtained from different approaches for the applied lateral load of 180 kN (as also applied during field investigation) were compared in Figure 3.15. Here, it can be seen that all the approaches matched closely with the field values. In contrast, this value obtained by Winkler and Broms’ approaches was rather conservative. It may also be indicated that the depth of maximum bending moment was identical for both continuum and Winkler approaches whereas depth of maximum bending moment by Broms’ method was slightly above as also observed in field test results.
Figure 3.14: Comparison of maximum bending moment for stiff clay soil ($M_p =$ Plastic moment capacity of pile section).

Figure 3.15: Comparison of bending moment profile for lateral load of 180 kN for stiff clay soil.
3.5.1.3 Case study 3

This case study was a full scale pile load test in soft clay soil conducted by Matlock (1970) for some soil companies. The test site was located at Sabine River in Texas. The pile was instrumented by strain gages to measure the bending moment pile deflection. Details of the test may be found in Matlock (1970).

Properties of pile

The test pile properties are as follows.

- Outer diameter \((D) = 0.32\) m
- Wall thickness \((t) = 0.0127\) m
- Length \((L) = 12.8\) m below ground and \(3.3\) m above the ground surface
- Modulus of rigidity of the pile section \((EI) = 30.444\) MN.m\(^2\).
- Plastic moment \((M_p) = 480\) kN.m

Soil profile characteristics

The soft clay at Sabine was a typical slightly over consolidated marine deposit. As in previous case studies, elastic properties (bulk modulus, shear modulus, and Poisson’s ratio) have been determined from the data available in the test report by following empirical relations given in Bowles (1996). Water table was considered near the ground surface. Material properties for the analysis of the stiff clay are presented in Table 3.7.

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Effective Unit weight, (\gamma') (kN/m(^3))</th>
<th>Bulk modulus, (K) (MPa)</th>
<th>Shear modulus, (G) (MPa)</th>
<th>Undrained shear strength (c_u) (kPa)</th>
<th>Normal stiffness (k_n) (Pa/m)</th>
<th>Shear stiffness (k_s) (Pa/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0 - 0.61</td>
<td>5.5</td>
<td>120</td>
<td>2.4</td>
<td>14.4</td>
<td>(2.5 \times 10^8)</td>
<td>(2.5 \times 10^8)</td>
</tr>
<tr>
<td>0.61 - 1.22</td>
<td>5.5</td>
<td>100</td>
<td>2.2</td>
<td>13.1</td>
<td>(2.1 \times 10^8)</td>
<td>(2.1 \times 10^8)</td>
</tr>
<tr>
<td>1.22 – 1.83</td>
<td>5.5</td>
<td>100</td>
<td>2.0</td>
<td>11.7</td>
<td>(2.0 \times 10^8)</td>
<td>(2.0 \times 10^8)</td>
</tr>
<tr>
<td>1.83 – 2.44</td>
<td>5.5</td>
<td>100</td>
<td>2.2</td>
<td>13.1</td>
<td>(2.1 \times 10^8)</td>
<td>(2.1 \times 10^8)</td>
</tr>
<tr>
<td>2.44 - 3.05</td>
<td>5.5</td>
<td>120</td>
<td>2.6</td>
<td>15.5</td>
<td>(2.5 \times 10^8)</td>
<td>(2.5 \times 10^8)</td>
</tr>
<tr>
<td>&gt; 3.05</td>
<td>5.5</td>
<td>120</td>
<td>2.5</td>
<td>15.2</td>
<td>(2.5 \times 10^8)</td>
<td>(2.5 \times 10^8)</td>
</tr>
</tbody>
</table>
Results

Static lateral loading was applied at the free pile head (0.3m above the ground surface) and the analyses have been carried out (Figure 3.16). The results were compared with the field test results along with the results obtained from different approaches in the following sections.

Lateral load–deflection behaviour

As can be seen from the Figure 3.17, the results obtained by FLAC$^{3D}$ and Winkler approaches were close to the field test values at low amplitude of loading. The load–deflection behaviour obtained from Winkler approach and FLAC$^{3D}$ was relatively away from the observed field test values at higher amplitudes. The pile stiffness values obtained from numerical approaches were in good agreement with the stiffness obtained from measured data.

\[ \gamma' = 5.5 \text{(kN/m}^3) \], \[ c_u = 14.4 \text{ kPa} \]

\[ \gamma' = 5.5 \text{(kN/m}^3) \], \[ c_u = 13.1 \text{ kPa} \]

\[ \gamma' = 5.5 \text{(kN/m}^3) \], \[ c_u = 11.7 \text{ kPa} \]

\[ \gamma' = 5.5 \text{(kN/m}^3) \], \[ c_u = 13.1 \text{ kPa} \]

\[ \gamma' = 5.5 \text{(kN/m}^3) \], \[ c_u = 15.5 \text{ kPa} \]

\[ \gamma' = 5.5 \text{(kN/m}^3) \], \[ c_u = 15.2 \text{ kPa} \]
Maximum bending moment has been computed for the pile section by different approaches and compared with the field values in Figure 3.18. It was observed that the bending moments in the lower range of lateral loading obtained from different approaches were more or less identical with the field test results. At the higher levels of lateral loading, deviation of results from different approaches with that of field test values became higher. In comparison, Broms’ approach displayed conservative values in view of design of pile section. The plastic moment of the pile was computed around 480kN.m which is shown in Figure 3.18.

The bending moment profile obtained from different approaches for the applied lateral load of 17.8kN (as also applied during field investigation) was compared in Figure 3.19. It was observed that the bending moments obtained from all the approaches were slightly conservative than the field test results. The depth of maximum bending moment was almost the same for all the approaches.
Figure 3.18: Comparison of maximum bending moment for soft clay soil ($M_p$ = Plastic moment capacity of pile section).

Figure 3.19: Comparison of bending moment profile for lateral load of 17.8 kN for soft clay soil.
3.5.2 Layered soils

3.5.2.1 Case study 4

Synder (2004) carried out full scaled lateral load tests of a 3×5 pile group in soft clays and silt. The test site was situated in a large unused lot owned by the Salt Lake City International Airport 300m north of the FAA control tower, Utah, USA and funded by the National Science Foundation. Static loads were applied 0.495m above ground level according to increments of deflection. The pile was instrumented to measure head deflections, loads and strains along the pile length.

Properties of pile

The test pile had the following properties.

- Outer diameter \((D)\) = 0.324m
- Wall thickness \((t)\) = 0.0095m
- Length \((L)\) = 11.6m below ground and 2.1m above the ground surface
- Modulus of rigidity of the pile section \((EI)\) = 28.600MN.m².
- Plastic moment \((M_p)\) = 376kN.m

Soil profile characteristics

The soil profile consisted of cohesive layers of soft to medium consistency underlain by interbedded layers of sands and fine-grained soils. The input soil parameters were idealised and considered as soft clays, silts, and layers of sand. Soil properties are presented in Table 3.8. The water table was at the base of the ground level.
**Table 3.8: Material properties for case study 4**

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Type of soil</th>
<th>Unit weight (γ) (kN/m³)</th>
<th>Bulk modulus (K) (MPa)</th>
<th>Shear modulus (G) (MPa)</th>
<th>Friction angle (φ) (Degree)</th>
<th>Undrained shear strength (c_u) (KPa)</th>
<th>Normal stiffness k_n (Pa/m)</th>
<th>Shear stiffness k_s (Pa/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-1.22</td>
<td>Soft clay</td>
<td>18.86</td>
<td>340</td>
<td>7.0</td>
<td>-</td>
<td>41.4</td>
<td>6.9×10⁸</td>
<td>6.9×10⁸</td>
</tr>
<tr>
<td>1.22-2.14</td>
<td>Soft clay</td>
<td>18.86</td>
<td>410</td>
<td>8.5</td>
<td>-</td>
<td>50</td>
<td>8.4×10⁸</td>
<td>8.4×10⁸</td>
</tr>
<tr>
<td>2.14-3.06</td>
<td>Soft clay</td>
<td>18.86</td>
<td>330</td>
<td>7.0</td>
<td>-</td>
<td>40</td>
<td>6.8×10⁸</td>
<td>6.8×10⁸</td>
</tr>
<tr>
<td>3.06-4.8</td>
<td>Sand</td>
<td>17.95</td>
<td>54</td>
<td>25</td>
<td>38</td>
<td>-</td>
<td>1.7×10⁸</td>
<td>1.7×10⁸</td>
</tr>
<tr>
<td>4.8-5.33</td>
<td>Soft clay</td>
<td>18.86</td>
<td>470</td>
<td>11</td>
<td>-</td>
<td>56.9</td>
<td>9.7×10⁸</td>
<td>9.7×10⁸</td>
</tr>
<tr>
<td>5.33-5.87</td>
<td>Soft clay</td>
<td>18.86</td>
<td>210</td>
<td>4.2</td>
<td>-</td>
<td>25</td>
<td>4.3×10⁸</td>
<td>4.3×10⁸</td>
</tr>
<tr>
<td>5.87-6.48</td>
<td>Soft clay</td>
<td>18.86</td>
<td>450</td>
<td>10</td>
<td>-</td>
<td>54</td>
<td>9.3×10⁸</td>
<td>9.3×10⁸</td>
</tr>
<tr>
<td>&gt;6.48</td>
<td>Sand</td>
<td>17.95</td>
<td>54</td>
<td>25</td>
<td>33</td>
<td>-</td>
<td>1.7×10⁸</td>
<td>1.7×10⁸</td>
</tr>
</tbody>
</table>

**Results**

Static lateral loading was applied at the free pile head (0.495m above the ground surface) and the analyses have been carried out (Figure 3.20). The results were compared with the field test results along with the results obtained from different approaches in the following sections.

![Figure 3.20: Idealised of pile and soil in case study 4.](image-url)
**Lateral load–deflection behaviour**

According to data in Figure 3.21, which is related to lateral loading versus lateral deflection, the results obtained by Winkler (Alp) and continuum approach (FLAC$^{3D}$) were in close proximity with the observed values from the field test. At the higher amplitudes of loading, the deviation of the load-deflection pattern of Winkler approach increased from that of the observed field values. From the pile stiffness point of view, as can be seen Winkler approach and FLAC$^{3D}$ illustrated the pile stiffness value in fairly good agreement with measured data.

![Lateral load–deflection behaviour graph](image)

**Figure 3.21**: Comparison of lateral load-deflection behaviour for layered soil.

**Lateral load–bending moment behaviour**

The graph of applied lateral load versus maximum bending moment in Figure 3.22 highlighted the accuracy of Winkler and continuum methods in compared to Broms’ method. It also revealed that the bending moments obtained from continuum method and Winkler approaches were in close comparison to the field test results. However, the moments obtained from Broms’ approach were significantly away from the observed
values. This implies that the pile section designed by Broms’ method may yield very conservative design. The computed plastic moment (376 kN.m) is shown in the figure.

![Figure 3.22: Comparison of maximum bending moment for layered soil ($M_p = \text{Plastic moment capacity of pile section}$).](image)

The bending moment profiles obtained from different approaches for the applied lateral load of 19.7 kN were compared in Figure 3.23. Based on the data, all the analyses methods showed underestimated profiles. It means that the measured maximum bending moment for this load exceeded that of the predicted by Broms, Winkler, and FLAC\textsuperscript{3D} methods. This point highlighted the question of the accuracy of the predictions made by all methods when compared to the actually behaviour of the pile in the soil.
Figure 3.23: Comparison of bending moment profile for lateral load of 19.7 kN for layered soil.

3.5.2.2 Case study 5

Walsh (2005) conducted field investigation on full scaled load test of a 3×5 pile group in sand. The test setting was identical to the Snyder’s (2004). More details of the test may be found in Walsh (2005).

Properties of the pile

The test pile had the following properties.

- Outer diameter \( D \) = 0.324m
- Wall thickness \( t \) = 0.0095m
- Length \( L \) = 13m below ground and 2.1m above the ground surface
- Modulus of rigidity of the pile section \( EI \) = 28,600MN.m².
- Plastic moment \( M_p \) = 376kN.m
Soil profile characteristics

The soil properties are presented in Table 3.9. The water table was at 2.13 m below ground surface.

Table 3.9: Material properties for case study 5

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Type of soil</th>
<th>Unit weight ($\gamma$) (KN/m$^3$)</th>
<th>Bulk modulus (K) (MPa)</th>
<th>Shear modulus (G) (MPa)</th>
<th>Friction angle ($\phi$) (Degree)</th>
<th>Undrained shear strength ($c_u$) (KPa)</th>
<th>Normal stiffness $k_n$ (Pa/m)</th>
<th>Shear stiffness $k_s$ (Pa/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-2.13</td>
<td>Sand</td>
<td>16.7</td>
<td>54</td>
<td>25</td>
<td>40</td>
<td>-</td>
<td>$1.7 \times 10^8$</td>
<td>$1.7 \times 10^8$</td>
</tr>
<tr>
<td>2.13-2.44</td>
<td>Sand</td>
<td>16.8</td>
<td>54</td>
<td>25</td>
<td>40</td>
<td>-</td>
<td>$1.7 \times 10^8$</td>
<td>$1.7 \times 10^8$</td>
</tr>
<tr>
<td>2.44-2.7</td>
<td>Soft clay</td>
<td>19.1</td>
<td>340</td>
<td>7.0</td>
<td>-</td>
<td>41</td>
<td>$6.9 \times 10^8$</td>
<td>$6.9 \times 10^8$</td>
</tr>
<tr>
<td>2.7-3.7</td>
<td>Soft clay</td>
<td>19.1</td>
<td>410</td>
<td>8.5</td>
<td>-</td>
<td>50</td>
<td>$8.4 \times 10^8$</td>
<td>$8.4 \times 10^8$</td>
</tr>
<tr>
<td>3.7-4.6</td>
<td>Soft clay</td>
<td>19.1</td>
<td>330</td>
<td>7.0</td>
<td>-</td>
<td>40</td>
<td>$6.8 \times 10^8$</td>
<td>$6.8 \times 10^8$</td>
</tr>
<tr>
<td>4.6-6.3</td>
<td>Sand</td>
<td>18.1</td>
<td>54</td>
<td>25</td>
<td>38</td>
<td>-</td>
<td>$1.7 \times 10^8$</td>
<td>$1.7 \times 10^8$</td>
</tr>
<tr>
<td>6.3-8</td>
<td>Soft clay</td>
<td>19.1</td>
<td>480</td>
<td>10</td>
<td>-</td>
<td>57</td>
<td>$9.7 \times 10^8$</td>
<td>$9.7 \times 10^8$</td>
</tr>
<tr>
<td>&gt; 8</td>
<td>Sand</td>
<td>16.7</td>
<td>54</td>
<td>25</td>
<td>33</td>
<td>-</td>
<td>$1.7 \times 10^8$</td>
<td>$1.7 \times 10^8$</td>
</tr>
</tbody>
</table>

Results

Static lateral loading was applied at the free pile head (0.495m above the ground surface) and the analyses were carried out (Figure 3.24). Results obtained from analyses were compared with the field test observations along with the comparison results obtained from different approaches in the following sections.
Figure 3.24: Idealised of pile and soil in case study 5.

*Lateral load-deflection behaviour*

As is shown in Figure 3.25 which is related to lateral loading versus lateral deflection, the results obtained from continuum and Winkler approaches were in close comparison to the field test results in low amplitude of lateral loading. Also, the pile stiffness obtained from Winkler approach and continuum methods were in good agreement with measured values.

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Density $\gamma$ (kN/m$^3$)</th>
<th>Friction Angle $\phi$ (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand</td>
<td>16.7</td>
<td>40</td>
</tr>
<tr>
<td>Sand</td>
<td>16.8</td>
<td>40</td>
</tr>
<tr>
<td>Soft clay</td>
<td>19.1 ($\gamma$ = 1 kN/m$^3$), $c_u$ = 41 kPa</td>
<td></td>
</tr>
<tr>
<td>Soft clay</td>
<td>19.1 ($\gamma$ = 1 kN/m$^3$), $c_u$ = 50 kPa</td>
<td></td>
</tr>
<tr>
<td>Soft clay</td>
<td>19.1 ($\gamma$ = 1 kN/m$^3$), $c_u$ = 40 kPa</td>
<td></td>
</tr>
<tr>
<td>Sand</td>
<td>18.1 ($\gamma$ = 3 kN/m$^3$), $\phi$ = 38°</td>
<td></td>
</tr>
<tr>
<td>Soft clay</td>
<td>19.1 ($\gamma$ = 1 kN/m$^3$), $c_u$ = 57 kPa</td>
<td></td>
</tr>
<tr>
<td>Soft clay</td>
<td>19.1 ($\gamma$ = 1 kN/m$^3$), $c_u$ = 40 kPa</td>
<td></td>
</tr>
<tr>
<td>Sand</td>
<td>16.7 ($\gamma$ = 3 kN/m$^3$), $\phi$ = 33°</td>
<td></td>
</tr>
</tbody>
</table>
Figure 3.25: Comparison of lateral load-deflection behaviour for layered soil.

**Lateral load-maximum bending moment**

Figure 3.26 plots the lateral load against the maximum bending moment. The trend of the curves plotted in the figure varied quite significantly between the predicted values. Data showed that Winkler approach and continuum method predicted the behaviour of pile similar to the measured values. However, by increasing the lateral loading continuum method showed a little gap in its values compared with Winkler approach and measured values. Interestingly, Broms’ predictions was extensively more conservative, compared to the other methods and the measured values. This predicted bending moments by Broms’ method could pass the plastic moment capacity (376 kN.m) for the loads over than ~120kN.
The bending moment profiles obtained from different approaches for applied lateral load of 50kN were compared in Figure 3.27. It can be seen that among all approaches, continuum analyse and Winkler method matched closely with the field test results. It may also worth to mention that the maximum bending moment obtained from Winkler approach was less conservative in upper depth and the bending moment profile was close to field values in lower depth. As shown, however, there was a huge gap between the bending moment obtained from Broms’ approach and other approaches. Broms’ approach again showed a conservative trend.
Figure 3.27: Comparison of bending moment profile for lateral load of 50 kN for layered soil.

3.5.2.3 Case study 6

Kim et al. (2009) and Jeong et al. (2007) carried out a series of field load tests in order to investigate the behaviour of piles subjected to lateral load. The test site was located at the Incheon Bridge, Korea. This location was a marine deposit. Full-scale field load tests were performed on six instrumented piles under a free pile head condition. Details of the test may be found in Kim et al. (2009) and Jeong et al. (2007).

Properties of pile

The test pile had the following properties.

- Outer diameter \( D \) = 1.016 m
- Wall thickness \( t \) = 0.016 m
- Length \( L \) = 25.6 m below ground and 1 m above the ground surface
- Modulus of rigidity of the pile section \( EI \) = 1257 MN.m²
- Plastic moment \( M_p \) = 6400 kN.m
Soil profile characteristics

According to Kim and Jeong (2011), the soil profile near the surface consisted of layers of silty clay and silty sand underlain by a marine clay deposit. The cohesive surface soils consisted of low-plasticity silts and clays. The water table was located near the natural ground surface. The undrained shear strength was typically between 18 to 42 kPa however, some layers had strengths of 125 kPa. Consolidation tests showed that the soils were normally to very slightly over consolidate. The soil properties are presented in Table 3.10.

Table 3.10: Material properties for case study 6

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Type</th>
<th>$\gamma_{sat}$ (kN/m$^3$)</th>
<th>$\varphi$ (Degree)</th>
<th>$c_u$ (kPa)</th>
<th>$\nu$</th>
<th>Bulk modulus (K) (MPa)</th>
<th>Shear modulus (G) (MPa)</th>
<th>Normal stiffness $k_n$ (Pa/m)</th>
<th>Shear stiffness $k_s$ (Pa/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-6.3</td>
<td>Upper clay soil</td>
<td>17.5</td>
<td>-</td>
<td>15-30</td>
<td>0.49</td>
<td>205</td>
<td>3.5</td>
<td>$4.2 \times 10^8$</td>
<td>$4.2 \times 10^8$</td>
</tr>
<tr>
<td>6.3-16.5</td>
<td>Lower clay soil</td>
<td>17.5</td>
<td>-</td>
<td>30-50</td>
<td>0.49</td>
<td>420</td>
<td>11</td>
<td>$8.7 \times 10^8$</td>
<td>$8.7 \times 10^8$</td>
</tr>
<tr>
<td>16.5-22</td>
<td>Silty clay</td>
<td>17.8</td>
<td>-</td>
<td>70</td>
<td>0.49</td>
<td>700</td>
<td>17</td>
<td>$1.4 \times 10^7$</td>
<td>$1.4 \times 10^7$</td>
</tr>
<tr>
<td>22-24</td>
<td>Residual soil</td>
<td>18</td>
<td>34</td>
<td>-</td>
<td>0.49</td>
<td>700</td>
<td>17</td>
<td>$1.4 \times 10^7$</td>
<td>$1.4 \times 10^7$</td>
</tr>
<tr>
<td>24-26.6</td>
<td>Weathered rock</td>
<td>20.2</td>
<td>-</td>
<td>-</td>
<td>0.25</td>
<td>150</td>
<td>80</td>
<td>$5.1 \times 10^8$</td>
<td>$5.1 \times 10^8$</td>
</tr>
</tbody>
</table>

Results

The lateral load was applied at a point 0.5 m above the ground surface (Figure 3.28). Results were compared with the field test and finite element results in the following sections.
**Figure 3.28: Idealised of pile and soil in case study 6.**

**Lateral load-deflection behaviour**

From the data in Figure 3.29, which is related to lateral loading versus lateral deflection, from both applied lateral load which were 200kN and 600kN, the FEM\(^3\)D using PLAXIS\(^3\)D and FDM\(^3\)D using FLAC\(^3\)D showed a fairly good agreement with the measured values. Both of these analyses, however, were based on continuum modelling and as was mentioned before the continuum modelling might better predict the real behaviour of the pile.
The bending moment profiles obtained from different continuum approaches for applied lateral loads of 200kN and 600kN were compared in Figure 3.30. It is clear that both FEM and FDM analyses had reasonable predictions. Both finite element and finite difference methods showed a fairly good agreement with measured values obtained from field test. The plastic capacity of the pile was computed around 6400kN.m.
3.6 Conclusion

This chapter presents three different methods of laterally loaded pile analysis: advanced computationally expensive approach (Continuum method), standard approach (Winkler spring type method where the pile-soil interaction is modelled by a set of non-linear springs), and simplified approach (Broms’ method which is Limit Equilibrium approach). Six field case records of laterally loaded piles have been analysed using all the methods. The main intention is to compare whether or not the popular and standard Winkler approach (the so called p-y springs approach) can provide similar answers to that of the computationally expensive continuum approach. It was also found that if appropriate p-y springs are used, comparable results can be obtained at a fractional cost. This conclusion supports the use of the Winkler method in practice, bearing in mind however, that it does not account for the transient response phase.
4.1 Introduction

Understanding the behaviour of structures during earthquakes is crucial for better (safer) future designs and constructions. Various number of damages of different structures have been reported during past earthquakes (e.g. Japan 2011, New Zealand 2011, Chile 2010, Italy 2009, and China 2008, and so on). These structures failed either due to the superstructure or foundation failure. The superstructure failure can be observed and is therefore a more understandable example of the failure mechanism. On the other hand, as foundations are the hidden part of any structure, there might be some difficulties in order to fully understand the failure mechanism of any foundations. However, this problem may not be a major issue for the shallow type foundations, as the problem may be solved by being able to excavate a
shallow top layer of the soil. Failure mechanism of pile foundations, as a deep foundations, due to the excavation difficulties, has been an issue of considerable concern. Experimental tests, however, are known as an expensive way to understand the behaviour of pile foundations. For instance, centrifuge and shake table tests have been carried out all over the world to have a more logical research approach about the pile failure mechanism understanding. These type of tests are thoroughly reported in the literature. Tables 4.1 and 4.2 list some of the representing model tests which were carried out using shake table and centrifuge facilities. Hence, at the University of Bristol, a series of shaking table tests have also been carried out to pursue the current research. This chapter specifically explains the shake table test set-up, materials, and other related instruments. The measured natural frequency of pile models in different conditions is also presented at the end of the chapter.
Table 4.1: Some experimental model tests on pile foundations using shaking table test

<table>
<thead>
<tr>
<th>No</th>
<th>Reference</th>
<th>The investigated problem</th>
<th>Conclusion</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Tang et al. (2015)</td>
<td>The response of a Reinforced Concrete (RC) pile foundation behind a sheet-pile quay wall under lateral spreading. Monotonic lateral response of pile was particularly studied.</td>
<td>The displacement of pile under lateral spreading was underestimated. Increasing the pile diameter could produce significantly larger moment and displacement of the pile. Therefore, it was suggested that the flow of soil materials pushed the pile foundation.</td>
<td>The effect of axial load from the superstructure was considered.</td>
</tr>
<tr>
<td>2</td>
<td>Chen et al. (2015)</td>
<td>Failure mechanism of subway structures in soft soil during strong earthquake.</td>
<td>Seismic response of the soil and structure depends on input motion with richer low frequency components.</td>
<td>FEM analysis was carried out to verify the experiment results. Also, the investigated structure was the prototype of subway station based on Xinjiekou station, Nanjing Metro line, China.</td>
</tr>
<tr>
<td>3</td>
<td>Lombardi &amp; Bhattacharya (2014)</td>
<td>Effect of liquefaction on modal parameters of pile foundations (natural frequency and damping ratio).</td>
<td>Natural frequency of pile foundation decreases due to liquefaction. They found that damping ratio will increase due to liquefaction.</td>
<td>FEM model based on Winkler springs and p-y curves was carried out to verify the measured data.</td>
</tr>
<tr>
<td>4</td>
<td>Tang and Ling (2014)</td>
<td>Failure mechanism of Reinforced Concrete (RC) pile group embedded in a two layer of strata.</td>
<td>The bending failure is more likely to happen due to the liquefaction caused by earthquake.</td>
<td>Opensees analysis was carried out to estimate the RC pile group response under the combination of bending and axial loading.</td>
</tr>
<tr>
<td>5</td>
<td>Motamed et al. (2013)</td>
<td>The effect of E-Defence facility on a pile group response during lateral spreading of liquefiable soil.</td>
<td>The maximum of the lateral displacement of soil was measured at the surface. Also, the larger bending moment of pile was measured near the quay wall at the rear row piles.</td>
<td>The two codes of practice of JRA and JSWA were compared. JSWA guideline was shown the larger results compared to JRA guideline.</td>
</tr>
<tr>
<td>6</td>
<td>Gao et al, (2011)</td>
<td>Macro phenomena research on seismic behaviour of soil-pile-bridge.</td>
<td>The frequency of motion cannot effect on pile and the soil response. However, the pile and the soil responses depend on the amplitude of motion.</td>
<td>The effect of axial load was considered.</td>
</tr>
<tr>
<td>No</td>
<td>Reference</td>
<td>The investigated problem</td>
<td>Conclusion</td>
<td>Remarks</td>
</tr>
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<td>-----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------</td>
<td>--------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------</td>
</tr>
<tr>
<td>7</td>
<td>Motamed &amp; Towhata (2010)</td>
<td>The behaviour of pile groups behind quay walls subjected to lateral spreading</td>
<td>Fixed end sheet pile shows the most effective results in terms of reduction in bending moment of pile.</td>
<td>The effect of axial load was considered in their research.</td>
</tr>
<tr>
<td>8</td>
<td>Chau et al. (2009)</td>
<td>The seismic interaction between soil-pile-structure.</td>
<td>Due to the shaking a pounding phenomenon was observed between soil and the pile foundations. Also they showed that the pile cap acceleration response might be up to three times more than the response of structure.</td>
<td>Both sinusoidal wave and the acceleration time history of the earthquake were employed for the experiment. Also, FEM analysis was carried out to evaluate the observed phenomena.</td>
</tr>
<tr>
<td>9</td>
<td>Dungca et al. (2006)</td>
<td>Focuses were on the liquefiable soil deformation during the large displacement between the pile foundation and the surrounded soil.</td>
<td>The influenced soil area would have a direct effect on lateral resistance.</td>
<td>The effect of loading rate on the lateral resistance of the pile in the liquefiable sand was also investigated.</td>
</tr>
<tr>
<td>10</td>
<td>Cubrinovski et al. (2006)</td>
<td>The behaviour of pile foundations under lateral spreading.</td>
<td>The construction material and as a consequence the pile rigidity can have an effect on pile foundation behaviour under lateral spreading.</td>
<td>The effect of axial load was not considered.</td>
</tr>
<tr>
<td>11</td>
<td>Tokimatsu et al. (2005)</td>
<td>The behaviour of pile supported structures was investigated under the combination of inertial and kinematic forces.</td>
<td>The pile foundation response depends on time period of the ground as well as the superstructure.</td>
<td>A pseudo static analysis was carried out to estimate the displacement and stresses of the pile.</td>
</tr>
<tr>
<td>12</td>
<td>Yao et al. (2004)</td>
<td>The effect of liquefaction on behaviour of the soil-pile-superstructure</td>
<td>Once the excess pore water pressure generates, predominant period of the system becomes longer. It is important to consider transient phase of liquefaction because the maximum value of bending moment and earth pressure can be happened in this phase.</td>
<td>The effect of axial load was considered.</td>
</tr>
<tr>
<td>No</td>
<td>Reference</td>
<td>The investigated problem</td>
<td>Conclusion</td>
<td>Remarks</td>
</tr>
<tr>
<td>----</td>
<td>----------------------------</td>
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<td>----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------</td>
<td>--------------------------------------------------------------------------------------------</td>
</tr>
<tr>
<td>13</td>
<td>Iwasaki et al. (1984)</td>
<td>Estimating liquefaction potential by using fundamental properties of the soil.</td>
<td>Two simplified methods were proposed to evaluate the liquefaction potential. These methods were based on liquefaction resistance factor (i.e. $F_L$) and liquefaction potential index (i.e. $I_L$). Shake table experiments was also shown that liquefaction can assess by $F_L$.</td>
<td>Shake table experiment was carried out to verify the liquefiable soil properties, pile foundation behaviour under liquefaction. Also, 64 liquefied sites and 23 non-liquefied sites were case studied to evaluate the proposed methods.</td>
</tr>
</tbody>
</table>

Table 4.2: Some experimental model test on pile foundations using centrifuge test

<table>
<thead>
<tr>
<th>No</th>
<th>Reference</th>
<th>The investigated problem</th>
<th>Conclusion</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Ng et al. (2014)</td>
<td>The effect of twin tunnelling on response of pile group</td>
<td>Pile group experienced extra settlement due to the twin tunnelling. Due to reduction in stress caused by twin tunnelling, the closet pile to the excavated tunnel carried less load.</td>
<td>In-flight load experiment was considered to obtain the pile group capacity.</td>
</tr>
<tr>
<td>2</td>
<td>Ng et al. (2013)</td>
<td>Effect of construction of twin tunnel on behaviour of pile group</td>
<td>The settlement of pile foundation due to tunnel construction depends on the depth of tunnel relative to the pile.</td>
<td>The effect of axial load on pile foundation was considered.</td>
</tr>
<tr>
<td>3</td>
<td>Holscher et al. (2012)</td>
<td>The effective factors on resistance of pile foundation during rapid loading.</td>
<td>During such a rapid loading the maximum toe resistance is higher than static loading.</td>
<td>The excess pore water pressure generation was measured during rapid loading.</td>
</tr>
<tr>
<td>4</td>
<td>Knappett &amp; Madabhushi, (2005)</td>
<td>Pile group instability failure due to liquefaction.</td>
<td>The pile groups under axial load may suffer the instability failure caused by liquefaction.</td>
<td>FEM method based on p-y curves was carried out to validate the results.</td>
</tr>
<tr>
<td>5</td>
<td>Brandenberg et al. (2005)</td>
<td>The behaviour of single pile and pile group foundations under lateral spreading.</td>
<td>Lateral load direction depends on the direction of relative movement between the soil and the pile foundation.</td>
<td>Back calculation was carried out to obtain dynamic soil-pile force.</td>
</tr>
<tr>
<td></td>
<td>Reference</td>
<td>Description</td>
<td>Observation</td>
<td>Methodology</td>
</tr>
<tr>
<td>---</td>
<td>-----------</td>
<td>-------------</td>
<td>-------------</td>
<td>-------------</td>
</tr>
<tr>
<td>6</td>
<td>Takahashi &amp; Takemura, (2005)</td>
<td>Failure mechanism of pile-supported wharf in front of backfilled due to liquefaction.</td>
<td>A large movement was observed due to the liquefaction of the backfill soil. Consequently, a large displacement gap was observed between the rubble mound and the bearing stratum which produced a large bending moment at the top of the pile.</td>
<td>Numerical analysis was carried out to verify the experiment results.</td>
</tr>
<tr>
<td>8</td>
<td>Wilson et al. (2000)</td>
<td>The effect of liquefaction on dynamic response of pile foundations.</td>
<td>The soil-pile interaction was directly obtained from the observed p-y response through back analysis of a single pile. The back analysed p-y curve represented the experimentally observed soil-pile interaction.</td>
<td>The experiment was carried out in loose and medium dense sand materials.</td>
</tr>
<tr>
<td>9</td>
<td>McVay et al. (1998)</td>
<td>The behaviour of laterally loaded pile group in sand with different size groups.</td>
<td>By changing the size of the group, there was no change in the group’s lateral resistance in an individual row’s contribution.</td>
<td>They implied that the p-multiplier concept is valid.</td>
</tr>
</tbody>
</table>
4.2 Objective of the experiments

A series of large scale shake table tests were carried out to understand the dynamic of soil-pile interaction mechanisms. The aims of these experiments were to understand the effect of liquefaction on different parameters such as time period change of pile models, transience of bending moment along the pile models, and the effect of time taken to reach liquefaction (i.e. speed of liquefaction) on transience of bending moment.

4.3 Test set-up

These tests were carried out at Bristol Laboratory for Advanced Dynamics Engineering (BLADE) at the University of Bristol. Experimental plan applied carefully throughout the experimental studies. Equipment, materials, and instruments used for these studies as well as the experimental set-up are explained as follows.

4.3.1 Shake table

The shake table dimension was 3m×3m and input motions were applied in all six degrees of freedom. The table was made up of cast aluminium and weighs about 3.8 tonnes which was placed inside the reinforced concrete seismic block with the mass of 300 tonnes (Crewe, 2007). Eight servo hydraulic actuators were used to attach the aluminium plate to the block. The dynamic capacity of each actuator was 70kN with the maximum movement of 300 mm which could provide a full control of motion in six degrees of freedom (Crewe, 2007). Figure 4.1 shows the photo of the shake table at the University of Bristol.
4.3.2 Soil container

A rigid soil container with energy absorbing boundaries was used to carry out the experiments. This container was made up of 18 “channel” steel profile section with the dimension of 100mm × 50mm. The container was a rigid modular container with the dimensions of 2.4m (length), 1.2m (width), and 2.4m (height). However, soft boundaries for absorbing energy might also be added in two sides of the container (Bhattacharya et al. 2012). The main limitation of the rigid container was the reflection of P-wave from the end walls due to the shaking which progressively disappears with distance. The finite dimensions of the container could not allow the P-wave, generated by the side boundaries, to dissipate. To reduce the effect of this limitation a layer of foam with 0.5 m thickness was added to absorb energy on both sides of the container. Moreover, to make the container water proof, 1 mm rubber was used to cover the inner sides of the container. Figure 4.2 shows the back view of the soil container and Figure 4.3 shows the soil container placed on the shake table.
Figure 4.2: Back view of the soil container.

Figure 4.3: Soil container placed on the shaking table.
4.3.3 Pile foundations

Four pile models including two single piles and two pile groups were tested in this experiment. The piles were made up of Aluminium alloy (L114-T4 6082-T4) with the length of 2 meter for all the pile models and with two different diameter sizes; 25.4 and 41.275 mm for small and large pile diameter, respectively. The material properties are mentioned in Table 4.3. The pile group arrangement was 2×2 with the 3D (D is pile diameter) space between the piles. Based on the literature, a distance between (3~4) D for is recommended to be a suitable arrangement for the piles distance in a pile groups (Tomlinson and Woodward, 2008; Fleming et al. 2009). As shown in Figure 4.4, a wooden base support was made and then placed at the bottom of the container. All the pile structures were then fixed from their bottom to the wooden base. Therefore, the bottom boundaries of the piles were considered as fixed boundaries. This boundary condition was then considered for the numerical analysis. A steel plate was applied for each pile structures to represent the pile cap. Figure 4.5 illustrates the pile models arrangement and shows pile models labels; SP1 (single pile with small diameter), SP2 (single pile with larger diameter), GP1 (pile group with small diameter), and GP2 (pile group with larger diameter).

Table 4.3: Aluminium alloy (L114-T4 6082-T4) properties

<table>
<thead>
<tr>
<th>Aluminium alloy type</th>
<th>Modulus of Elasticity (GPa)</th>
<th>Density (g/cm³)</th>
<th>Proof Stress (MPa)</th>
<th>Tensile Strength (MPa)</th>
<th>Shear Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>L114-T4 6082-T4</td>
<td>70</td>
<td>2.70</td>
<td>170</td>
<td>260</td>
<td>170</td>
</tr>
</tbody>
</table>
In order to represent superstructure on the top of the pile models, a number of masses were then added on the top of the pile models. The amount of mass was calculated based on the critical load of pile models which was computed based on the Euler equation (i.e. buckling criteria) as given in Equation 4.1;
\[ P_{cr} = \frac{n\pi^2 EI}{L^2} \]  

(4.1)

where, \( P_{cr} \) is critical load (N), \( E \) is modulus of elasticity \( (N/m^2) \), \( I \) is moment of inertia \( (m^4) \), \( L \) is length of column (m), and \( n \) is factor accounting for the end conditions.

More details of the pile models such as properties, dimensions, pile cap weight, and superstructure weight are mentioned in Table 4.4.

Table 4.4: Pile structure properties used in the experiment

<table>
<thead>
<tr>
<th>ID</th>
<th>Outer Diameter (mm)</th>
<th>Wall Thickness (mm)</th>
<th>Length (m)</th>
<th>EI (Nm²)</th>
<th>Pile cap Dimension (mm)</th>
<th>Pile cap weight (kg)</th>
<th>Superstructure weight (kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SP12</td>
<td>25.4</td>
<td>0.711</td>
<td>2</td>
<td>294</td>
<td>100×100×25.4</td>
<td>1.9</td>
<td>5</td>
</tr>
<tr>
<td>SP22</td>
<td>41.275</td>
<td>0.711</td>
<td>2</td>
<td>1305</td>
<td>150×150×25.4</td>
<td>8.44</td>
<td>20</td>
</tr>
<tr>
<td>GP12</td>
<td>25.4</td>
<td>0.711</td>
<td>2</td>
<td>294</td>
<td>260×260×25.4</td>
<td>13.08</td>
<td>65</td>
</tr>
<tr>
<td>GP22</td>
<td>41.275</td>
<td>0.711</td>
<td>2</td>
<td>1305</td>
<td>260×260×25.4</td>
<td>22.72</td>
<td>115</td>
</tr>
</tbody>
</table>

In order to distinguish between the pile models response with and without mass, the pile models are labelled as follow; pile models with pile cap only (no mass) are; SP11, SP21, GP11, and GP21 and pile models with pile cap and mass are; SP12, SP22, GP12, and GP22.

4.3.4 Soil properties

Redhill-110 sand was used to carry out the shake table tests. Redhill-110 was a fine-grained silica sand (Figure 4.6) which was sieved in order to obtain the particle size distribution. The obtained particle size distribution was then matched on the particle size distribution graph for liquefiable soil (Figure 4.7). As can be seen in Figure 4.7, Redhill-110 sand distribution stood within most liquefiable sand regions.

Once all the pile models were placed inside the container, the soil container was then poured with the sand using dry pluviation method. The barrel was filled with sand, lifted by ceiling crane and placed on the top of the soil container and poured from the certain height (i.e. 1.5m) to obtain homogeneous soil mixture. In order to compute the relative density of sand, the weight of each barrel was measured by using the scale and added together to have the total mass of the used soil. Figure 4.8 shows the barrel to carry the sand and the scale to measure the weight of the dry sand. Figure 4.9 shows dry pluviation method. The maximum
height of the soil after pouring process completion was 1.8 m. By having the total mass of the soil and also the volume of the soil container the relative density was then calculated. The relative density obtained was 14% for the sand inside the container after dry pluviation process. The sand was then saturated by adding water from top to bottom. The relative density of saturated sand obtained was measured to be about 34%. The main characteristics of the sand are listed in Table 4.5.

Table 4.5: Redhill-110 sand properties used in the experiment

<table>
<thead>
<tr>
<th>Sand</th>
<th>Specific gravity, ($G_s$)</th>
<th>D50 (mm)</th>
<th>Maximum void ratio, ($e_{max}$)</th>
<th>Minimum void ratio, ($e_{min}$)</th>
<th>Friction angle, ($\phi$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>RedHill-110</td>
<td>2.65</td>
<td>0.18</td>
<td>1.035</td>
<td>0.608</td>
<td>36°</td>
</tr>
</tbody>
</table>

Figure 4.6: Microscopic photo of Redhill-110 sand.
Figure 4.7: Particle size distribution of Redhill-110 sand based on grain size distribution of liquefaction-prone sand regarding to Japanese Seismic Code for Harbor Structures.

Figure 4.8: (a) Barrel filled with Redhill-110 sand and (b) scale.

Figure 4.8: (a) Barrel filled with Redhill-110 sand and (b) scale.
4.3.5 Instruments

In these series of experimental studies, different associated physical parameters were recorded by using proper instruments. The instruments used were strain gauges, Pore Pressure Transducers (PPT), and two types of accelerometers; SETRA and MEMS. Each of the instruments used is explained in the following section. Figure 4.10 also, schematically illustrates the location of the instruments. In order to have an accurate results all the instruments were calibrated before and after the experiment.
4.3.5.1 Strain gauges

Strain gauges (C2A-06-125-LW-350) were adapted to measure strains in the pile models. The strain gauges were manufactured by Micro-Measurements Group. The wired strain gauges were placed as a pair along the external surface of piles by using M-Bond 200 adhesive. In order to avoid any damage, wires were protected by being passed through inside the pile models tube. Figure 4.11 shows the strain gauge attached to the pile model. Four pairs of strain gauge were placed along the piles for pile models SP1 and GP1 and seven pairs of strain gauge for pile models SP2 and GP2. For pile groups only one pile was instrumented with the strain gauges. RDP 600 Multi-Channel Signal Conditioning was used in order to provide the excitation voltage. RDP 628-type strain gauge amplifier modules wire into the RDP 600 was used for providing Wheatstone bridge (see Figure 4.12). The data recorded from the strain gauges was used to calculate the bending moment along the piles. Table 4.6 presents the strain gauge characteristics.
Figure 4.11: Strain gauge attached on the external surface of the pile model.

Figure 4.12: RDP 628-type strain gauge amplifier modules mounted on the RDP 600.

Table 4.6: Characteristic of the C2A-06-125-LW-350 strain gauge

<table>
<thead>
<tr>
<th>Strain gauge characteristics</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grid resistance in Ohms</td>
<td>350±0.6%</td>
</tr>
<tr>
<td>TC of gauge factor 100°C</td>
<td>1.3±0.2</td>
</tr>
<tr>
<td>Gauge factor at 24°C</td>
<td>2.115±0.5%</td>
</tr>
<tr>
<td>Transverse sensitivity</td>
<td>0.3±0.2%</td>
</tr>
</tbody>
</table>
4.3.5.2 Pore Pressure Transducer (PPT)

Pore Pressure Transducer (PPT) (PDCR 811) was used to record pore water pressure at different depths of the soil. The pore pressure was recorded by the flexible silicon diaphragm which was located in front of each PPT (Figure 4.13). The silicon diaphragm was covered by the aluminium cap for protection. A 10 voltage power supply as well as 5mA (nominal) was provided using RDP 611 Multi-Channel Signal Conditioning to have 10 V voltage per 50 kPa. All the PPTs were calibrated before and after the experiment using known hydrostatic pressure. As shown in Figure 4.14, five PPTs were placed at different depths of the soil during dry pluviation. By this arrangement, it was possible to monitor the pore water pressure generation and as a consequence soil liquefaction at different levels. Figure 4.10 illustrates the location of the PPT in the soil container.

Figure 4.13: PPT and the silicon diaphragm.
4.3.5.3 Accelerometers

Two types of accelerometers were used to record the acceleration during the experiment; SETRA and Micro Electro Mechanical Systems (MEMS). These accelerometer were used for specific application which is explained as follow.

MEMS accelerometer

The three dimensional Micro Electro Mechanical Systems (MEMS) accelerometer is a waterproof accelerometer, which can be placed inside the saturated soil. A MEMS consisted of ADXL 335 chip with the dimension of 4mm×4mm×1.45mm was placed on the SEN 09269 breakout board having the dimension of 18mm×18mm×1.63mm (Bhattacharya et al. 2012). Figure 4.15 illustrates the MEMS in detail. There was a capacitor of 0.1 μ with the chip which provided frequency bandwidth up to 50 Hz. There are six connections in the breakout board; three wired connection for the X, Y, and Z direction, a ST connection for all the axis direction grounding connection, and VCC and GND connections were for power supply and ground connections, receptively (Figure 4.15).
The breakout board was placed into the small box (40mm×40mm×17mm) which called Poly Tetra Fluoro Ethylene (PTFE) and then protected with the hardening epoxy resin. The MEMS operated with a power supply within the range of 1.8–3.6 V and measured up to ±3g (Bhattacharya et al. 2012). Finally, RDP 611 signal conditioning amplifier was used to supply 3 voltage of power for the MEMS. MEMS were calibrated before and after the experiment using SETRA accelerometer. In order to calibrate the MEMS, the response of these accelerometers were correlated with the response of SETRA accelerometers and the derived calibration factor was used to calibrate MEMS accelerometers. As shown in Figure 4.16, the MEMS were placed at different depth levels of soil during dry pluviation. Therefore, it was possible to record the soil response during experiment.
SETRA accelerometer

SETRA accelerometer (141 A) was manufactured by SETRA, and consisted of ±8 g servo accelerometer. Model 141 was a linear accelerometer, which could produce a high level instantaneous DC output signal proportional to sensed accelerations ranging from static acceleration up to 3000 Hz. This type of SETRA represented a flat response between 0 to 300 Hz with the resonance frequency of about 600 Hz which were above the range of frequency for these tests. Figure 4.17 illustrates the SETRA accelerometer placed on the pile cap. SETRA was not waterproof and as a consequence could not be used inside the soil container. SETRA accelerometers which were already in-house calibrated were mounted on the side of the each pile cap along the direction of shaking in order to monitor the pile models response during shaking. Also, three SETRAs in three different directions were placed on the shake table to record the input motions (Figure 4.18).
Figure 4.17: SETRA placed on the side of the pile cap.

Figure 4.18: SETRA placed on the shaking table to record the input motion.
4.3.6 Instrumented impact hammer

The natural frequency of the pile structures was measured in different statuses of free standing, dry sand, and wet sand conditions by using an instrumented impact hammer (model 086C01). The hammer was manufactured by PCB Piezotronics and consisted of a head body containing a quartz force sensor and a handle with rubber grips. A Kistler (model 5134A) power supply was used to have a constant excitation voltage of 30VDC at a constant current excitation of 20 mA. Figure 4.19 illustrates the impact hammer and the Kistler power supply. The hammer characteristics are listed in Table 4.7. Figure 4.20 shows the natural frequency measurement test on GP22 on both free standing and dry sand situations.

![Impact hammer and Kistler power supply](image)

Figure 4.19: (a) Impact hammer and (b) Kistler power supply.

<table>
<thead>
<tr>
<th>Impact hammer characteristics</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hammer length</td>
<td>216 mm</td>
</tr>
<tr>
<td>Hammer mass</td>
<td>0.10 kg</td>
</tr>
<tr>
<td>Head diameter</td>
<td>15.7 mm</td>
</tr>
<tr>
<td>Tip diameter</td>
<td>0.063 mm</td>
</tr>
<tr>
<td>Measurement range</td>
<td>±444 N pk</td>
</tr>
<tr>
<td>Sensitivity</td>
<td>11.2 mV/N</td>
</tr>
<tr>
<td>Resonant frequency</td>
<td>≥15 kHz</td>
</tr>
</tbody>
</table>
4.3.7 Data acquisition system

To improve the quality of the data, amplification and filtering were employed. All the channels were passed through a low pass Butterworth filter set to 80 Hz. The data acquisition system consisted of four Microstar Laboratories MSXB028 analog-digital converter (ADC) cards, providing a total of 64 channels. A target frequency of 200 Hz was applied for channels sampling. However, an actual frequency of 200.64 Hz was considered during the test. The software called SIMACQ cer2.09 (HP-VEE version 4.01) was used to monitor the duration of the acquisition and sampling frequency. The outputs were exported to Matlab program to analyse the data.

4.4 Test procedure

After setting up the physical models, the natural frequency of the 4 pile models (with pile cap) were measured in 3 conditions: free standing as columns, confined by dry sand, and finally confined by saturated sand. The very low amplitude white noise motion was then applied to obtain the modal parameters of the structures i.e. fundamental frequency and damping (Test MR-1). Following the white noise tests, masses were placed on the top of the pile models one after another, followed by the measurement of the natural frequency of the pile models using instrumented hammer. The pile models having mass were subjected to the input motion to monitor the dynamic response of the physical models. The sequence of the tests were as follows:
• Pile model SP12 was the first physical model where superstructure mass was placed on its top and subjected to Christchurch earthquake with the scale factor of 0.5 (Test MR-2). Other pile models did not have masses. Also the mass was removed from pile model SP12 after its failure.

• GP22 was the next pile model to consider where the mass was placed on its top cap only (other pile models did not have masses) and was subjected to two Christchurch earthquake motion with the scale factors of 0.5 and 0.7 (Tests MR-3 & MR-4). The mass was then removed from pile model GP22 and the next test was carried out on the next pile model.

• The next pile model was GP12 where the superstructure mass was placed on its top cap only (other pile models did not have masses) and was subjected to two Christchurch earthquake with the scale factors of 0.5 and 0.7 (Tests MR-5 & MR-6). The mass was removed after GP12 failed.

• SP22 was the last pile model where the superstructure mass was placed on its top (other pile models did not have masses) and subjected to seven input motions as follows: Sine-dwell motion (Test MR-7), Christchurch earthquake (Test MR-8), Irpinia earthquake (Test MR-9), Friuli earthquake (Test MR-10), L’Aquila earthquake (Test MR-11), Northridge earthquake (Test MR-12), and finally, Christchurch earthquake with the scale factor of 1.3 (Test MR-13).

The main objective was to fully liquefy the soil and it is well accepted that the soil stiffness at full liquefaction is around 1 to 10% of its initial value. As a result, the small amount of densification due to many earthquakes will have a very little effect on the results. The purpose of the experiments were to study the effect of earthquake dynamics on the pile response. As discussed before, earthquakes are broadband with multiple frequencies. Attempts were made such the predominant frequency of some of the applied input motions coincide and get tuned with the structure frequency of the models. As liquefied soil offers damping, the frequency response is expected to be reduced and the pile modes were subjected to worst possible load cases. The applied input motions are explained in the following paragraphs.

4.4.1 Input motions

There were some pre-defined input motions in shake table facility which were used to carry out the experiments. Various numbers of input motions were considered in this
experiment which were White noise motion, Sine-dwell motion, Christchurch earthquake (2011), Irpinia earthquake (1980), Friuli earthquake (1976), L’Aquila earthquake (2009), and Northridge earthquake (1994). The input motions details are listed in Table 4.8. Brief information on each applied earthquake is implied in the following.

Table 4.8: Characteristics of the applied earthquakes

<table>
<thead>
<tr>
<th>Earthquake and Country</th>
<th>Year</th>
<th>Magnitude ( (M_w) )</th>
<th>( a_{\text{max}} ) (g)</th>
<th>Significant duration (sec)</th>
<th>Uniform duration (sec)</th>
<th>Fault type</th>
<th>Focal depth (km)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Friuli (Italy) 1976</td>
<td>6.4</td>
<td>0.35</td>
<td>19</td>
<td>14</td>
<td>Reverse</td>
<td>4-10</td>
<td></td>
</tr>
<tr>
<td>Irpinia (Italy) 1980</td>
<td>6.9</td>
<td>0.25</td>
<td>10</td>
<td>15</td>
<td>Normal</td>
<td>15</td>
<td></td>
</tr>
<tr>
<td>Northridge (US) 1994</td>
<td>6.8</td>
<td>1</td>
<td>5</td>
<td>6</td>
<td>Thrust</td>
<td>18.5</td>
<td></td>
</tr>
<tr>
<td>L’Aquila (Italy) 2009</td>
<td>6.3</td>
<td>0.32</td>
<td>11</td>
<td>11</td>
<td>Normal</td>
<td>8-9</td>
<td></td>
</tr>
<tr>
<td>Christchurch (New Zealand) 2011</td>
<td>6.2</td>
<td>1.53</td>
<td>7</td>
<td>7</td>
<td>Reverse</td>
<td>4</td>
<td></td>
</tr>
</tbody>
</table>

White noise motion

White noise motion was applied to measure the modal parameters of pile models. In this test, the modal response of the piles such as frequency and damping ratio was studied during liquefaction. Figure 4.21 shows the acceleration time history of the white noise motion. More detailed information of this motion can be found in Lombardi and Bhattacharya (2014) and Lombardi (2014).
Friuli earthquake (1976)

Friuli earthquake occurred on 6th May 1976 with the recorded magnitude ($M_w$) of 6.4 in Italy. The location of the earthquake was in the Southern Alps within the active collision zone between Eurasia and Adria. The type of the fault caused the earthquake was believed to be reverse slip fault (Aoudia et al. 2000). The focal depth was at a depth range between 4 and 10 km (Cipar 1980; Aoudia et al. 2000). Figure 4.22 shows the acceleration time history of Friuli earthquake as well as the power spectrum density of this earthquake. The power spectrum density of the motion was plotted using “pwelch” function in Matlab (Welch 1967). As can be seen the higher acceleration are between 4 to 10 seconds (Figure 4.22a). As shown in Figure 4.22b, the highest energy of the earthquake is distributed between 1.5 to 3 Hz.
The Irpinia earthquake occurred on 23rd November 1980 with the recorded magnitude ($M_w$) of 6.9 in Italy. The type of the fault was believed to be normal slip fault (Westaway, 1993). The focal depth was estimated around 15km. Figure 4.23 shows the Irpinia earthquake time history as well as its power spectrum density. As can be seen this earthquake has a wide frequency bandwidth with high energy.
Northridge earthquake (1994)

The Northridge earthquake occurred on Monday 17th January 1994 with the magnitude ($M_w$) of 6.8 on the Richter scale in United State. The focal depth was 18.5 km below the Northridge area of Los Angeles. The type of the fault caused the earthquake was thrust fault (Trifunac and Todorovska, 2013). Liquefaction failure was observed in Redondo Beach on the Pacific Ocean and also near the Los Angeles Dam. Sand boils, which is one of the sign of liquefaction, was observed at several areas on the downstream side of the Lower San Fernando Dam as well as Los Angeles Port area. Figure 4.24 shows the time history of Northridge earthquake as well as its power spectrum density. As can be seen this earthquake had a wide frequency bandwidth with high energy.

![Northridge earthquake (1994)](image)

Figure 4.24: Northridge earthquake (1994) (a) time history (b) power spectrum density.

L’Aquila earthquake (2009)

The L’Aquila earthquake occurred on 6th April 2009 with the magnitude ($M_w$) of 6.3 in Italy. The epicentre was located near the city of L’Aquila, around 95 km north-east of Rome. The focal depth was estimated to be between 8.9 km. The Fault type was believed to be normal slip (EEFIT, 2009). Time history of the earthquake as well as its power spectrum density is shown in Figure 4.25. Higher acceleration were between 1 to 11 seconds (Figure 4.25a). The highest energy of the earthquake was between 0.5 to 2.5 Hz (Figure 4.25b).
Christchurch earthquake (2011)

The Christchurch earthquake occurred on the 22nd February 2011 with the magnitude ($M_w$) of 6.2 and a focal estimated depth of 4km in New Zealand. The earthquake location was underneath the Christchurch’s Port Hills, which was approximately 8 km to the south east of the Christchurch central business district. The type of fault caused the earthquake was believed to be reverse slip fault (EEFIT 2009). The acceleration time history and power spectrum density of the earthquake is shown in Figure 4.26. Based on Figure 4.26(b) the higher energy of the earthquake occurred between 2 to 6 Hz.
**Sine-dwell motion**

The sine-dwell test, which is also known as the quasi-static test, was developed in order to derive the quasi-static condition loads. ‘Sine-dwell’ has the sinusoidal signal input accelerations with a certain frequency and amplitude which is maintained during a certain period of time. The ultimate quasi-static loads are equal to maximum amplitude of the input signal (Wijker, 2008). It is clear from Figure 4.27 that the applied Sine-dwell consisted of three phases of 10 seconds. The amplitude of acceleration increased from zero to 0.2g in the first 10 seconds. The amplitude of 0.2g was kept constant for the following 10 seconds. Finally, the amplitude decreased from 0.2 g to zero during the last 10 seconds. Contrary to earthquake which has a several frequencies, Sine-dwell motion has just one frequency. Figure 4.27b shows the frequency of the applied motion.

![Sine-dwell motion](image)

Figure 4.27: Sine-dwell motion (a) time history (b) power spectrum density.

Table 4.9 summarises the shake table experiment with more details of each experimental test. As seen in the table, the input motions were scaled in order to liquefy the sand inside the container. As already mentioned in Chapter 2, the potential of liquefaction can be evaluated by computing the factor of safety against liquefaction which is the ratio between Cyclic Resistance Ratio (CRR) and Cyclic Stress Ratio (CSR) as given in Equation 4.2. The value of CRR and CSR were calculated from the soil element test results (Chapter 7) for Redhill-110 sand and used to compute the FOS against liquefaction.

$$FOS_{liquefaction} = \frac{CRR}{CSR}$$  \hspace{1cm} (4.2)
The Factor of Safety (FOS) against liquefaction was computed for all the tests and plotted in Figure 4.28. As shown, soil was liquefied in all the tests as the FOS obtained less than one.

Figure 4.28: Factor of safety against liquefaction which was obtained for all the tests.
Table 4.9: Input motion properties applied on the structures

<table>
<thead>
<tr>
<th>Test ID</th>
<th>Input motion and Earthquake</th>
<th>Scaled factor</th>
<th>Maximum acceleration (g)</th>
<th>Time taken to reach full liquefaction (sec)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>MR-1</td>
<td>White noise</td>
<td>0.02~0.15</td>
<td></td>
<td>50</td>
<td>No mass on structures. This was carried out to understand the modal properties of the system.</td>
</tr>
<tr>
<td>MR-2</td>
<td>Christchurch (2011)</td>
<td>0.5</td>
<td>0.63</td>
<td>4.5</td>
<td>Mass applied only on SP1 to have a $T_o$ as 0.44 sec. The SP1 structure failed.</td>
</tr>
<tr>
<td>MR-3</td>
<td>Christchurch (2011)</td>
<td>0.5</td>
<td>0.63</td>
<td>4</td>
<td>Mass applied only on GP2 to have a $T_o$ as 0.28 sec. The GP2 structure did not fail.</td>
</tr>
<tr>
<td>MR-4</td>
<td>Christchurch (2011)</td>
<td>0.7</td>
<td>0.92</td>
<td>4</td>
<td>Mass applied only on GP2 to have a $T_o$ as 0.28 sec. The GP2 structure failed.</td>
</tr>
<tr>
<td>MR-5</td>
<td>Christchurch (2011)</td>
<td>0.5</td>
<td>0.63</td>
<td>5</td>
<td>Mass applied only on GP1 to have a $T_o$ as 0.41 sec. The GP1 structure did not fail.</td>
</tr>
<tr>
<td>MR-6</td>
<td>Christchurch (2011)</td>
<td>0.7</td>
<td>0.92</td>
<td>4</td>
<td>Mass applied only on GP1 to have a $T_o$ as 0.41 sec. The GP1 structure failed.</td>
</tr>
<tr>
<td>MR-7</td>
<td>Sine-Dwell</td>
<td>1</td>
<td>0.2</td>
<td>12</td>
<td>Mass applied only on SP2 to have a $T_o$ as 0.5 sec. The SP2 structure did not fail.</td>
</tr>
</tbody>
</table>
Table 4.9: Continue

<table>
<thead>
<tr>
<th>Test ID</th>
<th>Input motion and Earthquake</th>
<th>Scaled factor</th>
<th>Maximum acceleration (g)</th>
<th>Time taken to reach full liquefaction (sec)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>MR-8</td>
<td>Christchurch (2011)</td>
<td>1</td>
<td>1.53</td>
<td>4</td>
<td>Mass applied only on SP2 to have a $T_o$ as 0.5 sec. The SP2 structure did not fail.</td>
</tr>
<tr>
<td>MR-9</td>
<td>Irpinia (1980)</td>
<td>1</td>
<td>0.247</td>
<td>6</td>
<td>Mass applied only on SP2 to have a $T_o$ as 0.5 sec. The SP2 structure did not fail.</td>
</tr>
<tr>
<td>MR-10</td>
<td>Friuli (1976)</td>
<td>1</td>
<td>0.35</td>
<td>3</td>
<td>Mass applied only on SP2 to have a $T_o$ as 0.5 sec. The SP2 structure did not fail.</td>
</tr>
<tr>
<td>MR-11</td>
<td>L’Aquila (2009)</td>
<td>1</td>
<td>0.32</td>
<td>6.8</td>
<td>Mass applied only on SP2 to have a $T_o$ as 0.5 sec. The SP2 structure did not fail.</td>
</tr>
<tr>
<td>MR-12</td>
<td>Northridge (1994)</td>
<td>1</td>
<td>0.928</td>
<td>5</td>
<td>Mass applied only on SP2 to have a $T_o$ as 0.5 sec. The SP2 structure did not fail.</td>
</tr>
<tr>
<td>MR-13</td>
<td>Christchurch (2011)</td>
<td>1.3</td>
<td>1.69</td>
<td>4.5</td>
<td>Mass applied only on SP2 to have a $T_o$ as 0.5 sec. The SP2 structure failed.</td>
</tr>
</tbody>
</table>
4.5 Natural frequency of the structures

Before carrying out each of the experimental tests, the natural frequency of four pile models were estimated from free vibration tests, in which an instrumented hammer tapped the pile cap of each model and generated its free decay response. As known the natural frequency is dependent on two factors; the mass and the stiffness. From the literature, the natural frequency of Single Degree Of Freedom (SDOF) can be calculated using Equation 4.3.

\[ f_n = \frac{1}{2\pi} \sqrt{\frac{k}{m}} \]

(4.3)

where, \( f_n \) is the natural frequency of system in Hz, \( k \) is the stiffness of the system, and \( m \) is the mass of the system. The Frequency Response Function (FRF) method was considered in order to measure the natural frequency of the pile models. Basically, FRF is a mathematical relationship between the input and the output of a system which is used in vibration and modal analysis. Therefore, FRF is a function transfers signals between two points of any structure; the input excitation (input motion) and the response acceleration (output response). The FRF function can describe the relationship between these two points as a function of frequency as given in Equation 4.4.

\[ H(f) = \frac{S_{xy}(f)}{S_{xx}(f)} \]

(4.4)

where, \( S_{xy}(f) \) and \( S_{xx}(f) \) is cross spectral density in the frequency domain and the auto spectral density respectively in frequency domain and \( H(f) \) is the frequency response function (Thorby 2008). FRF is a popular method for single input (hammer impact) and single output (SETRA response). In the free vibration tests, whereby an impact hammer was as external excitation, the frequencies of the pile models were computed based on the FRF considering as output the acceleration response of the model and input the force excitation imposed by the hammer.

The measured natural frequency of the pile models with only pile cap (i.e. SP11, SP21, GP11, and GP21) in different conditions of free-standing, dry sand, and wet sand were compared in Table 4.10.
Table 4.10: Natural frequency of structures in different conditions

<table>
<thead>
<tr>
<th>Structure</th>
<th>Frequency (Hz) of pile models with pile caps and no mass</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Free standing</td>
</tr>
<tr>
<td>SP11</td>
<td>1.08</td>
</tr>
<tr>
<td>SP21</td>
<td>1.08</td>
</tr>
<tr>
<td>GP11</td>
<td>1.18</td>
</tr>
<tr>
<td>GP21</td>
<td>1.86</td>
</tr>
</tbody>
</table>

Based on the Equation 4.3, as the mass increases the natural frequency decreases. A similar change was also observed in the experiment. The natural frequency of the structure with mass was measured for each structure using hammer test in saturated sand. Figure 4.29 shows the hammer test on GP22 pile model. The natural frequency of the pile models was measured in saturated sand and compared in two conditions of i) pile models with pile cap only and ii) pile models with pile cap and mass as shown in Figures 4.30 to 4.33.
Figure 4.30 shows the natural frequency and the schematic view of single pile model with smaller diameter (SP1) in two different conditions; pile cap only (SP11) and pile cap and mass (SP12) in saturated sand. The natural frequency of SP11 was around 4.8 Hz and SP12 was around 2.25 Hz. As expected, this finding is based on the concept that “when the mass increases the natural frequency decreases”.

![Figure 4.30](image)

Figure 4.30: Natural frequency measurement for SP11 & SP12.

Figure 4.31 illustrates the natural frequency and schematic view of single pile with larger diameter (SP2). Similarly to SP1, the natural frequency decrease from 4.7 Hz with the condition of pile model with pile cap only (SP21) to 1.96 Hz for pile model with pile cap and mass (SP22).
The natural frequency of pile group with small diameter (GP1) is shown in Figure 4.32. Clearly, the natural frequency declined from 8.23 Hz for pile model with pile cap only (GP11) to 2.35 Hz for pile model with pile cap and mass (GP12).
Finally, the natural frequency and schematic view of pile group with larger diameter (GP2), is shown in Figure 4.33. The natural frequency of the model decreased from 10.78 Hz to 3.63 Hz for the conditions of pile model with pile cap only (GP21) and the pile model with pile cap and mass (GP22), respectively.
Table 4.11 summarises the difference between the amount of natural frequency for all the pile models with and without mass.

Another point from Figures 4.30-4.33 is the amplitude of the accelerations. In all four cases, the amplitude of acceleration decreased when the mass was added on the top of the pile caps. This will confirm that by increasing the mass, the response acceleration decreases.
Table 4.11: Comparison of natural frequency of structure with and without mass

<table>
<thead>
<tr>
<th>Pile model ID</th>
<th>Natural frequency (Hz) in saturated sand</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>With pile cap and no mass</td>
<td>With pile cap and mass</td>
</tr>
<tr>
<td>SP11</td>
<td>4.8</td>
<td>-</td>
</tr>
<tr>
<td>SP12</td>
<td>-</td>
<td>2.25</td>
</tr>
<tr>
<td>SP21</td>
<td>4.7</td>
<td>-</td>
</tr>
<tr>
<td>SP22</td>
<td>-</td>
<td>1.96</td>
</tr>
<tr>
<td>GP11</td>
<td>8.23</td>
<td>-</td>
</tr>
<tr>
<td>GP12</td>
<td>-</td>
<td>2.35</td>
</tr>
<tr>
<td>GP21</td>
<td>10.78</td>
<td>-</td>
</tr>
<tr>
<td>GP22</td>
<td>-</td>
<td>3.63</td>
</tr>
</tbody>
</table>

4.6 Conclusion

As earthquakes are low probability events, field measurements of instrumented pile-supported structures to verify various theories/hypothesis are not feasible. Experimental techniques such as white noise and instrumented impact hammer have been developed in this research to study soil-structure-interaction. The experimental results showed the observation and measurements as expected i.e. the natural frequency of the pile models in dry sand is more than the natural frequency in free standing condition (without the soil) as soil stiffness contributes greatly to the pile stiffness. In the case of saturated sand, the natural frequency reduced due to the lower stiffness of the saturated sand owing to the reduced mean effective stress. The effect of mass on the natural frequency of the pile models was also considered and, as expected, the natural frequency reduced in the case of having superstructure mass on the top of the pile models. Therefore, the experiments can be considered valid and the techniques can be used to investigate dynamic soil-structure-interaction.
Chapter 5
Shake table test results, analysis and discussion

5.1 Introduction

As mentioned in Chapter 4, a series of shake table tests were carried out to understand the dynamic response of soil-pile interaction. Four pile models consisting of two single piles and two pile groups of 2×2 were tested. Redhill-110 sand was used and seven different motions were applied to the pile models. Measurements were taken of pile head response, soil response, pore water pressure generation, and bending moment along the pile models. This thesis is focused on the bending moment of pile models especially in the transient phase (i.e. how does bending moment change from pre to post liquefaction states). However, soil response during these tests is also presented briefly in this chapter.
5.2 Objective of the analysis

As discussed in Chapter 2, soil liquefaction process takes a certain amount of time and depends on soil profile and earthquake characteristics. As the time period of structure changes during liquefaction (structure becomes more flexible) the bending moment of a pile may change during liquefaction. There were many aims that have been chased to have a better understanding of the pile response in liquefiable soil during earthquake events. The effect of time taken to reach liquefaction (i.e. transient phase) on different parameters such as time period change of the pile models and transience of bending moment along the pile models have been investigated. Two dynamic amplification factors consisting of the ratio between the measured maximum bending moment in transient phase over the maximum bending moment in i) pre liquefaction (factor $\eta_1$) and ii) post liquefaction (factor $\eta_2$) are presented. The effects of some parameters such as time to reach liquefaction (speed of liquefaction), and elongation of time period of structure are investigated on dynamic amplification factor.

5.3 Time period change of pile models during liquefaction

The time period of any structure (like pile foundation) changes (increases) during liquefaction and the structure becomes more flexible. Hence, it is important to understand the effect of time period change on pile foundations. As a result, the time period of pile models is obtained from the measured natural frequency. As discussed in Chapter 4, the natural frequency of pile models were measured before shaking using hammer test and was used as a natural frequency of pile model in pre liquefaction phase. The frequency of the pile models were also measured after reaching liquefaction. The Frequency Response Function (FRF) method was applied to measure the natural frequency as is explained in Chapter 4 (section 4.5).

Table 5.1 and 5.2 show the natural frequency for all pile models tested at pre and full liquefaction. As known there is an inverse relationship between natural frequency and time period (i.e. $T = \frac{1}{f_n}$ where, $f_n$ is the natural frequency and $T$ is time period of the system). The time period of pile models in pre and post liquefaction was then compared with the power spectral density of the input motion.
Alternatively, the time period of pile models in post liquefaction can be estimated and computed theoretically. Two other simplified methods were considered based on the mass ratio and unsupported length. Before liquefaction, due to the soil strength and stiffness, foundations are assumed to be rigid. During liquefaction, as liquefiable soil cannot support pile foundation, based on the depth of liquefaction the length of pile foundation becomes unsupported length. Table 5.3 compares the value of the post liquefaction time period of pile models based on measured and computed methods. As can be seen the measured values were fairly close to the value computed from based on mass ratio. However, there is a gap between the measured value and the value based on unsupported length. It seems soil stiffness plays an important role in the system stiffness. Due to the soil stiffness, the unsupported length might be reduced and the system stiffness may increase and as a result the time period decrease. As the time period in pre liquefaction ($T_{pre-\text{liq}}$) was measured using FRF method, and in order to consider the same methodology, the post liquefaction time period ($T_{post-\text{liq}}$) measured from the FRF method was chosen for further investigation.
Figure 5.1: Example of measuring the natural frequency of SP1 at full liquefaction.
Table 5.1: Time period of pile models at full liquefaction

<table>
<thead>
<tr>
<th>Pile model ID</th>
<th>Natural frequency (Hz) with pile cap only (White noise test)</th>
<th>Saturated sand</th>
<th>Full liquefaction</th>
</tr>
</thead>
<tbody>
<tr>
<td>SP11</td>
<td>4.8</td>
<td>3.84</td>
<td></td>
</tr>
<tr>
<td>SP21</td>
<td>4.7</td>
<td>1.88</td>
<td></td>
</tr>
<tr>
<td>GP11</td>
<td>8.23</td>
<td>4.12</td>
<td></td>
</tr>
<tr>
<td>GP21</td>
<td>10.78</td>
<td>4.31</td>
<td></td>
</tr>
</tbody>
</table>

Table 5.2: Time period of pile models at full liquefaction

<table>
<thead>
<tr>
<th>Pile model ID</th>
<th>Natural frequency (Hz) with pile cap and mass</th>
<th>Saturated sand</th>
<th>Full liquefaction</th>
</tr>
</thead>
<tbody>
<tr>
<td>SP12</td>
<td>2.25</td>
<td>0.78</td>
<td></td>
</tr>
<tr>
<td>SP22</td>
<td>1.96</td>
<td>0.78</td>
<td></td>
</tr>
<tr>
<td>GP12</td>
<td>2.35</td>
<td>0.39</td>
<td></td>
</tr>
<tr>
<td>GP22</td>
<td>3.63</td>
<td>0.39</td>
<td></td>
</tr>
</tbody>
</table>

Table 5.3: Comparison of post liquefaction time period of pile models at full liquefaction

<table>
<thead>
<tr>
<th>Test ID</th>
<th>Pile model ID</th>
<th>$T_{pre-\text{liq}}$ (sec)</th>
<th>$T_{post-\text{liq}}$ measured FRF function (sec)</th>
<th>$T_{post-\text{liq}}$ measured PSD function (sec)</th>
<th>$T_{post-\text{liq}}$ based on Mass ratio (sec)</th>
<th>$T_{post-\text{liq}}$ based on Unsupported length (sec)</th>
</tr>
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<tbody>
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<td>MR-2</td>
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<td>0.64</td>
<td>0.51</td>
<td>4.8</td>
</tr>
<tr>
<td>MR-3</td>
<td>GP22</td>
<td>0.28</td>
<td>1.3</td>
<td>0.6</td>
<td>0.57</td>
<td>3</td>
</tr>
<tr>
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<td>1.03</td>
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<td>3</td>
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<td>1.02</td>
<td>1</td>
<td>0.61</td>
<td>3.4</td>
</tr>
<tr>
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<td>SP22</td>
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<td>2</td>
<td>0.61</td>
<td>3.4</td>
</tr>
<tr>
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<td>SP22</td>
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<td>0.8</td>
<td>0.7</td>
<td>1</td>
<td>4</td>
</tr>
<tr>
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<td>SP22</td>
<td>0.51</td>
<td>1.7</td>
<td>1.03</td>
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<td>4</td>
</tr>
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<td>0.85</td>
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<tr>
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<td>0.9</td>
<td>0.85</td>
<td>1</td>
<td>4</td>
</tr>
<tr>
<td>MR-11</td>
<td>SP22</td>
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<td>1.5</td>
<td>1</td>
<td>1</td>
<td>4</td>
</tr>
<tr>
<td>MR-12</td>
<td>SP22</td>
<td>0.51</td>
<td>1.25</td>
<td>0.9</td>
<td>1</td>
<td>4</td>
</tr>
<tr>
<td>MR-13</td>
<td>SP22</td>
<td>0.51</td>
<td>5</td>
<td>2.6</td>
<td>1</td>
<td>4</td>
</tr>
</tbody>
</table>
5.4 Time period change of pile models

The Power Spectral Density (PSD) versus frequency of each motion was obtained for all the input motions applied in the experiment. The PSD of the input motions was obtained using the “pwelch” function in the Matlab program. The PSD graph which illustrates the energy distribution of the input motion was plotted for all the applied input motions. The natural frequency and the relevant time period of pile models is also shown in these plots for the two different phases of pre and full liquefaction. These graphs were plotted using the “3-y axis plot” command. Therefore, it was possible to have all the graphs (i.e. PSD of input motion and FRF of time period in pre and at full liquefaction) of each pile model in one graph in order to compare the time period change of the pile model during shaking. These parameters can help to describe the dynamic response of pile models during the applied motion. Figures 5.2 to 5.13 show the time period changes of the pile models as well as the power spectral density of the input motions for the tests. In these figures the black line represents the PSD of the input motion, the red line illustrates the time period of the pile model in pre-liquefaction, and the blue line identifies the time period of the pile model at full liquefaction. Each test is explained in the following paragraphs.

Test MR-2:

In this test, the small diameter single pile (SP1) was monitored. The pile properties can be found in Table 4.3 (Chapter 4). The mass was placed on model SP1 only (pile model ID: SP12). The Christchurch earthquake was scaled down by a factor of 0.5 and applied as input motion. Figure 5.2 shows the PSD of the earthquake versus frequency. As can be seen, the applied earthquake had a higher energy between 2 to 5.5 Hz, whereas the frequency of the SP12 decreased from 2.25 to 0.59 Hz (from pre to at full liquefaction). The time period of SP12 increased from 0.44 to 1.7 seconds. Model SP12 failed during this test.
Figure 5.2: Time period change at pre and full liquefaction (Test MR-2).

**Test MR-3:**

The pile group with a larger diameter (GP2) was monitored in this test. The mass was added on model GP2 only (pile model ID: GP22). The Christchurch earthquake was scaled down by a factor of 0.5 and applied as input motion to the shake table. As shown in Figure 5.3, which plots the (PSD) of the earthquake versus frequency, the earthquake had a higher energy between 2 to 5.5 Hz. The frequency of the GP22 model before liquefaction and at full liquefaction decreased from 3.62 and 0.78 Hz respectively (time period increased from 0.28 to 1.3 seconds). Model GP22 did not fail during this test.

Figure 5.3: Time period change at pre and full liquefaction (Test MR-3).
Test MR-4:

Model GP2 (group pile with large diameter) was monitored in this test. Mass was placed on GP2 only (pile model ID: GP22). Christchurch earthquake with a scale factor of 0.7 was applied to the shaking table and the model. From Figure 5.4, the earthquake had a higher energy between 2 to 5.5 Hz whereas the frequency of the model (GP22) decreased from 3.62 to 0.39 Hz. The GP22 time period increased from 0.28 to 2.56 seconds. In this test GP22 failed.

![Figure 5.4: Time period change at pre and full liquefaction (Test MR-4).](image)

Test MR-5:

In this test, the pile group with the small diameter pile (GP1) was monitored. The mass was applied on model GP1 only (pile model ID: GP12). The Christchurch earthquake was applied to the shake table by a factor of 0.5. Figure 5.5 shows the PSD of the earthquake versus frequency. As can be seen the earthquake had a higher energy between 2 to 5.5 Hz. The frequency of GP12 from pre to full liquefaction decreased from 2.35 to 0.98 Hz (i.e. time period increased from 0.42 to 1.02 seconds). Model GP12 did not fail during this test.
CHAPTER – 5 SHAKE TABLE TEST RESULTS, ANALYSIS, AND DISCUSSION

Test MR-6:

Model GP1 (group pile with small diameter) was monitored in this test. Mass was placed on GP1 only (pile model ID: GP12). Christchurch earthquake with a scale factor of 0.7 was applied to the shaking table and the model. As can be seen from Figure 5.6, the earthquake had a higher energy between 2 to 5.5 Hz. The natural frequency of GP12 decreased from 2.35 to 0.39 Hz during liquefaction. The time period of GP12 increased from 0.42 to 2.56 seconds from pre to post liquefaction during this test. GP12 failed during this test.

Figure 5.5: Time period change at pre and full liquefaction (Test MR-5).

Figure 5.6: Time period change at pre and full liquefaction (Test MR-6).
**Test MR-7:**

Model SP2 (single pile model with a larger diameter) was monitored in this test. Mass was placed on SP2 only (pile model ID: SP22). Sine-dwell motion was applied to the model. As can be seen from Figure 5.7, this motion had the highest energy at 2 Hz whereas the frequency of the model (SP22) decreased from 1.96 Hz at pre liquefaction to 1.28 Hz at full liquefaction (i.e. the time period increased from 0.58 to 0.8 seconds). SP22 did not fail during this test.

![Figure 5.7: Time period change at pre and full liquefaction (Test MR-7).](image)

**Test MR-8:**

The next test on SP2 was under Christchurch earthquake. Mass was placed on SP2 only (pile model ID: SP22). Figure 5.8 shows the PSD of the earthquake. As shown in the figure the higher energy of the motion occurred between 2 to 5.5 Hz. The frequency of model SP22 decreased from 1.96 Hz at pre liquefaction to 0.58 Hz at full liquefaction. Therefore, the time period increased from 0.51 to 1.7 seconds. SP22 did not fail during this test.

![Figure 5.8: PSD of input motion and FRF of SP22 before and after liquefaction.](image)
Figure 5.8: Time period change at pre and full liquefaction (Test MR-8).

**Test MR-9:**

The next test on SP2 was during Irpinia earthquake. Mass was placed on SP2 only (pile model ID: SP22). Figure 5.9 illustrates the PSD of the earthquake. The highest energy of the motion occurred up to around 8 Hz whereas the frequency of model SP22 decreased from 1.96 Hz at pre liquefaction to 1.2 Hz at full liquefaction. The relevant time period increased from 0.51 to 0.83 seconds. SP22 did not fail during this test.

Figure 5.9: Time period change at pre and full liquefaction (Test MR-9).
Test MR-10:

Another test on SP2 was under Friuli earthquake. Mass was placed on SP2 only (pile model ID: SP22). As shown in Figure 5.10, earthquake had a higher energy between 1.5 to 3.5 Hz. The frequency of model SP22 decreased from 1.96 Hz at pre liquefaction to 1.1 Hz at full liquefaction (i.e. the time period increased from 0.51 to 0.9 seconds from pre to post liquefaction). SP22 did not fail during this test.

![FRF of SP22 and PSD of input motion](image)

Figure 5.10: Time period change at pre and full liquefaction (Test MR-10).

Test MR-11:

L’Aquila earthquake was the next motion which was applied to model SP2. Mass was placed on SP2 only (pile model ID: SP22). As illustrates in Figure 5.11, the highest energy of the motion occurred between 0.6 to 3.5 Hz whereas the frequency of model SP22 decreased from 1.96 Hz at pre liquefaction to 1 Hz at full liquefaction (time period increased from 0.51 to 1 second). SP22 did not fail during this test.
Test MR-12:

Model SP2 was tested under Northridge earthquake. Mass was placed on SP2 only (pile model ID: SP22). As shown in Figure 5.12 the motion had a distributed density between 0 to 15 Hz. The frequency of model SP22 decreased from 1.96 Hz at pre liquefaction to 0.8 Hz at full liquefaction. Therefore, the time period of the model increased from 0.51 to 1.25 seconds from pre to at full liquefaction. SP22 did not fail during this test.
**Test MR-13:**

The large diameter of single pile (SP2) was monitored in this test. Mass was placed on SP2 only (pile model ID: SP22). The Christchurch earthquake was scaled up to 30% and applied as an input motion to the shake table. As can be seen from Figure 5.13, the applied earthquake had a higher energy between 2 to 5.5 Hz, whereas the frequency of the SP22 decreased from 1.96 to 0.39 Hz (from pre to full liquefaction). Therefore, the relevant time period of SP22 from pre to post liquefaction increased from 0.51 to 5 seconds. Model SP22 failed during this test.

![Figure 5.13: Time period change at pre and full liquefaction (Test MR-13).](image)

As can be seen from Figures 5.2 to 5.13, the frequency of the pile models were varied in post liquefaction phase. From the results, it can be noted that the frequency of the pile models (apart from pile model stiffness and mass) may depend of soil stiffness and input motion time history.

### 5.5 Bending moment of the pile models

The bending moment of the structures was obtained by using the recorded strain during the applied motion. The strain was recorded using the strain gauges which were placed along pile models as a pair in different levels (Chapter 4, Figure 4.10). The bending moment is computed based on Equation (5.1) as given.
\[
M = EI \frac{2 \left( \varepsilon_{\text{right}} - \varepsilon_{\text{left}} \right)}{D}
\]  
(5.1)

where, \( M \) is the bending moment, \( \varepsilon_{\text{right}} \) and \( \varepsilon_{\text{left}} \) are the recorded data from the right and left side of strain gauges, \( EI \) is the bending rigidity and \( D \) is the outer diameter of pile. The yield and plastic moment were calculated for the pile models and listed in Table 5.4 in order to compare the measured bending moment with the yield and plastic values. Yield and plastic moments can be calculated by Equation 5.2 and 5.3 respectively.

\[
M_y = \sigma_y Z_e
\]  
(5.2)

where, \( M_y \) is the yield moment, \( \sigma_y \) is the yield stress, and \( Z_e \) is the elastic modulus of the section.

\[
M_p = \sigma_p Z_p
\]  
(5.3)

where, \( M_p \) is the plastic moment, \( \sigma_p \) is the yield stress, and \( Z_p \) is the plastic modulus of the section.

<table>
<thead>
<tr>
<th>Pile model ID</th>
<th>Outer diameter ((D_o), \text{(m)})</th>
<th>Inter diameter ((D_i), \text{(m)})</th>
<th>Elastic section modulus, ((Z_e), (\text{m}^3))</th>
<th>Plastic section modulus, ((Z_p), (\text{m}^3))</th>
<th>Yield stress ((\sigma_y), (\text{MPa}))</th>
<th>Yield moment ((M_y), (\text{Nm}))</th>
<th>Plastic moment ((M_p), (\text{Nm}))</th>
</tr>
</thead>
<tbody>
<tr>
<td>SP12 &amp; GP12</td>
<td>0.0254</td>
<td>0.024</td>
<td>3.3×10^{-3}</td>
<td>4.27×10^{-7}</td>
<td>170</td>
<td>55</td>
<td>73</td>
</tr>
<tr>
<td>SP22 &amp; GP22</td>
<td>0.0413</td>
<td>0.0398</td>
<td>9.5×10^{-7}</td>
<td>1.23×10^{-6}</td>
<td>170</td>
<td>162</td>
<td>200</td>
</tr>
</tbody>
</table>

The bending moment for all pile models are computed and plotted separately. These plots consist of four parts which are plotted in the following order from the bottom; the acceleration of the input motion, the excess pore water pressure ratio \((r_u)\), the acceleration response of the pile head, and the bending moment at different levels of pile models. The following paragraphs present the response of pile models which were failed caused by the
applied input motions (i.e. SP12 (Test MR-2), GP22 (Test MR-4), GP12 (Test MR-6), and SP22 (Test MR-13)). The rest of the results are presented in Appendix C.

**SP12 (Test MR-2):**

Figure 5.14 shows the various responses (pile head, pore pressure generation, and bending moment along the pile) caused by the applied input motion. As shown in Figure 5.14(a), the 0.5-scaled Christchurch earthquake was applied as an input motion, as a result of which, pore water pressure inside the soil increased and caused pile head deflection and bending moment along the pile. Figure 5.14(b) shows the pore water pressure generation during applied input motion. As can be seen this pressure increased dramatically and soil liquefied within a few seconds (4~5 seconds). As shown in Figure 5.14(c), the pile head experienced more movement during the time taken to reach liquefaction as the soil gradually reduced its stiffness. The bending strain of SP12 was measured at four different levels along the pile model (Figures 5.14(d) to 5.14(g)). As shown, the maximum bending moment happened at the middle of the pile (at the level of -1.095m below soil surface). This response may normally happen in free-headed pile foundations. It was observed that the amplitude of bending moment increased from pre liquefaction and reached the maximum during liquefaction transience and reduced at post liquefaction. The amplitude of the bending moment at post liquefaction may be similar to its amplitude at the pre liquefaction phase. Figure 5.15 shows the failure photo of model SP12.
Figure 5.14: Measured bending moment along the SP12 (Test MR-2). (For the instrumental layout please refer to Chapter 4, Figure 4.10b).
The yield and plastic moment of pile model SP12 was calculated in order to compare with the results of the shake table. As already mentioned in Table 5.4 the yield moment of the section was around 55 Nm whereas the pile model failed under around 20Nm. This might be due to the reason that the pile model failed due to buckling first as it was subjected to the axial load as well. The bending moment of pile when it is subjected to the lateral load only could be different from the bending moment of pile when it is subjected to a combination of lateral and axial load.

**GP22 (Test MR-4):**

The bending moment of GP22 was calculated based on seven pairs of strain gauges along the pile. Figure 5.16 shows the pile group response and bending moment along the pile for test MR-4. As shown the maximum bending moment occurred at the top of the pile models. This type of response is normally observed for the fixed-headed pile. As excess pore water pressure increased due to the shaking, the soil lost its stiffness and became more liquid like material. Therefore, the soil could not support the embedded pile and as a result the pile demonstrated a flexible response. This process occurred in around 4 seconds in both tests. As shown the maximum bending moment happened during the liquefaction phase. In post liquefaction the amplitude of the bending moment dropped. GP22 failed during this test as depicted in Figure 5.17. The results of Test MR-3 are presented in Appendix C (Figure C.1).
Figure 5.16: Measured bending moment along the GP22 (Test MR-4).
(For the instrumental layout please refer to Chapter 4, Figure 4.10b).
Figure 5.17: GP22 failure under Test MR-4 (a) after the test and (b) after excavation.

**GP12 (Test MR-6):**

The bending moment of GP12 was calculated based on four pairs of strain gauges along the pile. Figure 5.18 shows the pile group response and bending moment along the pile for test MR-6. The maximum bending moment occurred at the top of the pile models as the pile group models are like fixed-headed pile. The maximum bending moment happened during liquefaction phase. In post liquefaction the amplitude of bending moment dropped. GP12 failed during test MR-6 as shown in Figure 5.19. The result of Test MR-5 is presented in Appendix C (Figure C.2).
Figure 5.18: Measured bending moment along the GP12 (Test MR-6). (For the instrumental layout please refer to Chapter 4, Figure 4.10c).
Figure 5.19: GP12 failure under Test MR-6 (a) after the test and (b) after excavation. (Poor quality picture is due to the fact that it was taken from video).

SP22 (Test MR-13): 

Figure 5.20 shows the response of SP22 subjected to Christchurch earthquake as an input motion. As can be seen the bending moment of SP22 was computed at seven different levels of strain gauges placed along SP22. The maximum bending moment happened at the middle of the pile for all the tests. This response is similar to the response of free-headed pile. The amplitude of bending moment reduced in post liquefaction. SP22 failed during this test as depicted in Figure 5.21. The results from the other tests on pile model SP22 are presented in Appendix C (Figures C.3-C.8).
Figure 5.20: Measured bending moment along the SP22 (Test MR-13).
(For the instrumental layout please refer to Chapter 4, Figure 4.10c).
Based on the measured bending moment, three different values of bending moment were taken from three different steps: pre-liquefaction, transient phase and the post-liquefaction. These values of bending moment were then plotted versus the pile length in order to compare the bending moment profile of each pile model in various status (Figures 5.22 to 5.33). From the figures it can be seen that the amplitude of the bending moment is increased in the transient phase and reach the maximum bending moment. However, the amplitude of bending moment after liquefaction (i.e. post liquefaction) decreased to a value which could be compared to the value in pre liquefaction phase. From all the graphs, it can be seen that the value of the bending moment might be vary along the pile. For the single piles (i.e. SP12 and SP22), the maximum bending moment happened at the middle of the pile. In pile group models (i.e. GP12 and GP22), the maximum bending moment happened at the pile head.
Figure 5.22: Bending moment profile of SP12 (Test MR-2),
(For the instrumental layout please refer to Chapter 4, Figure 4.10b).

Figure 5.23: Bending moment profile of GP22 (Test MR-3).
(For the instrumental layout please refer to Chapter 4, Figure 4.10b).
Figure 5.24: Bending moment profile of GP22 (Test MR-4).
(For the instrumental layout please refer to Chapter 4, Figure 4.10b).

Figure 5.25: Bending moment profile of GP12 (Test MR-5).
(For the instrumental layout please refer to Chapter 4, Figure 4.10c).
Figure 5.26: Bending moment profile of GP12 (Test MR-6).
(For the instrumental layout please refer to Chapter 4, Figure 4.10c).

Figure 5.27: Bending moment profile of SP22 (Test MR-7).
(For the instrumental layout please refer to Chapter 4, Figure 4.10c).
Figure 5.28: Bending moment profile of SP22 (Test MR-8).
(For the instrumental layout please refer to Chapter 4, Figure 4.10c).

Figure 5.29: Bending moment profile of SP22 (Test MR-9).
(For the instrumental layout please refer to Chapter 4, Figure 4.10c).
Figure 5.30: Bending moment profile of SP22 (Test MR-10).
(For the instrumental layout please refer to Chapter 4, Figure 4.10c).

Figure 5.31: Bending moment profile of SP22 (Test MR-11).
(For the instrumental layout please refer to Chapter 4, Figure 4.10c).
Figure 5.32: Bending moment profile of SP22 (Test MR-12).  
(For the instrumental layout please refer to Chapter 4, Figure 4.10c).

Figure 5.33: Bending moment profile of SP22 (Test MR-13).  
(For the instrumental layout please refer to Chapter 4, Figure 4.10c).
As the bending moment amplifies during transient phase to reach liquefaction, a dynamic amplification factor can be derived from the ratio of the bending moment amplitude in different considered conditions. As this amplification is caused by dynamic loading like an earthquake, this factor is called “dynamic amplification factor”. This factor was derived from the shake table test results and the effect of different parameters was studied on this factor.

### 5.6 Dynamic bending amplification factor

From the obtained results of shake table experiments, the experimental dynamic amplification factor was calculated for each pile models. The two dynamic amplification factors were defined.

1) The experimental dynamic amplification factor \(\eta_1\) computed by dividing the maximum measured bending moment in transient phase \(M_{\text{max-transient}}\) over the maximum measured bending moment in pre-liquefaction \(M_{\text{pre-liq}}\) as given in Equation 5.4.

\[
\eta_1 = \frac{M_{\text{max-transient}}}{M_{\text{pre-liq}}}
\]  

(5.4)

2) The experimental dynamic amplification factor \(\eta_2\) computed by dividing the maximum measured bending moment in transient phase \(M_{\text{max-transient}}\) to the maximum measured bending moment at full liquefaction \(M_{\text{post-liq}}\). As given in Equation 5.5.

\[
\eta_2 = \frac{M_{\text{max-transient}}}{M_{\text{post-liq}}}
\]  

(5.5)

The dynamic amplification factors were computed for all the tests. For each pile model these factors were calculated for all the levels where bending strain was recorded (4 levels for SP12 and GP12, and 7 levels for SP22 and GP22). Table 5.5 summaries the average value computed for all the tests carried out. The value of dynamic amplification factor for each test is presented in Appendix C (Tables C.1 to C.13). The experimental dynamic amplification factors were plotted for parameters such as time to reach liquefaction (i.e. speed of liquefaction), ratio of time period at pre and post liquefaction stage.
Table 5.5: The average value of the bending amplification factor for all the tests.

<table>
<thead>
<tr>
<th>Test ID</th>
<th>Pile model ID</th>
<th>Average bending moment before liquefaction ($M_1$) (Nm)</th>
<th>Average bending moment at full liquefaction ($M_2$) (Nm)</th>
<th>Average maximum bending moment at transient phase ($M_3$) (Nm)</th>
<th>Average experimental dynamic amplification factor ($\eta_1$) ($\frac{M_1}{M_1}$)</th>
<th>Average experimental dynamic amplification factor ($\eta_2$) ($\frac{M_2}{M_2}$)</th>
<th>Reference Appendix C</th>
</tr>
</thead>
<tbody>
<tr>
<td>MR-1</td>
<td>SP11</td>
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<tr>
<td>MR-12</td>
<td>SP22</td>
<td>34.3</td>
<td>32.6</td>
<td>58.2</td>
<td>2.1</td>
<td>2.1</td>
<td>Table C.12</td>
</tr>
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<td>MR-13</td>
<td>SP22</td>
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<td>75</td>
<td>163.1</td>
<td>2.7</td>
<td>2.4</td>
<td>Table C.13</td>
</tr>
</tbody>
</table>

5.7 Discussion on dynamic bending amplification factors

Figure 5.34 presents the effect of time taken to reach full liquefaction on dynamic amplification factor ($\eta_1$). In this figure the results from this research are compared to the results from the White noise test in order to present the effect of time to reach liquefaction. As can be seen, soil liquefies in 50 seconds during the White noise test whereas in real earthquakes liquefaction is observed below 10 seconds. It was observed that for a particular pile model, by increasing the time taken to reach liquefaction, the dynamic amplification factor increases. This indicates that the more time needed to obtain soil liquefaction the greater the amplification of the bending moment of the pile model (see section 5.6 for explanation). As a consequence, the more time taken to liquefy, the great the flexibility of the pile model. As time taken to reach liquefaction increases there is more possibility for pile model time period to become tuned with the input motion leading to a resonance phenomenon. The parameter of time to reach liquefaction can be alternatively present by the term of “speed of liquefaction”. Speed of liquefaction can be computed by considering the depth of liquefaction over time to reach liquefaction. Figure 5.35 presents the effect of speed
of liquefaction on dynamic amplification factor ($\eta_1$). This figure indicates that as speed of liquefaction increases the $\eta_1$ decreases. Figure 5.36 and 5.37 show the similar results for $\eta_2$.

Figure 5.34: Dynamic Amplification factor ($\eta_1$) versus time to reach liquefaction ($t_{liq}$).
Figure 5.35: Dynamic Amplification factor ($\eta_1$) versus speed of liquefaction ($v_{liq}$).

Figure 5.36: Dynamic Amplification factor ($\eta_2$) versus time to reach liquefaction ($t_{liq}$).
The dynamic amplification factors $\eta_1$ and $\eta_2$ are also plotted versus the time period elongation ratio, which can be computed by considering the ratio of time period in post liquefaction ($T_{\text{post-liquef}}$) over the time period in pre liquefaction ($T_{\text{pre-liquef}}$) as shown in Figures 5.38 and 5.39. Results show that when the time period elongation increases, $\eta_1$ and $\eta_2$ also increase. When the ratio of time period elongation is high, the flexibility response of pile models takes longer as the time to reach liquefaction is long.
Figure 5.38: Dynamic Amplification factor ($\eta_1$) versus time period elongation ratio.

Figure 5.39: Dynamic Amplification factor ($\eta_2$) versus time period elongation ratio.
The time period elongation percentage is computed by dividing the difference of time periods in pre and post liquefaction phases ($T_{\text{post}} - T_{\text{pre}}$) over the time period in pre liquefaction ($T_{\text{pre}}$) multiply by 100. The dynamic amplification factors $\eta_1$ and $\eta_2$ are plotted against the normalised time period elongation for particular time to reach liquefaction. As can be seen from Figures 5.40 and 5.41, by increasing the time period elongation $\eta_1$ and $\eta_2$ increase.

![Figure 5.40: Dynamic Amplification factor ($\eta_1$) versus stiffness elongation percentage.](image)

Figure 5.40: Dynamic Amplification factor ($\eta_1$) versus stiffness elongation percentage.
5.8 Soil response

The response of soil was recorded by using MEMS accelerometers. The MEMS were placed in different levels of the soil profile (Chapter 4, Figure 4.10). Liquefaction was considered by defining the excess pore water ratio, which is computed by dividing the excess pore water by the initial effective stress. The ratio of 1 ($R_u = 1$) indicates liquefaction phenomena. Figure 5.42 illustrates the generation of pore water pressure ratio caused by applying input motion. As shown this plot can be divided into four parts. Part (i) presents the onset of input motion with a very small amplitude. As a result of this amplitude there was no observation of excess pore water and soil response. Part (ii) demonstrates the highest amplitude of the input motion. The excess pore water pressure was generated as a result of the input motion. As can be seen from Figure 5.42(b), in a few seconds soil profile transferred from solid to liquid and the excess pore water pressure ratio reached 1 ($R_u = 1$) at the top of the soil profile and around 0.93 ($R_u = 0.93$) at the bottom of the soil profile. Liquefaction caused a higher response of soil profile as shown in Figure 5.42(c) & (d). The acceleration response of soil at -1m below soil surface was higher than the acceleration response at -0.6m.
below surface due to have a higher effective stress. This response was observed in all the tests. Due to the strong input motions the excess pore water pressure may exceed 1 ($r_u > 1$) in this part. This is due to the location of the pore pressure transducers change during shaking. As a result the estimated effective stress for calculating the excess pore water pressure ratio might be overestimated. Part (iii) shows the input motion continuing with a smaller acceleration amplitude. The soil response decreases by decreasing the input motion amplitude. In this part the excess pore water pressure ratio either remained constant or started to dissipate slowly. Finally in part (iv) when the input motion stopped the excess pore water pressure dissipated. The dissipation of excess pore water pressure which depends on soil profile density, takes some time. The dissipation process for all the tests took around 600 seconds. During this process soil profile obtains its stiffness and strength gradually. The soil response was recorded in terms of acceleration and plotted for two different levels which are presented in Appendix C (Figure C.9 to C.20).
5.9 Conclusion

A series of shake table tests have been carried out in order to better understand the transient dynamic behaviour of pile supported structures. The time period of pile models in pre and post liquefaction was measured using the Frequency Response Function (FRF) method. As observed during the experiments, soil liquefied progressively from top to bottom.
and the process was termed as transient phase. The effect of time taken to reach full liquefaction was studied, together with its impact on behaviour of pile models such as the transient bending moment along the pile. It was observed that the maximum bending moment occurred in the transient phase. As the bending moment was amplified in the transient phase, this amplification was presented by identifying dynamic amplification factors as defined by the ratio of the maximum bending moment in transient phase over the bending moment in pre liquefaction and post liquefaction (for easy implementation in the design). The effect of time taken to reach liquefaction and time period elongation of the models was also investigated. It was observed that with increasing time taken to reach liquefaction, the dynamic amplification factor of the bending moment also increased. As time taken to reach liquefaction increased, the pile models became more flexible and, as a result, the bending moment became greater due to dynamic amplification.
Chapter 6
Advanced soil element test

6.1 Introduction

Understanding the behaviour of soil under sudden pressure is one of the challenging topics in geotechnical engineering. Since Niigata earthquake (Japan, 1964), when liquefaction was considered a significant issue in structure failure during earthquake, there have been many studies carried out on understanding the resistance behaviour of the soil. However, the response of soil under earthquake loading is not fully understood and needs more research. Soil behaviour becomes a more significant issue when liquefaction occurs during an earthquake. There are many experimental approaches and methods to understand the soil behaviour (i.e. in-situ field tests and laboratory tests). For example, soil element tests are widely used in order to have a better understanding of soil behaviour under earthquake events. These tests can be carried out using different apparatus like Cyclic Triaxial, Resonant Column, Dynamic Simple Shear, and Hollow Cylinder apparatus. Cyclic Triaxial apparatus was chosen for this study as it was interested to investigate the behaviour of liquefied soil.
This apparatus is able to carry out multi-stage tests that are suitable tests to study liquefaction and post liquefaction phenomenon. The tests were carried out at the Surrey Advanced Geotechnical Engineering (SAGE) laboratory, University of Surrey. This chapter explains the objectives of the experiments followed by the test set-up procedure and sample preparation techniques. The details of the tests carried out are also listed at the end of the chapter.

### 6.2 Experimental Objectives

Fully understand the soil behaviour under sudden stress is a complex issue. This complexity might be increased, when soil is subjected under high magnitude type earthquake events. Therefore, to better understand soil behaviour under these circumstances, soils are usually being modelled either numerically or experimentally. For example, soil element test is one of the experimental type methods used to simulate the soil layers resistivity. Hence, in this study, the response of sand during liquefaction and at post liquefactions were investigated. This was done by performing a series of advanced soil element tests on four different types of sand using Cyclic Triaxial apparatus. The sand types used in this investigation were: Redhill-110 sand (UK) and silica sand No. 8 (Japan), which are commercially available sands; and Assam and Ganga sands, which are two natural sands from India. Specifics on different parts of the apparatus are presented along with the properties of the four types of sand used and sample preparation techniques.

### 6.3 Test set-up

This section presents the details of the Cyclic Triaxial apparatus, sand properties used, and sample preparation procedure.

#### 6.3.1 Cyclic Triaxial Apparatus

The Cyclic Triaxial Apparatus is an advanced machine to specifically carry out soil element tests. This apparatus is also widely used in soil mechanics type research to understand the resistance behaviour of soil during cyclic loading. This apparatus has the capability to represent the initial effective stress as well as follow up stress changes in the soil under study. One of the challengeable topics in soil mechanics is the behaviour of the liquefiable soils, especially after the liquefaction process (i.e. post liquefaction behaviour of sand). In other words, the behaviour of the soils in terms of their stiffness and strength in post liquefaction
phase needs more attention and research. The emphasis of this chapter is to further explain, one of the apparatus capabilities of allowing the performance of carrying out multi-stage tests. Therefore, for the reasons explained above, this apparatus was specifically selected to carry out a series of tests to better understand the resistance behaviour of the liquefiable soils.

The cyclic triaxial apparatus consists of different parts such as cyclic triaxial frame, electro mechanical motor, back pressure controller, cell pressure controller, and data acquisition system. Figure 6.1 shows a schematic illustration of the view of the apparatus as well as the associated connections. For more clarity, Figure 6.2 shows a photo of the apparatus and its different connected parts.

![Figure 6.1: Schematic view of Cyclic Triaxial Apparatus.](image-url)
Following is the brief explanation of each of the different critical parts of the apparatus.

**Back pressure controller**

The back pressure controller is a digital signal controller based on hydraulic actuator for the precise regulation and measurement of liquid pressure. The pressure and volume of the liquid located in soil sample voids is measured by back pressure controller during the test (GDS Handbook, 2012). As shown in Figure 6.3, the controller consists of different parts of gear box, piston, pressure cylinder, and smart keypad. The controller can be operated either by using the GDSLab software (the software to run the apparatus) or smart keypad. In order to obtain fully saturated sample, the controller is normally filled by de-aired water and used during saturation process.
Cell pressure controller

The cell pressure controller is the pneumatic regulator which consists of two valves to fill and release the air. There is a pressure transducer, which monitors the pressure inside the cell chamber and connects to the data acquisition system. The controller is supplied with air from the external air supply source as water is not permitted for use with this controller. The maximum output pressure that the system can hold is about 1000 kPa (GDS Handbook, 2012). Figure 6.4 shows the front view of the cell pressure controller.

Data acquisition system

The experimental data is logged and stored by the data acquisition system through four main channels. Figure 6.5 shows the front view of the data acquisition system and illustrates the associated channels. As is shown, there are four channels; three to record data from load cell, pore pressure transducer, and cell pressure transducer, and the fourth channel is to
measure optional pressure transducer, which is not used in these series of tests performed here (GDS Handbook, 2012). The apparatus is operated by using the software called “GDSLab”. All the acquired data can also be exported to external software programs such as Excel or Matlab for further analyses.

![Figure 6.5: Data acquisition system.](image)

**Cyclic triaxial load frame and actuator**

Figure 6.6 shows the Cyclic Triaxial load frame. This strong frame consists of a base, two vertical columns, an adjustable beam, and an electro mechanical actuator unit, which is placed on the adjustable beam. The displacement measurement can be measured through the encoder of the motor. The position of the load ram can be controlled by a quadrature encoder which is located into the motor unit. The motor unit includes a transformer, DC power supply and the brushless motor controller. The maximum load capacity is 10kN (GDS Handbook, 2012).
6.3.2 Soil properties

Four types of sand were used to carry out these experimental investigations; two commercially available sands, Redhill-110 sand (UK) and silica sand No. 8 (Japan), which are typically used in laboratory studies; and two natural sands from India, Assam sand and Ganga sand. Figure 6.7 shows the microscopic photos of the sands while their index properties, based on ASTM standards (D4253, 2006; D4254, 2006; and D854, 2010), are listed in Table 6.1. The grain size distribution curves of the sands are shown in Figure 6.8, where it was observed that all sands had uniform grain size distribution and low fines content. Also indicated in the figure the range of grain size distributions, which are deemed to have high possibility of liquefaction, based on past historical earthquakes in Japan and being stipulated in the design code for port and harbour facilities (PHRI, 1997). It can be seen from the figure that all the sands used were highly liquefiable.
Figure 6.7: Microscopic photo of sands used in the tests: (a) Redhill-110 sand; (b) Silica sand No. 8 (Japan); (c) Assam sand (India); and (d) Ganga sand (India).

Table 6.1: Properties of sands used in the tests

<table>
<thead>
<tr>
<th>Sand name</th>
<th>Specific gravity, $(G_s)$</th>
<th>$D_{50}$ (mm)</th>
<th>$e_{\text{max}}$</th>
<th>$e_{\text{min}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Redhill-110 (UK)</td>
<td>2.65</td>
<td>0.18</td>
<td>1.035</td>
<td>0.608</td>
</tr>
<tr>
<td>Silica sand No. 8 (Japan)</td>
<td>2.65</td>
<td>0.16</td>
<td>1.385</td>
<td>0.797</td>
</tr>
<tr>
<td>Assam sand (India)</td>
<td>2.68</td>
<td>0.30</td>
<td>0.962</td>
<td>0.622</td>
</tr>
<tr>
<td>Ganga sand (India)</td>
<td>2.67</td>
<td>0.35</td>
<td>1.003</td>
<td>0.8534</td>
</tr>
</tbody>
</table>
Figure 6.8: Particle size distribution curves of the sands used plotted with respect to range of particle sizes observed to be susceptible to liquefaction (Modified from PHRI, 1997).

### 6.4 Test procedure

As known in the literature, liquefaction phenomenon happens during sudden motions such as earthquake or blast. Consequently, during these fast motions, there is no adequate time for the generated pore water (i.e. excess pore water) to dissipate. Therefore, this behaviour can represent the undrained condition of the soil.

Free field stress soil condition can be simulated in laboratory tests such as cyclic triaxial tests. An element of saturated soil has been consolidated for a long term before an earthquake happens (Figure 6.9a). Vertical and horizontal effective stresses are applied to the soil element. The vertical effective stress for a certain depth is calculated by unit weight of soil \( \gamma' \) multiplied by depth \( z \) as given below.

\[
\sigma'_v = \gamma'z
\]  

(6.1)

where \( \sigma'_v \) is the vertical effective stress.
The horizontal effective stress is obtained by multiplied the vertical effective stress by the lateral earth pressure coefficient \((K_o)\).

\[
\sigma'_h = K_o \sigma'_v
\]  
(6.2)

where, \(\sigma'_h\) is the horizontal effective stress.

Figure 6.9b illustrates the Mohr circle in free field state and the amount of effective confining stress is shown like a point. During an earthquake, this soil element is subjected to undrained loading, due to the shear stress cycles. This condition can be simulated in triaxial apparatus (Figure 6.9c). Therefore, cyclic triaxial apparatus was used to carry out a series of soil element tests. Figure 6.9d illustrates the stress state on Mohr circle in both compression and extension cyclic loading positions.

Figure 6.9: Stress state of soil element in free field and laboratory test.
6.4.1 Sample preparation

Preparation of a sample may probably the most important phase of the soil element test. It is usually a time consuming procedure, which takes usually half a day to one full day to make a sample (depends on the type of soil and test performed). Sample preparation usually consists of several steps; making a sample, saturation process, and the consolidation phase. These steps are explained in more details as follows.

6.4.1.1 Making soil sample

Soil sampling is considered the most challenging step in the sample preparation process. Ishihara (1996) proposed three different methods for making a sample: moist placement (wet tamping), dry deposition, and water sedimentation. Figure 6.10 illustrates the schematic of the three methods.

In the dry pluviation method, the density of sample is controlled by adjusting the funnel height or the rate of fall (Kolbuszewski (1948a,b). In other words, the prepared sample would be denser either by increasing the funnel height or reducing the rate of fall. In the dry pluviation method, the desired density is actually being controlled by the compaction procedure. In this process, pluviation is continued layer by layer, followed by compaction of
each layer. It is believed that, perhaps due to the upper soil compaction process, the lower layers would be compacted slightly more than upper layers. In order to avoid this problem, Ladd (1978) recommended to compact the lower layers slightly less than the upper layers.

Methods of sample preparation can affect on liquefaction resistance. Mulilis et al. (1977) and Tatsuoka et al. (1986), carried out a series of tests by using different methods of sample preparation. They showed that the samples that were prepared by dry pluviation had the least liquefaction resistance. Dry pluviation represents sand particle contacts under the gravity field. This is similar to the sedimentation process which occurs in real field conditions (Towhata, 2008).

In these series of experiments, dry pluviation was applied primarily in order to pour the mould. The sample size was 100mm in diameter and 200mm in height. Figure 6.11 illustrates the schematic position of the mould, the membrane, the porous disc, and the soil sample itself.

![Figure 6-11: Schematic of membrane, mould and soil sample position on triaxial apparatus pedestal.](image)

To have a fully saturated sample, de-aired water was used during the sample preparation process. Water was stored in a tank and de-aired by using vacuum pump (Figure 6.12). Before starting to make a sample, it is important to check and make sure that all the tubes and valves are connected to the triaxial pedestal are being de-aired. Therefore, all of them were flushed by using de-aired water. Figure 6.13 shows how to de-air the valves and tubes by flushing de-aired water through them. Silicon grease was also put around the pedestal to reduce the friction between the pedestal and membrane. In this case, membrane can be placed around
the pedestal much easier. A porous disc was then placed on the pedestal. Figure 6.14 shows these steps.

Figure 6.12: (a) water tank and (b) vacuum pump.

Figure 6.13: Flushing de-aired water through (a) cell chamber drainage valves and (b) back pressure valve as well as top cap.
A membrane was placed around the pedestal and sealed by using two O-Rings around it. The membrane was then passed through the two split mould which was then placed around the pedestal. The membrane was folded down around the top side of the mould as shown in Figure 6.15b.
Figure 6.15: membrane and O-ring position.

As explained earlier, in dry pluviation method, soil particles should be poured from the certain height and compacted layer by layer to obtain the desire relative density. For this reason, a funnel was used for pouring soil particle and was held at the certain height (Figure 6.16a) and compacted layer by layer in order to obtain the desired relative density.
As is shown in Figure 6.17, when the mould was filled with sand, another porous disc was placed on the top of the sample followed by the top cap. A small negative pressure (i.e. vacuum) of \(-10\text{kPa}\) was applied to the sample in order to avoid any damage to the sample when the mould was gently removed from around the sample as shown in Figure 6.18. The Vylastic sleeve (Figure 6.18b) was then placed on the top cap. This sleeve was employed to attach the load ram to the top cap. When the load ram was placed on the top cap, the sleeve was placed around both load ram and top cap followed by vacuuming the air between them. Therefore, these two components were attached and moved together during loading.
Figure 6.17: Schematic of the porous disc and top cap position.
Figure 6.18: (a) vacuum pressure and (b) Take off the mould and place the Vylastic sleeve.

The cell chamber was then placed and fixed on the triaxial base and filled with the de-aired water. As illustrated in Figure 6.19, a small gap was left between the level of the de-aired water inside the cell chamber and the cell pressure valve which was on the top of the cell chamber and connected to the cell controller. The reason to leave this gap was that the cell pressure controller was functioned with air and could be damaged if water reached inside the controller. That gap helped the user to make sure that the water could not go to the controller.
The next step was to replace the negative pressure inside the sample (i.e. -10kPa) with the external positive pressure around the sample. Hence, the applied negative pressure was swapped with the positive pressure around the sample (i.e. cell pressure). In this case, the sample was held by the applied external pressure which is cell pressure. As sample was made by using dry pluviation, the soil sample consisted of the sand particles and air bubbles. Therefore, in order to have a fully saturated sample, these air bubbles should be replaced by de-aired water. Flushing the soil sample can help to replace the air bubbles with de-aired water. The sample was then ready for saturation process.

### 6.4.1.2 Saturation

Next step of the sample preparation process was saturation. In this step, the cell and back pressures were applied to the sample simultaneously (see Figure 6.20). It should be noted that the cell pressure should be always greater than back pressure otherwise the sample can be lost. The difference between the cell and back pressures (i.e. effective stress) is normally around 20-30 kPa as was kept 20kPa in these tests. The duration of saturation process could be different for various types of soil as for example, for the fine sand it might take around 24 hours to saturate.
The degree of saturation should be measured after the saturation process. Skempton (1954), presented the increment of pore water pressure under undrained stress. He proposed an equation to explain the degree of saturation, as given in Equation 6.3.

\[ \Delta u = B(\Delta \sigma_3 + A(\Delta \sigma_1 - \Delta \sigma_3)) \]  

where \( \Delta u \) is the pore water pressure increment, \( \Delta \sigma_1 \) and \( \Delta \sigma_3 \) are the maximum and minimum of principal stress increment, respectively and A and B are skempton pore water pressure parameters. When the stress increases isotropically, the maximum and minimum of principal stress increment might be the same (i.e. \( \Delta \sigma_1 = \Delta \sigma_3 \)). As a result, B-value could be defined as the following equation (Equation 6.4);

\[ B = \frac{\Delta u}{\Delta \sigma_3} \]  

The B-value equal or greater than 0.95 is considered for the fully saturated samples. The sample might be left under the higher pressure to be saturated, if the desired B-value was not achieved. Saturated sample might be used in either drained or undrained tests where volume change and pore water pressure generation could be measured in drained and undrained tests, respectively.
6.4.1.3 Consolidation

Consolidation is the last step of sample preparation process. A soil sample can be consolidated either isotropically or anisotropically, however the isotropically consolidation was considered in this research. In this phase of sample preparation, the cell pressure was applied to the sample whereas the back pressure was kept constant. The pressure difference between the cell and the back pressures might be the final target pressure for the desired effective stress (Figure 6.21). The process was considered finished when the volume change was less than 0.005mm³ over a period of 5 minutes (Figure 6.22). The duration of consolidation was significantly dependent on the soil relative density and the target mean effective stress.

Figure 6.21: Consolidation process in cyclic triaxial apparatus.
6.4.1.4 Multi-stage test

Figure 6.23 illustrates the schematic diagram of the testing scheme adapted for the multi-stage soil element test ($q$ represents the deviator stress). In these tests, undrained stress-controlled sinusoidal cyclic loading with frequency of 0.1 Hz was initially applied in order to liquefy the soil sample. A few tests were carried out in order to understand the effect of frequency of loading on cyclic response of sand and as shown in Figure 6.24 the cyclic response of sand is independent of frequency of loading. The similar results were also presented by Yasuda and Soga (1984) and Hyodo et al. (1998). Therefore, to reduce the effect of viscosity the frequency of 0.1 Hz was chosen to apply cyclic load. The amplitude of the cyclic load was varied for the cases investigated. This cyclic load was stopped when the onset of liquefaction was monitored. The onset of liquefaction depends on the soil density: for loose to medium sand, the onset of liquefaction occurs when the condition of zero effective stress is achieved, i.e. the “initial liquefaction” as proposed by Seed and Lee (1966); while for dense sand, the onset of liquefaction is defined as the development of 5% double amplitude of axial strain (Ishihara, 1993). Note that in dense sand, the condition of zero effective stress occurred only momentarily. Once the specimen was deemed to have liquefied, strain-controlled monotonic load was then applied under undrained condition to obtain the stress-strain curve of the liquefied sand. The monotonic load was applied at a rate of 0.1% axial strain per minute. Such multi-stage tests on the four types of sands were conducted under
different conditions of initial relative densities, effective confining stress, and applied cyclic deviator stress (in order to have a different levels of cyclic stress ratios, CSR). Table 6.2 lists the conditions considered in the tests performed. Note that the effective confining pressure investigated ranged from 50-150kPa, corresponding to the usual depths of soil considered for liquefaction analysis.

Figure 6.23: Testing scheme adopted for the multi-stage element test.
Figure 6.24: Effect of frequency of loading on Cyclic Stress Ratio (CSR) of sand

Table 6.2: List of multi-stage soil element tests

<table>
<thead>
<tr>
<th>Test ID</th>
<th>Sand type</th>
<th>Relative density, $D_r$ (%)</th>
<th>Effective confining stress, $\sigma_c$ (kPa)</th>
<th>Cyclic deviator stress, $q$ (kPa)</th>
<th>Cyclic Stress Ratio, (CSR)</th>
</tr>
</thead>
<tbody>
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<td>MR-1</td>
<td>Redhill-110</td>
<td>30</td>
<td>97</td>
<td>30</td>
<td>0.154</td>
</tr>
<tr>
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<td>105</td>
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<td>0.143</td>
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<tr>
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<td>30</td>
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<td>30</td>
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### Test Results

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### 6.5 Conclusion

Soil element tests are widely used to obtain soil parameters for design purpose. Therefore, a series of advanced soil element tests were carried out using Cyclic Triaxial apparatus which is widely used to understand the behaviour of the soil during cyclic loadings. Multi-stage tests (advanced tests) were considered in order to study liquefaction and post-liquefaction processes as the soils were liquefied first by undrained stress controlled cyclic loading. As observed, the cyclic response of soil was independent from the frequency of loading. As a result, cyclic loading was applied with a low frequency in order to reduce the effect of viscosity. The undrained strain controlled monotonic load was then applied to understand the post-liquefaction behaviour of sands.
Chapter 7
Advanced soil element test results, analysis, and discussion

7.1 Introduction

Understanding the behaviour of soil under earthquake events has been a challenging topics in geotechnical engineering. Although there have been many studies and investigations on this critical issue, better understanding of the soil response under earthquake loading, especially when liquefaction process occurs during an earthquake event, is required. The response of liquefiable soils has been studied by many investigators; however, the behaviour of liquefiable soil during post liquefaction process needs to be more well understood, especially the responses such as post-liquefaction stiffness and strength. This chapter
explains the studies on stress-strain relation of sands during post-liquefaction stage through performing series of advanced element tests using Cyclic Triaxial apparatus.

7.2 Objective of the analysis

In this research, several series of multi-stage soil element tests were conducted on four different types of sand where the specimens were subjected to undrained monotonic shearing condition after full liquefaction has been achieved. The considered sands were reconstituted at different relative densities, consolidated under various effective confining stresses and were made to liquefy under different levels of cyclic shear stress ratio (CSR). The obtained post-liquefaction stress-strain curve was modelled in terms of the initial shear modulus ($G_1$), critical state shear modulus ($G_2$) and a parameter called post-dilation shear strain ($\gamma_{\text{post-dilation}}$), which is related to the dissipation of excess pore water pressure during the monotonic shearing of the liquefied sand. Subsequent examination indicated that the above mentioned three parameters, and consequently the post-liquefaction stress-strain curve of the sand, were mainly affected by the initial relative density while the effect of initial effective confining stress was negligible, at least, within the range considered in the tests. Thus, the parameters to model the post-liquefaction behaviour can be expressed in terms of the initial relative density of the sand.

7.3 Results and discussion

7.3.1 Undrained cyclic response

Results of the multi-stage soil element tests are discussed in this section represent the response of the different types of sands used in this investigation. Figures 7.1-7.8 show the results corresponding to medium-dense sand samples ($D_r = 50\%$), which were isotropically consolidated under 100kPa effective confining stress and cyclically sheared with 30kPa deviator stress. Figures 7.1, 7.3, 7.5, and 7.7 represent the stress paths of the samples during multi-stage test and stress-strain curves during the cyclic phase, while Figures 7.2, 7.4, 7.6, and 7.8 depict the variation of the excess pore water pressure ratio, effective confining stress and axial strain with the number of cycles of cyclic loading, respectively.

Figure 7.1 shows $p'-q$ stress path and the stress-strain curve of the Redhill-110 sand sample under multi-stage test. These $p'$ and $q$ stresses represent the mean effective stress and the
deviator stress, respectively (Roscoe et al. 1958). These stresses can be calculated by the following equations:

\[ p' = \frac{(\sigma'_a + 2\sigma'_c)}{3} \]  

(7.1)

\[ q = \sigma'_a - \sigma'_c \]  

(7.2)

where \( \sigma'_a \) and \( \sigma'_c \) are the axial and the effective confining stresses, respectively.

As seen in the figure, the development of the axial strain in the sample was slow during the early part of cyclic loading; then large axial strain mobilised and the sample liquefied at nearly 10 cycles. Figure 7.2 shows the changes in excess pore water pressure ratio \( (u_r) \), effective confining stress \( (\sigma'_c) \), and axial strain \( (\varepsilon_a) \) with increasing number of cycles. With the generation of the excess pore water pressure, the effective confining stress decreased; consequently, the axial strain in the soil increased. The onset of liquefaction was observed at the condition of zero effective stress (i.e. \( u_r = 1 \)).

Figure 7.1: Cyclic behaviour of Redhill-110 sand: (a) stress path; and (b) deviator stress versus axial strain during cyclic phase.
Figure 7.2: Variation of (a) excess pore water pressure ratio; (b) effective confining stress; and (c) axial strain with the number of cycles of loading for Redhill-110 sand.

The multi-stage test results for the medium dense sample of Japanese silica sand No. 8, which was isotropically consolidated under 100kPa and sheared under 30kPa of deviator stress, is shown in Figures 7.3 and 7.4. As seen in Figure 7.3, the mean effective stress decreased from the initial value towards zero during the undrained cyclic loading. The stress-strain curve apparently closed the loops at the beginning of the shearing; however, once the sample liquefied (after about 6 cycles), the loops became butterfly in shape and large deformation occurred. Figure 7.4 shows the changes in excess pore water pressure ratio ($r_u$), effective confining stress ($\sigma'_c$), and axial strain ($\varepsilon_a$) with increasing the number of cycles. From the data in this figure, it is clear that the sample was liquefied at the condition of zero effective stress (i.e. $r_u = 1$).
Figure 7.3: Cyclic behaviour of Japanese silica sand No.8: (a) stress path; and (b) deviator stress versus axial strain during cyclic phase.

Figure 7.4: Variation of (a) excess pore water pressure ratio; (b) effective confining stress; and (c) axial strain with the number of cycles of cyclic loading for Japanese silica sand.

Figures 7.5 and 7.6 display the results for medium dense Assam sand under similar initial effective confining pressure and amplitude of cyclic deviator stress. From data in Figure 7.5, it appears that the sand liquefied after around 23 cycles while the data behaviour in Figure 7.6, clearly showed that the sample liquefied with the development of 5% double amplitude
axial strain. Furthermore, the condition of momentary zero effective stress was also observed in this test.

Figure 7.5: Cyclic behaviour of Assam sand: (a) stress path; and (b) deviator stress versus axial strain during cyclic phase.

Figure 7.6: Variation of (a) excess pore water pressure ratio; (b) effective confining stress; and (c) axial strain with the number of cyclic loading for Assam sand.
Finally, the experimental test results for Ganga sand under the same conditions are presented in Figures 7.7 and 7.8. From these data, it can be seen that the behaviour of this sand is similar to the Assam sands in terms of the onset of liquefaction; however, the notable difference was that it liquefied in only 20 cycles.

Figure 7.7: Cyclic behaviour of Ganga sand: (a) stress path; and (b) deviator stress versus axial strain during cyclic phase.

Figure 7.8: Variation of (a) excess pore water pressure ratio; (b) effective confining stress; and (c) axial strain with the number of cyclic loading for Ganga sand.
In this study, a specimen of Ganga sand with the same relative density and effective confining stress (50% and 100kPa, respectively) was tested under higher deviator stress (40kPa). The results obtained are plotted in Figures 7.9 and 7.10 for comparison. As shown in Figure 7.9, Ganga sand under higher deviator stress (i.e. 40kPa) liquefied in less number of cycles. Therefore, under constant relative density and effective confining stress, as the deviator stress increased, the number of cycles required to initiate liquefaction decreased. Data presented in Figure 7.10b shows the condition of momentarily zero effective stress due to cyclic mobility was observed. Therefore, the onset of liquefaction process was considered to correspond to the 5% double amplitude axial strain.

Figure 7.9: Cyclic behaviour of Ganga sand under deviator stress of 40kPa: (a) stress path; and (b) deviator stress versus axial strain during cyclic phase.
Figure 7.10: Variation of (a) excess pore water pressure ratio; effective confining stress; and (c) axial strain with the number of cyclic loading for Ganga sand ($q=40kPa$).

The cyclic response of sands has already been presented for the medium dense sands. In order to compare the behaviour of loose and dense sands, a portion of the results from the Redhill-110 and Japanese silica sand No. 8 are presented as follow.

**Loose sand**

Studies concerning loose samples of Redhill-110 and Japanese silica sand No. 8 sand ($D_r = 30\%$) were tested under multi-stage undrained loadings. The sands were isotropically consolidated under 100kPa pressure of confining stress condition. The samples were then cyclically sheared under 30kPa pressure of deviator stress. Figure 7.11 and 7.13 show the graph of stress path from the multi-stage test and the stress-strain curve. Figure 7.12 and 7.14 present the variation of the excess pore water pressure ratio, effective confining stress and axial strain with the number of cycles of cyclic loading. Results from these figures show that by increasing the number of cycles, pore water pressure increased and therefore, the confining effective stress reduced. As a consequence, the axial strain increased when the effective stress reached zero and therefore the onset of liquefaction also observed in the condition of zero effective stress.
Figure 7.11: Cyclic behaviour of Redhill-110 loose sand: (a) stress path; and (b) deviator stress versus axial strain during cyclic phase.

Figure 7.12: Variation of (a) excess pore water pressure ratio; (b) effective confining stress; and (c) axial strain with the number of cyclic loading for Redhill-110 loose sand.
Figure 7.13: Cyclic behaviour of Japanese silica sand No. 8 loose sand: (a) stress path; and (b) deviator stress versus axial strain during cyclic phase.

Figure 7.14: Variation of (a) excess pore water pressure ratio; (b) effective confining stress; and (c) axial strain with the number of cyclic loading for Japanese silica sand No. 8 loose sand.

*Dense sand*

Studies regarding dense samples of Redhill-110 and Japanese silica sand No. 8 sand ($D_r = 70\%$) were tested under multi-stage undrained loading. The sands were isotropically consolidated under 100kPa pressure of confining stress. The samples were then cyclically...
sheared under 30kPa of deviator stress. Figures 7.15-7.18 show the results obtained from these experimental tests. From the results, it can be interpreted that the momentary zero effective stress was observed for these dense type sand materials.

Figure 7.15: Cyclic behaviour of Redhill-110 dense sand: (a) stress path; and (b) deviator stress versus axial strain during cyclic phase.

Figure 7.16: Variation of (a) excess pore water pressure ratio; (b) effective confining stress; and (c) axial strain with the number of cyclic loading for Redhill-110 dense sand.
Figure 7.17: Cyclic behaviour of Japanese silica sand No. 8 sand: (a) stress path; and (b) deviator stress versus axial strain during cyclic phase.

Figure 7.18: Variation of (a) excess pore water pressure ratio; (b) effective confining stress; and (c) axial strain with the number of cyclic loading for Japanese silica sand No. 8 dense sand.

**Cyclic Stress Ratio (CSR)**

CSR is the parameter that explains the behaviour of sandy soil under cyclic loading and liquefaction resistance. Liquefaction resistance can be evaluated by consideration of cyclic...
stress ratio. As discussed in Chapter 2, this parameter is the ratio of shear stress over effective confining stress \(CSR = \frac{\tau}{2\sigma_c}\) used in the triaxial test. According to this ratio, the CSR for all the carried out tests were calculated and plotted versus the number of the cycles to obtain 5% double amplitude of axial strain in order to consider either flow liquefaction or cyclic mobility. Figures 7.19 and 7.20 show the CSR values versus the number of cycles to reach 5% double amplitude of axial strain. As seen in these figures, the results from this study are also comparable to the results presented by Hyodo et al. (1998) on Toyoura sand for loose, medium, and dense sands. From the results, it can be interpreted that for the particular relative density by decreasing CSR the number of cycles to reach 5% double amplitude of axial strain increased. In the case of considering the specific CSR for a sample with different relative densities, as the relative density decreased the number of cycles to reach 5% double amplitude of axial strain also declined.

![Figure 7.19: Cyclic Stress Ratio (CSR) versus number of cycles to 5% double amplitude of axial strain for loose sand.](image-url)
The term Cyclic Resistance Ratio (CRR) is also used in order to present the soil resistance. This parameter can be considered as a cyclic stress ratio required for 5% double amplitude axial strain in 20 cycles. Based on the typical number of cycles in earthquake, 10 to 20 cycles are considered to present CRR (Ishihara 1993). Therefore, CRR values for this experimental research were estimated and plotted versus relative density of sand in Figure 7.21. The results from this research compared with the results presented by Ishihara (1996) on Toyoura sand. As seen in this figure, the soil resistance increased as the soil relative density increased.
Figure 7.21: Cyclic Resistance Ratio (CRR) versus relative density of sand.

An interesting point to note in terms of sand behaviour is its response after full liquefaction process. In the experimental tests, this response could be expected during the undrained monotonic loading after reaching liquefaction, induced by undrained cyclic loading. The next section focuses on the stress-strain response of liquefied sand and the discussion on the observed response.

7.3.2 Post-liquefaction characteristics of sand

During undrained cyclic loading, the mean effective stress of sand decreased from the initial value, which was applied during the consolidation stage, to zero; at this stage, liquefaction occurred. In the case of zero effective stress, the contact between the sand particles may likely be lost and the particles appeared to be floating in water. Figures 7.22-7.25 show the post-liquefaction response of the four different types of sand in terms of variation of deviator stress and excess pore water pressure with axial strain. From data presented in these figures, it can be seen that the sand specimen had no initial stiffness up to a certain level of axial strain. When the undrained monotonic load was applied to the liquefied sand, the sand sample showed very low stiffness at the beginning of the loading, until a certain level of axial strain was reached. After that, the resistance of the sand specimen
increased dramatically due to the dilatancy induced by the particle rearrangement. The axial strain, when such increase in resistance occurred, depends on the initial density of the sand.

![Graph showing the post liquefaction response of Redhill-110 sand (CSR=0.15). Variation of: (a) deviator stress; and (b) excess pore water pressure ratio ($r_u$) with axial strain.]

Figure 7.22: Post liquefaction response of Redhill-110 sand (CSR=0.15). Variation of: (a) deviator stress; and (b) excess pore water pressure ratio ($r_u$) with axial strain.
Figure 7.23: Post liquefaction response of Japanese silica sand (CSR=0.15). Variation of: (a) deviator stress; and (b) excess pore water pressure ratio ($r_u$) with axial strain.

Figure 7.24: Post liquefaction response of Assam sand (CSR=0.15). Variation of: (a) deviator stress; and (b) excess pore water pressure ratio ($r_u$) with axial strain.
To investigate the effect of initial effective confining stress on the post-liquefaction behaviour of the sands, Japanese silica sand No. 8 ($D_r = 50\%$) was tested under three different levels of initial effective confining stress: 50, 100, and 150kPa pressure. As shown in Figure 7.26, the confining stress affected the stress-strain relation of the liquefied sand, especially on the slope of the curve when the sand dilated, which was denoted as the critical state shear modulus ($G_2$). With the increase in the level of confining stress, $G_2$ is increased (i.e. $G_{2(1)} > G_{2(2)} > G_{2(3)}$). Similar response was also reported by Lombardi et al. (2014) on their tests performed on Redhill-110 sand. However, because of the limited tests conducted involving other confining stress levels, the impact of effective confining stress on the post-liquefaction behaviour is not pursued further in this study. More tests, however, are planned to investigate this issue in the future studies.
As discussed earlier, when the liquefied sand is sheared monotonically, the sand particles can be brought to contact with each other again, and as a consequence, the inter-locking occurs after the condition reaching a certain amount of axial strain phase. Figure 7.27(a) shows a typical stress-axial strain curve of liquefied sand. During the initial stage of monotonic loading, the stiffness of the sand was almost negligible, indicating that practically no shear strength exist; with continuous straining, the strength was mobilised when a certain level of axial strain was reached (in the figure, this value is about 4% axial strain). In order to study the post-liquefaction behaviour, this axial strain-deviator stress curve was converted to shear strain-shear stress curve which is shown in Figure 7.27b. Following equations were used to estimate shear stress and shear strain in the conversion process:

$$\tau = \frac{q}{2}$$  \hspace{1cm} (7.3)

$$\gamma = (1 + \nu)\varepsilon_u$$  \hspace{1cm} (7.4)

where $\tau$ and $\gamma$ are shear stress and shear strain, respectively, while $\nu$ is the Poisson’s ratio which can be considered as 0.5 for undrained condition.

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Figure 7.26: The effect of initial effective confining stress on post liquefaction behaviour of Japanese silica sand No. 8.

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Japanese silica sand No. 8

$\sigma_c = 150$ kPa  
$\sigma_c = 100$ kPa  
$\sigma_c = 50$ kPa
In the data plotted in Figure 7.27b, the term $G_2$ corresponds to the critical state shear modulus during post-liquefaction stage of sand, which is discussed later in this section. To investigate the post-liquefaction stress-strain behaviour of the sand further, a portion of the plot is magnified (Figure 7.27c) to clearly see the response of liquefied sand at the beginning of the shearing process. It clearly shows that the curve had an initial slope (i.e. initial shear modulus) at the beginning of the loading, which is called “$G_1$”. In terms of excess pore water response, as depicted in Figures 7.27(d) and 7.27(e), the excess pore water pressure ratio started to decrease gently from the initial value of 1.0 at the start of monotonic loading. Data in this figure shows that when a certain level of axial strain was reached, the excess pore water pressure ratio decreased significantly. Again, this portion of the plot is also magnified in Figure 7.27f in order to better clarify the response observed. It is also clear that at this level of shear strain, when the excess pore water pressure started to significantly decrease, the same level of shear strain obtained when the shear stress increased remarkably. This shear strain, called “$\gamma_{\text{post-dilation}}$” in this study, corresponding to the strain when the stiffness increased dramatically. As shown in Figure 7.27f, this point was the intersection of two tangent lines in the excess pore water pressure ratio versus shear strain curve. Note that Dash (2010) also approximated a similar parameter to be related to the shear strength at a very small level (i.e., 1 kPa pressure) and referred to it as the “take-off” shear strain with the assumption that it was the start of the dilative response of liquefied sand. In this study, based on the tests performed, during the post-liquefaction process, the sand sample started to dilate when the excess pore water started to dissipate. This dilation can happen at different levels of shear stress. Therefore, the point where the excess pore water started to dissipate was chosen to represent the post-dilation shear strain of the sample.

Furthermore, looking back closely at Figures 7.22, 7.23, and 7.25, it can be noticed that the post-liquefaction response of initially very dense sample was different from that of medium-dense and loose samples. For example, the response of soil with initial relative density of 80% was different from those of similar sands but with lower relative densities. It seems that very dense sand has a significantly higher stiffness from the beginning of monotonic loading (i.e., $\gamma_{\text{post-dilation}} = 0$). The void ratio in very dense sand was almost at a minimum value (i.e. the soil structure is mainly consisted of sand particles); therefore, during liquefaction, the amount of water between the sand particles in dense sample was quite less due to the minimum void ratio. Therefore, the sand particles do not completely float in that minimum water content levels (i.e., there was expected to be small contact between the sand
particles). As a consequence, there was no initial zero stiffness for very dense sand in the post-liquefaction behaviour. Alternatively, because of the dense packing, dilatancy immediately occurred in dense liquefied sand once shearing was applied.

Figure 7.27: Example of post-liquefaction stress-strain and excess pore water pressure ratio of Redhill-110 sand in normal and magnified situation: (a) axial strain versus deviator stress; (b) shear strain versus shear stress; (c) magnified shear strain-shear stress curve; (d) axial strain versus excess pore water pressure ratio; (e) shear strain versus excess pore water pressure ratio; and (f) magnified shear strain versus excess pore water pressure ratio.

In summary, as shown in Figure 7.28, the stress-strain behaviour of the liquefied sand can be modelled as bi-linear curve and be defined in terms of the following three parameters: the initial shear modulus \((G_1)\), critical state shear modulus \((G_2)\), which are shown in Figure 7.28a; and post-dilation shear strain \((\gamma_{\text{post-dilation}})\), the shear strain when the soil starts to dilate in post-liquefaction state (Figure 7.28b). From data presented in Figures 7.22-7.25, it would also appear that each of these parameters were dependent on the initial relative density of the sands. Such dependency is discussed in detail below.
Figure 7.28: Post-liquefaction behaviour of liquefied sand: (a) shear strain versus shear stress; and (b) shear strain versus excess pore water pressure ratio.

**Initial shear modulus** ($G_1$)

In this study, the value of the initial shear modulus obtained directly from the shear stress-shear strain curve of liquefied sand, as shown for example in Figure 7.27c. The values of $G_1$ were plotted against the relative density, as shown in Figure 7.29. It can be seen that when the relative density increased, the initial shear modulus $G_1$ also increased. Under constant volume conditions, the denser sand samples had apparently more sand particles and when they were under sheared pressure, the shear resistance prior to the post-dilation strain was expected to be larger, as more sand particles were available to provide the higher resistance.

The correlation relationships derived from the experiments between the relative densities and $G_1$ values is given by Equation 7.5.

$$G_1 = 2.82e^{(0.041D_r)}$$  \hspace{1cm} (7.5)

It should be noted, however, that this correlation is valid only for the relative densities considered in the test series performed (i.e. $30\% < D_r < 75\%$).
Post dilation shear strain ($\gamma_{\text{post-dilation}}$)

In the excess of pore water pressure ratio versus shear strain plot, the magnitude of the shear strain when the sand started to dilate are called “post dilation shear strain”. From data in Figures 7.22b-7.25b, it is clear that the location of post dilation shear strain was dependent on the relative density of sand. To further elucidate on such relation, the effect of the initial relative density of the sands on $\gamma_{\text{post-dilation}}$ was investigated, and the results were summarised in Figure 7.30. As seen, this strain level generally decreased as the relative density of the sand increased. At this condition, the void ratio of very dense sand was almost at the minimum levels (i.e. the soil matrix mainly consisted of sand particles) and therefore, during liquefaction, the amount of water between the sand particles was quite less due to the very small void ratio and the sand particles were not expecting to completely float in the water medium. As a consequence, there was no initial zero stiffness for very dense sand in the post liquefaction behaviour. Moreover, the dilatancy could be immediately occurred in the dense sand and therefore shear strength was recovered immediately with particle rearrangement.
Based on the experimental data obtained, the correlation between post-dilation shear strain and relative sand density can be expressed by Equation (7.6):

\[ \gamma_{\text{post-dilation}} = 43.45 - 9.8 \ln(D_r) \]  \hspace{1cm} (7.6)

As the sand samples considered in these series of tests had relative densities between 30% to 75%, the above correlation is therefore valid only within the range tested (i.e. 30% < \( D_r \) < 75%).

Figure 7.30: Post-dilation shear strain versus relative density relation.

**Critical state shear modulus (\( G_s \))**

The critical state shear modulus of the liquefied sand was calculated directly from the undrained monotonic test, as illustrated in Figure 7.27b. The post-liquefaction response for liquefied soil is dilative and as the soil sample is sheared following the critical state line, therefore, the obtained shear modulus is so called “critical state shear modulus”. The values obtained for all the sand types tested were plotted versus the relative densities (Figure 7.31). As shown in this graph, there was a linear relationship between the relative density and the critical state shear modulus; \( G_s \) increased with the increase in the soil’s relative density. By
considering the constant volume condition, the dense sand samples might show dilative
tendency which could be manifested with the development of negative pore water pressure.
On the other hand, looser sand samples might show lesser tendency to dilate, and therefore
were considered to have lesser shear modulus. Based on the trend observed, the correlation
between the critical shear modulus and the soil relative density is obtained from the following
Equation.

\[ G_c = 150.1D_r + 343.7 \]  \hspace{1cm} (7.7)

Again, it is important to note that the above correlation is valid only for the relative
densities considered in the test series (i.e. 30% < \(D_r\) < 75%).

![Figure 7.31: Critical state shear modulus (\(G_c\)) versus relative density relation.](image)

7.3.3 Shear modulus changes from pre to post-liquefaction stage

At the initial state of the saturated sand (i.e. prior to the cyclic loading), the maximum
shear modulus of the sand was defined as \(G_0\), (i.e., shear modulus at small strain). In this
study, the \(G_0\) values for all test specimens were calculated from the first cycle of loading
during the cyclic loading phase (i.e., when the sample is still behaving elastically). This is illustrated in Figure 7.32, as the slope of the first cycle of the shear stress-shear strain curve indicated the value of $G_0$. As the soil softened due to excess pore water pressure generation, the mean effective stress decreased and approached zero, resulting in very low stiffness of the liquefied soil. When the liquefied sand was subjected to undrained monotonic load, the sand initially showed very low initial stiffness when compared to its maximum shear modulus. From Figure 7.33, the initial shear modulus after liquefaction ($G_1$) was reduced to $1/10000$ of the maximum shear modulus. Similar results were also presented by Yasuda et al. (1995, 1998). During the post-liquefaction monotonic loading, the liquefied sand gradually recovered its stiffness. The critical state shear modulus ($G_2$) obtained was about $1/10$ of the maximum shear modulus. It should be noted that these results are from the undrained soil element tests; however, in the real grounds situation, where drainage is possible, the critical state shear modulus could reach the maximum shear modulus after sometime, with the dissipation of excess pore water pressure. The critical shear modulus may even be exceeded, if the sand densifies as a result of re-consolidation.

![Figure 7.32: $G_0$ measured from the first cycle of the deviator stress versus axial strain during cyclic loading.](image-url)
7.3.4 Undrained monotonic response

Undrained monotonic response of saturated sand is dependent on its relative density ($D_r$). Loose to medium dense saturated sands normally show contractive response whereas dense saturated sands show dilative response under undrained monotonic loading. The four different types of sand were made with different relative densities and isotropically consolidated under 100kPa of effective stress. The samples were then sheared under undrained monotonic loading. Figure 7.34 illustrates the stress path of these sands under undrained monotonic loading. In the figure the star signs indicate the transformation phase from softening to hardening phase. As shown, based on the sand relative density the undrained response was slightly different as; Japanese silica sand No. 8 and Ganga sand show loose to medium dense sand behaviour whereas Redhill-110 sand and Assam sand illustrate dense sand behaviour. Figure 7.35 present the axial strain-deviator stress curve of these sands during undrained monotonic loading.
Figure 7.34: Stress path of sands during undrained monotonic loading.

Figure 7.35: Deviator stress-axial strain of sand during undrained monotonic loading.
7.4 Application of post-liquefaction stress-strain curve

This section describes the application of post-liquefaction stress-strain curve. This curve can be used in evaluating lateral spreading of liquefied soil, liquefaction-induced settlements of structures and other liquefaction-related ground deformations. This curve can also be converted to p-y curve in order to analyse soil-structure interaction using Winkler method. In order to demonstrate the application of post-liquefaction stress-strain curve, this section explains the method of converting post-liquefaction stress-strain curve to p-y curve. The derived p-y curves were then assigned to Winkler springs in SAP2000 to back analyse the shake table test results.

7.4.1 Post-liquefaction stress-strain curve to p-y curve

The post-liquefaction stress-strain curve obtained from this experiment was used to derive p-y curve. The transformation of post-liquefaction stress-strain curve to p-y curve is schematically shown in Figure 7.36 where three parameters are required: \( M_s \), \( N_s \), and \( D \) (pile diameter). \( M_s \) and \( N_s \) are scaling parameters and further details of obtaining them can be found in Dash (2010) and Bouzid et al. (2013). It must be mentioned that \( M_s \) and \( N_s \) are based on Mobilizable Strength Design (MSD) concept developed by Bolton (2012). Rouholamin et al. (2015) carried out back analyses on the shake table test using Winkler approach. Figure 7.37 shows p-y curves obtained based on the above formulation for 25.4mm diameter pile which was used in shake table test. In the same figure empirical p-y curve taking p-multiplier as 0.33 was also plotted together with API non-liquefied sand. They compared the measured amount of bending moment of pile model with the two types of p-y curves; empirical p-y curve using API (2000) and proposed p-y curve obtained from the soil element test. The results showed that the proposed p-y curves which was based on the properties of liquefied soil provide a reasonable prediction of the bending moment in the pile. Therefore, the proposed p-y curve obtained from the soil element test was chosen for the further analysis on the shake table test results.

The next section aims to compare the pile models bending moment measured in shake table test with the numerical analyses results using proposed p-y curves.
7.4.2 Back analyse of shake table test results

The results from the shake table tests were simulated using SAP2000 software using "Beam on Non-linear Winkler Foundation" model as described in Chapter 2. The single piles (i.e. SP12 and SP22) and the pile groups (i.e. GP12 and GP22) from the shake table experiment were modelled as a beam section. Mass was considered on the top of the pile head. Due to high
axial load, non-linear P-delta analysis has been carried out. Link elements were drawn to represent Winkler springs. The fully fixed condition was applied to restrain the pile model base. Base shear force was applied to the top of the pile models. Figure 7.38 shows the pile models in SAP2000 ver.15. In this analysis the proposed p-y curves have been considered for non-liquefiable and liquefiable soil to represent pre and post liquefaction behaviour of soil as shown in Figure 7.39. These p-y curves were obtained for both pre liquefaction (from the undrained monotonic results of soil element test on saturated sand) and full liquefaction (from the undrained monotonic results of soil element test on liquefied sand) based on the method presented by Bouzid et al. (2013). These p-y curves were assigned to the model springs to represent the soil.

Figure 7.38: Pile models for numerical analysis in SAP2000 ver.15
Figure 7.39: Pre and post liquefaction p-y curve comparison obtaining by soil element test (80 cm below the soil surface)

Figure 7.40 to 7.51 compare the results obtained from the numerical analysis based on proposed p-y curves along with the measured values from the shake table test. As can be seen, there was a good agreement between the experimental and the numerical results. Therefore, it can be mentioned that the obtained p-y curves from the soil element test can present the better soil behaviour in either pre or full liquefaction states.

Figure 7.40: Comparing proposed p-y curve to shake table result (bending moment profile of SP12, test MR-2).
Figure 7.41: Comparing proposed p-y curve to shake table result (bending moment profile of GP22, test MR-3).

Figure 7.42: Comparing proposed p-y curve to shake table result (bending moment profile of GP22, test MR-4).
Figure 7.43: Comparing proposed p-y curve to shake table result (bending moment profile of GP12, test MR-5).

Figure 7.44: Comparing proposed p-y curve to shake table result (bending moment profile of GP12, test MR-6).
Figure 7.45: Comparing proposed p-y curve to shake table result (bending moment profile of SP22, test MR-7).

Figure 7.46: Comparing proposed p-y curve to shake table result (bending moment profile of SP22, test MR-8).
Figure 7.47: Comparing proposed p-y curve to shake table result (bending moment profile of SP22, test MR-9).

Figure 7.48: Comparing proposed p-y curve to shake table result (bending moment profile of SP22, test MR-10).
Figure 7.49: Comparing proposed p-y curve to shake table result (bending moment profile of SP22, test MR-11).

Figure 7.50: Comparing proposed p-y curve to shake table result (bending moment profile of SP22, test MR-12).
7.5 Conclusion

The study results revealed that as liquefaction occurred, due to the excess pore water pressure generation, the effective stress decreased. As a consequence, the shear modulus of the sand samples dropped from the maximum shear modulus to a very low value (nearly zero). When the liquefied sand was monotonically sheared, the liquefied sand recovered its stiffness gradually in a manner which depends on the initial relative density of the sample. Based on the multi-stage testing conducted on four different types of sand materials, the stress-strain behaviour of liquefied sand can be modelled as bi-linear curve. This modelling process was basically defined in terms of the three following parameters: the initial shear modulus ($G_i$), critical state shear modulus ($G_2$), and post-dilation shear strain ($\gamma_{post-dilation}$), which was the shear strain when the soil started to dilate in post-liquefaction state. Results related to this modelling process also showed that each of these parameters was a function of the initial relative density of the sands. Therefore, by increasing the relative sand density, $G_i$ and $G_2$ increased; however, $\gamma_{post-dilation}$ decreased. Based on these tests results, correlations between these parameters and the relative densities of sand were established. Thus, if the

![Figure 7.51: Comparing proposed p-y curve to shake table result (bending moment profile of SP22, test MR-13).](image-url)
relative density of sand was estimated empirically from, for example, penetration resistance or any other means, the three post-liquefaction parameters can be approximated and the post-liquefaction behaviour can be defined.

Results also showed that as the soil liquefied, the initial shear modulus of sand \( (G_i) \) reduced to about \( 1/10000 \) of the maximum shear modulus \( (G_0) \). Consequently, during the post-liquefaction monotonic loading, the liquefied sand gradually recovered its stiffness. The critical state shear modulus \( (G_c) \) obtained was about \( 1/10 \) of the maximum shear modulus value. It should be noted that these results were determined from undrained soil element tests; however, in real ground situations where drainage is possible, the critical state shear modulus could reach the maximum shear modulus after sometime, with the dissipation of excess pore water pressure. Incidentally, the critical shear modulus may even be exceeded in situations, if the sand densifies as a result of re-consolidation process.

The focus of this research was to investigate and describe the post-liquefaction response of the sand materials in terms of their stress-strain curve. It was anticipated that the stress-strain curve obtained can be used in evaluating lateral spreading of liquefied soil, liquefaction-induced settlements of structures and other liquefaction-related ground deformations. Furthermore, this curve can also be converted to p-y curve in order to analyse soil-structure interaction using Winkler method. The results of back analysis of shake table test using Winkler approach and the proposed p-y curve showed a good agreement with the measured value from the shake table test. Thus, findings from these experimental tests provided critical information and helped to better examine the post-liquefaction behaviour of the sand through the simplified stress-strain relation presented in this research investigation.
Chapter 8
Design method of pile foundations during seismic liquefaction

8.1 Introduction

Based on the results presented in Chapter 5, it has been demonstrated that time to reach liquefaction may play an important role in the performance of pile foundation. When earthquake occurs, it takes a certain time for soil to liquefy which is dependent on soil profile and earthquake characteristics. During the transience to reach full liquefaction, the time period of a structure changes (increases) from $T_{\text{pre-} \text{liq}}$ (i.e. time period of structure at pre-liquefaction phase) to $T_{\text{post-} \text{liq}}$ (i.e. time period of structure at post-liquefaction phase). As discussed in Chapter 5, the more the time taken to reach liquefaction the more the possibility
of structure damage as there is time for resonance to set in. The design method of pile foundation in all the current codes of practice such as Eurocode 8 (1998, 2003), JRA (1996), NEHRP (2000), is based on bending failure mechanism without any consideration of dynamics.

In this chapter a new framework of design method of pile foundation in seismic liquefaction is proposed based on the understanding developed in this study. The aim of this independent chapter is to propose a new practical design method which can be used in any seismic areas. Therefore, firstly, the response of pile foundations in seismic liquefaction is reviewed followed by presenting Winkler approach and the practical method to obtain p-y curve. The proposed design method is explained step by step and finally, an example of pile design is provided using the proposed method to demonstrate the method.

8.2 Pile foundation response during transient phase to reach liquefaction

When an earthquake occurs in a seismic area having loose to medium dense saturated sand, pore water pressure starts to increase and the effective stress reduces simultaneously causing liquefaction. As a consequence, during this process the time period of a structure increases from $T_{\text{pre-\text{liq}}}$ to $T_{\text{post-\text{liq}}}$ and also the damping ratio from $\zeta_{\text{pre-\text{liq}}}$ to $\zeta_{\text{post-\text{liq}}}$. Figure 8.1 schematically illustrates the modal parameter change in pile foundation during seismic liquefaction.

Figure 8.2 schematically compares the pile-soil interaction in pre and post liquefaction conditions. To simplify the model, the structure is considered as a single degree of freedom and the pile is modelled as a free standing columns which is fixed at a depth of fixity, $D_f$ (a particular depth below the ground surface). The equivalent shear force may be applied to the top of pile to represent the seismic loading (Bhattacharya and Goda, 2013). As in practice piles are usually analysed as laterally loaded beams using Beam on Nonlinear Winkler Foundation model (i.e. Winkler method), this method is explained in details in the following.
Figure 8.1: Time period and damping ratio change during liquefaction.
Figure 8.2: Schematic view of response of structure in pre and post liquefaction.

### 8.3 Winkler method

Winkler method (i.e. spring) is a common method to analyse soil-pile interaction. In Winkler method (Beam on Non-linear Winkler Foundation) of analysis of piles, the pile-soil interactions are represented by a set of nonlinear soil springs: p-y springs (commonly known as curves incorporate the lateral pile-soil interaction), t-z springs (models the shaft resistance i.e. pile-soil friction) and q-z spring (models the end-bearing interaction). Figure 8.3 shows a simple model of a pile which can be analysed using any standard structural software and can incorporate advanced features such as P-delta effects, non-linearity in the material of the pile. For any load or displacement applied to the pile either at the pile head (represents inertia load from the superstructure) or along the pile, the required analysis outputs are pile deflection, rotation, bending moment, shear and soil reaction. However, undoubtedly the critical inputs for a realistic analysis are the springs which represent the interactions. This section deals with p-y springs/curves for seismically liquefied soil and explores method for its construction.

p-y springs are generally constructed using a set of scaling rules as prescribed by codes of practice and necessary input parameters are obtained from stress-strain of the soil.
8.3.1 Obtaining p-y curve

Basically, soil properties which can be obtained from the stress-strain curve obtained from soil tests are assigned to Winkler springs. In static analysis, non-liquefiable p-y curve is employed to assign to the springs. These non-liquefiable p-y curves can be either obtained from the empirical equation suggested by codes of practice (e.g. API2000) or from the soil stress-strain curve from the soil element test (as explained in Chapter 7). In practice, p-y curves are obtained from codes of practice, see for example API (2000) and the input required is the stress strain of the soil. As discussed earlier in Chapter 2 the shape of the p-y curve for sand and clay is similar to their stress-strain behaviour and the reason is explored by Bouzid et al. (2013).

As soil liquefies, the stress-strain curve of soil varies during liquefaction period. The shape of stress-strain curve of soil in pre and post liquefaction stage is presented in past research. However, the stress-strain behaviour of liquefiable soils during liquefaction period was proposed in this research. As discussed in chapter 7, the liquefied p-y curve can be obtained using two methods; i) from soil element test and ii) Mechanics based p-y curve proposed by Dash 2010. As the aim of this chapter is to propose a design method which can be used in
different situations therefore, as carrying out soil element test may not be possible in some cases, the mechanics based p-y curve proposed by Dash 2010 is presented here in order to obtain p-y curve for liquefied soil.

8.3.2 Mechanics based p-y curve for liquefied sand

The Lateral Pile-Soil Interaction (LPSI) is very much dependent on the p-y springs and as a result it is necessary to understand the stress-strain of liquefied soil. Dash (2010), proposed a simple post liquefaction stress strain curve as shown in Figure 8.4. He introduced four key parameters to present the liquefiable curve. These parameters are take-off shear strain, initial shear modulus, critical state shear modulus, and maximum shear stress. Each of these parameters are explained as follow. More information of these parameters can be found in Dash (2010).

Take-off shear strain ($\gamma_{to}$)

During undrained monotonic loading, the shear strain equivalent to 1kPa of shear stress is called take-off strain.

Initial shear modulus ($G_1$)

The secant shear modulus in the first section of the post liquefaction stress strain curve is called initial shear modulus.

Critical state shear modulus ($G_2$)

The tangent shear modulus is relevant to when the soil is sheared following the critical state line. The tangent shear modulus is approximately constant.

Maximum shear stress ($\tau_{max}$)

The maximum shear stress can be calculated theoretically. As can be seen in Figure 8.4, there are three states for $\tau_{max}$: (i) possible minimum excess pore water pressure, (ii) minimum non negative pore water pressure during post liquefaction shearing, and (iii) residual strength of soil which is obtained from the back analysis of case studies.
The maximum shear stress can be theoretically calculated based on three different conditions (Dash, 2010).

1. The maximum theoretical maximum shear stress as given by Equation 8.1.

$$\tau_{\text{max}(1)} = \frac{M_c(p'_{im} + 100kPa)}{2}$$  \hspace{1cm} (8.1)

   where, $p'_{im}$ is the initial overburden pressure and $M_c$ is the stress ratio and based on critical angle of friction ($\phi_{cs}$) as given in Equation 8.2;

$$M_c = \frac{6 \sin \phi_{cs}}{3 - \sin \phi_{cs}}$$  \hspace{1cm} (8.2)

2. The maximum shear stress in condition of non-negative excess pore water pressure can be calculated by Equation 8.3.

$$\tau_{\text{max}(2)} = \frac{M_c p'_{im}}{2}$$  \hspace{1cm} (8.3)
(3) The maximum shear stress can be calculated by the residual strength as given in Equation 8.4. The shear stress can be mobilized when a soil sample was monotically strained in a very large strain which is called as the residual strength of soil ($s_u$).

\[ \tau_{\text{max}(3)} = s_u \quad (8.4) \]

There has been much research on strength of liquefied soils (e.g. Poulos et al. 1992; Seed 1987; Stark and Mesri 1992). Among these research studies, Olson and Stark (2002) defined the strength of liquefied soil which was not related to laboratory experiments. They mentioned “the liquefied shear strength ($s_u$) is defined as the shear strength mobilised at large deformation after liquefaction is triggered in saturated contractive sandy soils.” They proposed the relationship between the liquefied strength ratio based on normalised SPT blowcount ($N_{1-60}$), as shown in Figure 8.5. This figure is used to estimate the liquefied strength of soil.

Figure 8.5: Liquefied strength ratio based on normalised SPT blowcount ($N_{1-60}$), (Olson and Stark, 2002).
As already explained there are three different conditions that can be considered to calculate $\tau_{\text{max}}$. Therefore, it is important to choose the right condition to calculate $\tau_{\text{max}}$.

Based on Dash (2010) discussion, $\tau_{\text{max}(1)}$ is the maximum shear stress which is referred to the possible minimum excess pore water pressure (i.e. absolute vacuum condition). This condition is less likely to happen in real field. $\tau_{\text{max}(3)}$ comes from back analysis of liquefied soil from the past case studies of flow failure. Therefore, the dilative response has not been considered and is more likely to underestimate the soil strength. This condition can be considered for shallow depth soils with no impermeable top layer such as slope failure during earthquake. $\tau_{\text{max}(2)}$ can be considered in the deeper depth of strata where the dilative behaviour is more likely to happen due to the undrained boundary condition. Knappett and Madabhushi (2009) presented a similar concept based on a series of centrifuge tests. As a consequence, the maximum shear stress can be calculated based on shallow or deep depth.

Klar and Randolph (2008) explained the failure pattern of soil in the case of laterally loaded pile. They presented the wedge type and a flow around type of failure for shallow depth and deep depth respectively. American Petroleum Institute guideline (API, 2011) suggested a similar concept of shallow and deep depth based on angle of friction to estimate the lateral ultimate capacity of pile foundation either in clay or sandy soil. The critical depth ratio ($\beta$) is considered as a boundary to distinguish between shallow and deep depth and plotted versus relative density of soil (Dash, 2010). There is also another ratio of $\left(\frac{h}{D}\right)$ which can be compared with $\beta$. In this ratio, $h$ is soil depth at the particular depth and $D$ is pile diameter. Based on this comparison, the maximum shear stress can be estimated.
The \( p-y \) curve for liquefied soil can be created by having four main parameters of \( \gamma'_{so}, G_1, G_2, \) and \( \tau_{\text{max}} \). The input parameters in order to create the \( p-y \) curve for liquefied soil are listed in Table 8.1. As can be seen the input parameters are consisted of three main parts; soil, site and pile details.

Table 8.1: Input parameters to create \( p-y \) curves for liquefied soil (Dash, 2010)

<table>
<thead>
<tr>
<th>Input parameters</th>
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</thead>
<tbody>
<tr>
<td>Soil</td>
</tr>
<tr>
<td>Site</td>
</tr>
<tr>
<td>Pile</td>
</tr>
<tr>
<td>• Relative density, ((D_r))</td>
</tr>
<tr>
<td>• Critical angle of friction, ((\varphi_s))</td>
</tr>
<tr>
<td>• Pile outer diameter, ((D))</td>
</tr>
</tbody>
</table>

Based on the input parameters the calculation to create \( p-y \) curve are summarised in Table 8.2.
Table 8.2: Calculation steps to create p-y curve (summarised from Dash 2010)

<table>
<thead>
<tr>
<th>Steps</th>
<th>Description</th>
<th>Calculation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Estimating Stress ratio, ( M_c )</td>
<td>[ M_c = \frac{6 \sin \phi_s}{3 - \sin \phi_s} ]</td>
</tr>
<tr>
<td>2</td>
<td>Calculating ( N_{1-60} )</td>
<td>[ D_r = 21 \left[ \frac{N}{0.7 + \frac{\sigma_v}{98}} \right] ] (Meyerhof, 1957) [ N_{1-60} = \frac{N}{\frac{\sigma_v}{98}} ] (Lias &amp; Whiteman, 1986)</td>
</tr>
<tr>
<td>3</td>
<td>Obtaining residual strength of soil, ( s_u )</td>
<td>This can be obtained from Figure 8.5 (Olson and Stark, 2002)</td>
</tr>
<tr>
<td>4</td>
<td>Obtaining critical depth ratio, ( \beta )</td>
<td>This can be obtained from Figure 8.6 (Dash, 2010)</td>
</tr>
<tr>
<td>5</td>
<td>Computing actual to critical depth ratio</td>
<td>[ \frac{h}{D} / \beta ]</td>
</tr>
<tr>
<td>6</td>
<td>Estimating ( G_{max} )</td>
<td>[ G_{max} = 219 k_{2, max} \sqrt{\rho'} ] (Seed and Idriss, 1970) ( k_{2, max} ) is soil modulus coefficient and can be estimated by the values suggested by Seed and Idriss (1970) as given below.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>( D_r(%) )</td>
</tr>
<tr>
<td></td>
<td></td>
<td>( k_{2, max} )</td>
</tr>
<tr>
<td>7</td>
<td>Estimating ( \gamma_{to} ), ( G_1 ), ( G_2 ) and ( \tau_{max} )</td>
<td>[ \gamma_{to} = 89 - 20 \ln(D_r) ] [ G_1 = \frac{1}{\gamma_{to}} ] [ G_2 = \frac{G_{max}}{5 \sqrt{\rho'}} ]</td>
</tr>
<tr>
<td>Steps</td>
<td>Description</td>
<td>Calculation</td>
</tr>
<tr>
<td>-------</td>
<td>-------------</td>
<td>-------------</td>
</tr>
<tr>
<td>8</td>
<td>Assuming scaling factors, ((N_s, M_s))</td>
<td>(N_s) is scaling factor for stress and is based on pile-soil interface; for smooth interface = 9.2 and for rough interface = 11.94 (M_s) is scaling factor for strain and is 1.87 for fully liquefied soil More details of these factors can be found in Dash (2010) and Bouzid et al. (2013).</td>
</tr>
<tr>
<td>9</td>
<td>Estimating (p_1, y_1, P_u) and (y_u)</td>
<td>(p_1 = N_s 1.25\gamma t'oD) (y_1 = \frac{1.25\gamma t'oD}{M_s}) (P_u = N_s \tau_{\text{max}}D) (y_u = \left(1.25\gamma t'o + \frac{\tau_{\text{max}}}{G_2}\right) - \frac{G_1 1.25\gamma t'o}{M_s} \times \frac{D}{M_s})</td>
</tr>
<tr>
<td>10</td>
<td>Estimating p-y curve</td>
<td>(p = \omega \frac{p_1}{y_1} y + A(1 - \omega) \left{ \frac{p_u + p_1}{2} + \frac{p_u - p_1}{2} \tanh \frac{2\pi}{3(y_u - y_1)} \left( y - \frac{y_u + y_1}{2} \right) \right} ) Where (\omega) is a weight function and can be calculated by the given equation; (\omega = \frac{1}{2} \left[ 1 - \tanh \left( \frac{6\pi}{y_u} \left( y - \frac{4y_1 + y_u}{6} \right) \right) \right] ) (A) is constant (\left{ \begin{array}{ll} A = 0 \quad &amp; \text{for } y = 0 \ A = 1 \quad &amp; \text{for } y \neq 0 \end{array} \right. )</td>
</tr>
</tbody>
</table>
8.4 Design method of pile foundation embedded in liquefiable areas

As discussed from the observations from the pile model test (i.e. shake table test), the maximum bending moment of pile is occurred during transient phase to reach liquefaction. Based on the observed results from the experiment, Figure 8.7 schematically illustrates the bending moment profile along the pile foundations (free-headed and fixed headed piles) in three different phases of pre liquefaction, transient phase, and post liquefaction.

Figure 8.7: Bending moment profile along the pile for (a) free headed pile and (b) fixed headed pile.

As p-y curves are obtained from the soil properties, and the soil behaviour is not stable during liquefaction transformation, therefore due to that reason it is difficult to predict the properties of soil in transient phase condition. However, the soil properties in two conditions of pre and post liquefaction can be obtained from soil element test. As Winkler method is a pseudo-static non-linear method, it is more likely impossible to design pile foundation during transient phase to reach liquefaction. Hence, the following methodology is proposed to estimate the maximum design bending moment during transient phase to reach liquefaction.

In this methodology pile foundations are considered in two different conditions of pre liquefaction and post liquefaction phases (as the p-y curve is known for these two
Therefore, piles are designed as a beam and Winkler springs are employed to represent soil-pile interaction. Two types of springs are considered; non-liquefiable and liquefiable springs which are represented pre-liquefaction and post-liquefaction conditions respectively. The properties of these springs are obtained from p-y curves. The non-liquefiable p-y curve can be calculated by empirical equations suggested by codes of practice such as API, 2000. The liquefiable p-y curve can be obtained either from the soil element test (if there is a possibility to carry out) or mechanics based p-y curve proposed by Dash, 2010.

As building time period changes during liquefaction, the base shear force changes from pre liquefaction to post liquefaction. The lateral load to apply on the top of the pile foundations can be calculated from the response spectrum analysis. The response spectrum graph can be generated for any types of input motion and soil profile based on codes of practice such as Eurocode. Figure 8.8 schematically illustrates the response spectrum for two different conditions of pre and post liquefaction phases. As shown, the magnitude of spectral acceleration for pre-liquefaction phase is higher than post-liquefaction phase as this phase has a higher damping ratio than pre-liquefaction stage.

![Schematic response spectrum of pre and post liquefaction conditions.](image)

In order to calculate the amount of force applying on the top of the pile, two spectral accelerations (i.e. $S_{a,\text{pre-liq}}$ and $S_{a,\text{post-liq}}$) are chosen regarding to two particular amount of
time period; pre and post-liquefaction phase (i.e. $T_{\text{pre-\text{eq}}} \text{ and } T_{\text{post-\text{eq}}}$). The base shear force can be then calculated by multiplying the spectral acceleration by the dead load of the superstructure as given below;

$$H = V.S_a$$

where, $H$ is the base shear force, $V$ is the dead load of the building, and $S_a$ is the spectral acceleration taken from the response spectra graph. The base shear is calculated twice for pre-liquefaction and post-liquefaction phases based on the related spectral acceleration. As can be seen from Figure 8.9, the maximum bending moment related to pre and post-liquefaction base shear force are computed for each condition separately. As the bending moment amplifies in transient phase (i.e. the maximum bending moment occurs in transient phase to reach liquefaction), the maximum bending moment in this phase can be estimated by multiplying the pre liquefaction bending moment by the dynamic amplification factor which is dependent on the time taken to reach liquefaction (as discussed in Chapter 5).
CHAPTER – 8 DESIGN METHOD OF PILE FOUNDATION DURING SEISMIC LIQUEFACTION

Figure 8.9: Bending moment of pile foundations in pre and post liquefaction conditions obtaining from Winkler analysis.

The effect of axial load on the bending moment of pile foundations is schematically shown in Figure 8.10. As illustrated in Figure 8.10(a) pile displacement \( (y) \) can be normalised by the pile diameter \( (D) \) whereas pile bending moment \( (M) \) is normalised by the plastic bending moment \( (M_p) \) as shown in Figure 8.10(b). The x axis (based on types of Winkler analysis; displacement based or force based) can be normalised either by \( \frac{\Delta_{soil}}{D} \) or \( \frac{F}{F_{max}} \). As
shown in the figure once the axial load \( (P) \) exceeds the critical load \( (P_{cr}) \), the pile deflection and bending moment show larger values.

![Diagram of pile deflection and bending moment](image)

Figure 8.10: Effect of axial load on pile head deflection and bending moment.

As the above methodology can be summarised Table 8.3 shows the steps of the methodology as well as the requirements to calculate the parameters.
Table 8.3: Design methodology steps

<table>
<thead>
<tr>
<th>Step ID</th>
<th>Step description</th>
<th>Required parameters</th>
</tr>
</thead>
</table>
| 1       | Obtaining depth of liquefaction (can be obtained by different methods such as Idriss and Boulanger, (2008), Eurocode 8, and Japanese code). | • Earthquake moment magnitude \( (M_w) \)  
• Earthquake PGA \( (a_{\text{max}}) \)  
• Soil profile and properties |
| 2       | Calculating the pre-liquefaction time period and post-liquefaction time period. | • Building dimensions  
• Pile dimensions, depth of liquefaction or length of unsupported pile, and the total weight of the building |
| 3       | Obtaining elastic response spectrum graph (can be drawn by using Eurocode 8). | • Earthquake PGA \( (a_{\text{max}}) \)  
• Soil type and properties |
| 4       | Calculating the pre and post-liquefaction base shear force \( (H_1 \) and \( H_2 \) respectively). (see Figure 8.9). | • Dead load of the superstructure  
• Spectral acceleration related to the pre and post-liquefaction time period |
| 5       | Producing \( p-y \) curves for two different conditions: | For non-liquefiable soil:  
• Non-liquefiable soil (from empirical equations suggested by codes of practice such as API, 2000).  
• Liquefiable soil (either from the soil element test or the practical method suggested by Dash, 2010). |
|         | \( p-y \) curves | For liquefiable soil:  
• Soil properties \( (N_{\text{SPT}}, \phi, D) \)  
• Soil coefficients of C1, C2, C3  
• Initial modulus of subgrade reaction \( (k) \)  
\( \text{For liquefiable soil:} \)  
• Soil properties \( (\tau_{\text{max}}, G_1, G_2, \gamma, N_{\text{SPT}}, \phi) \) |
| 6       | Pseudo static analysis using Winkler approach to calculate the bending moments in pre and post-liquefaction conditions \( (M_{\text{pre-liq}} \) and \( M_{\text{post-liq}} \) respectively). The maximum value is chosen as \( M_{\text{max}} \) to use in step 9. | • Pile properties  
• \( p-y \) curves  
• Pre and post-liquefaction shear base force |
| 7       | Time taken to reach maximum or full liquefaction (can be obtained by any site response analysing). | • Soil profile  
• Earthquake time history |
| 8       | Estimating the dynamic amplification factor \( (\eta) \). | • Time taken to reach liquefaction (i.e. speed of liquefaction) |
| 9       | Estimating the maximum bending moment occurs in transient phase to reach liquefaction. \[ M_{\text{Design}} = \eta \times M_{\text{max}} \] | • Dynamic amplification factor \( (\eta) \) |
The next section presents an example which is explained this methodology step by step by considering a building located in a seismic liquefiable soil.

### 8.5 Example of the proposed methodology

A 5-storey building located in a seismic liquefiable area was investigated as an example to present the proposed methodology of pile foundation design. The height of the building was assumed to be 14.5 m which was supported by 38 pile foundations in two rows of 19 piles having 7.5 m distance between the piles. The total dead load of the building was 15656 kN. Piles length and diameter were 20 m and 0.4 m respectively. Figure 8.11 shows the building details.

![Building and pile foundation](image)

**Figure 8.11: Investigated building with building and pile foundation dimensions.**

The soil profile was consisted of several strata as shown in Figure 8.12. This building was subjected to Kobe earthquake (Japan, 1995) with magnitude and peak ground acceleration of 6.9 and 0.24g respectively. Pile foundation of this building was designed using the proposed methodology as explains step by step as follow.

#### Step-1: Depth of liquefaction

The depth of liquefaction can be obtained by using different methods such as Idriss and Boulanger, (2008) and Eurocode8 (2003). In this example Eurocode8 method was used to work out the depth of liquefaction. As illustrates in Figure 8.13 the soil profile consists of
different layers of fill, sand, sandy silt, and gravelly sand. Standard Penetration Test (SPT) has been carried out in order to identify the soil strength. The obtained N-value from in-situ SPT was plotted in Figure 8.12. Depth of liquefaction was defined by considering the factor of safety against liquefaction. Based on Eurocode 8, the required parameters are $M_w=6.9$ and $PGA=0.24g$ are considered to work out the depth of liquefaction. According to the calculation this soil profile was liquefied up to 16m.

![Figure 8.12: Soil profile details, in situ N value, and the calculated factor of safety against liquefaction.](image)

Step-2: Calculating the pre and post-liquefaction time period

*Pre-liquefaction time period:*

Bhattacharya and Goda (2013) suggested an equation to estimate the pre-liquefaction time period of building supported by pile foundation. The required parameters are:

- Height of the building ($H_B$) = 14.5 m
- Foundation width between the piles ($B$) = 7.5 m, therefore,

$$T_{pre-\text{eq}} = \frac{0.09H_B}{B^{0.5}} = \frac{0.09 \times 14.5}{7.5^{0.5}} = 0.48 \text{ sec}$$
Post-liquefaction time period:

Bhattacharya and Goda (2013) suggested an equation to estimate the post-liquefaction time period of building supported by pile foundation. The required parameters are:

- Number of the piles \( (N_p) = 38 \)
- Total weight of the building \( (W) = N_p \times P_{\text{static}} = 38 \times 412 = 15656 \text{kN} \)
- Lateral stiffness of each pile \( (EI) = 32.35 \text{ MNm}^2 \)
- Depth of liquefaction \( (D_l) = 16 \text{ m} \), therefore,

\[
T_{\text{post-liquefaction}} = 2\pi \sqrt{\frac{W}{gN_p \times 12EI}} = 2\pi \sqrt{\frac{15656}{9.81 \times 38 \times 12 \times 32.35 \times 1000}} = 4 \text{ sec}
\]

Step-3: Obtaining elastic response spectrum graph

The response spectrum graph can be plotted using Eurocode 8 guideline. Based on that the ground was assumed to be type D. Therefore, the required parameters are:

- Soil factor \( (S) = 1.35 \)
- The lower limit of the period of the constant spectral acceleration \( (T_l) = 0.2 \text{ sec} \)
- The upper limit of the period of the constant spectral acceleration \( (T_u) = 0.8 \text{ sec} \)
- The period value related to the beginning of the constant displacement response range of the spectrum \( (T_p) = 2 \text{ sec} \)
- Ground acceleration \( (a_g) = 0.24g \)
- Damping correction factor \( (\eta) = 1 \) for 5% viscous damping (pre-liquefaction) condition. For post liquefaction condition the viscous damping is assumed 20%. Therefore the damping correction can be calculated by the given equation;

\[
\eta = \sqrt{\frac{10}{5 + \xi}} = \sqrt{\frac{10}{5 + 20}} = 0.63
\]

Figure 8.13 plots the response spectral acceleration based on the required parameters. As shown the spectral accelerations of pre and post-liquefaction conditions \( (S_{a,\text{pre-liquefaction}} \) and \( S_{a,\text{post-liquefaction}} \) respectively) were chosen related to the pre and post-liquefaction time periods
(\(T_{\text{pre-liq}}\) and \(T_{\text{post-liq}}\) respectively). These spectral acceleration were used to calculate the base shear force as is mentioned in Step-4.

**Figure 8.13: Elastic response spectrum in pre and post liquefaction.**

**Step-4: Calculating the pre and post-liquefaction base shear force**

**Pre-liquefaction base shear force:**

The required parameters are:

- The spectra acceleration at pre-liquefaction phase = 0.81
- Dead load of the building acting per pile = 412 kN

\[ H_1 = S_a \times V = 0.81 \times 412 = 334kN \]
Post-liquefaction base shear force:

The required parameters are:

- The spectra acceleration at post-liquefaction phase = 0.05
- Dead load of the building acting per pile = 412 kN

\[ H_2 = S_v \times V = 0.05 \times 412 = 21kN \]

Step-5: Producing p-y curves for two different conditions

The non-liquefiable p-y curve was produced using API, 2000 guideline. The liquefiable p-y curve was produced using the proposed practical method suggested by Dash, (2010). These p-y curves were produced based on soil properties and modelled along the pile foundations to represent the soil-pile interaction. Figure 8.14 compares these two types of p-y curve.

Figure 8.14: Comparison of p-y curves of non-liquefiable soil obtained from (API, 2000) and liquefiable soil obtained based on mechanics based method (Dash, 2010).
Step-6: Pseudo static analysis using Winkler approach

Winkler approach was employed to carry out pseudo static analysis on pile foundations. This method is used in some practical software such as SAP2000 (CSI, 2011) and Alp (Oasys, 2013). The analyses were carried out twice for pre and post-liquefaction conditions in SAP2000 ver.15. The P-delta effect was chosen for the post-liquefaction condition in order to have the effect of the axial load in liquefiable soil. Pile was modelled as a beam and p-y curves were modelled to represent soil-pile interaction. In post liquefaction pile was analysed in two different conditions; i) considering p-y springs for liquefiable layer and ii) without considering p-y springs for liquefiable layer. The calculated base shear forces were applied at the top of the pile. Two different of pile head fixity i.e. free-headed and fixed-headed pile were considered in these analyses. Figures 8.15 and 8.16 show the bending moment profile along the pile for pre and post-liquefaction conditions respectively. From Figure 8.15 the bending moments in free-headed pile foundations were around 700kNm and 90kNm for pre and post-liquefaction with p-y springs for liquefiable layer respectively. However, the computed value of bending moment in post liquefaction phase without p-y springs was around 460kN.m. In the case of fixed-headed pile foundation, the bending moment were around 400kNm and 90kNm for pre and post-liquefaction with p-y springs for liquefiable layer respectively. The bending moment of pile in post liquefaction phase without considering p-y springs for liquefiable layer was computed around 250kN.m. Figure 8.17 shows the pile head deflection in free-headed pile foundation. As can be seen the deflection in pre liquefaction was obtained around 70mm. In post liquefaction state the deflection were obtained around 160mm and 1300mm in the conditions of with and without p-y springs for liquefiable layer.
Figure 8.15: Bending moment profile in pre and post liquefaction for free-headed pile.

Figure 8.16: Bending moment profile in pre and post liquefaction for fixed-headed pile.
Step-7: Time to reach maximum or fully liquefaction

Time taken to reach full or maximum liquefaction can be obtained by any site response analysis. Cyclic1D is one of the site response analysis program which can be used to carry out this analysis. The soil profile was modelled in the program and subjected to the Kobe earthquake time history and the Excess Pore Water Pressure Ratio (EPWPR) was calculated for different levels of the soil profile. Figure 8.18 plots the results from the analysis. As mentioned before the depth of liquefaction was 16 meter. Therefore, the average time taken to reach liquefaction for the three levels of 5m, 10m, and 15m was chosen and computed around 6 seconds as time taken to reach full liquefaction.
Step-8: Obtaining the dynamic amplification factors

In the absence of more research in this area the following is proposed to estimate the dynamic amplification factor. Based on the discussion on Chapter 5, the dynamic amplification factor can be estimated by considering speed of liquefaction (i.e. time taken to reach liquefaction). As the average time taken to reach liquefaction was computed around 6 seconds, it is suggested that the amplification factor to be assumed 2.5 ($\eta = 2.5$).
Step-9: Estimating the maximum bending moment occurs in transient phase to reach liquefaction

In order to estimate the design bending moment, the maximum computed bending moment was chosen from the results in step 6. This bending moment was multiplied by the suggested amplification factor in step 8. Therefore the design bending moment for the two conditions of free and fixed-headed pile are as follow;

For free-headed pile:

\[ M_{\text{Design}} = \eta \times M_{\text{max}} = 2.5 \times 700 = 1750 \text{kNm} \]

For fixed-headed pile:

\[ M_{\text{Design}} = \eta \times M_{\text{max}} = 2.5 \times 400 = 1000 \text{kNm} \]

8.6 Conclusion

The chapter proposed a new design method for pile supported structures in seismic liquefaction, which can be applied in any seismic area. As current methods of design of pile supported structure are based on the bending failure without consideration of dynamic effects, the method that has been proposed is based on understanding developed in this research. The method is based on the following key parameters: depth of liquefaction, time period of the structure at pre and post liquefaction phases, and time taken to reach full liquefaction. The base shear force acting on the pile head is calculated by using response spectrum analysis based on the time period of structure at pre and post-liquefaction phases. The Beam on Non-linear Foundation analysis (Winkler method) is carried out to analyse the pile foundation and the maximum bending moment obtained from this analysis is multiplied by the dynamic amplification factor (which is based on the time taken to reach liquefaction) to estimate the design moment. It was also found that the post-liquefaction p-y curve may introduce stiffness which can affect the amount of deflection and the bending moment results, which requires further investigations.
Chapter 9
Main findings and recommendation for further research

9.1 Introduction

The attempt of this chapter is to summarise the important findings observed through performing a series of laboratory experiments on understanding the critical factors and parameters involved in pile failure in liquefiable soils under earthquake events.

Collapse of pile supported structures is still observed after most major earthquakes around the world, especially in the seismic liquefiable areas. There are, however, many factors that need to be considered to better understand the actual causes of pile failure in liquefiable soils. These factors are mainly dependent on ground profile and earthquake characteristics. As liquefaction process happens during an earthquake event, the
surrounding soil becomes more in the liquid-like material state which can no longer support the pile foundations. As a result, pile foundation which loses its support from the surrounding soil, demonstrates a more flexible response and its time period increases from the initial value. The research conducted here presents, primarily, the results of pile foundations failure, based on time taken to reach liquefaction (i.e. speed of liquefaction) and the time period change during the liquefaction process. Specifically, a series of shake table tests have been carried out to collect critical data in order to better understand and present the pile response in the liquefiable soils under earthquake events. In addition, advanced soil element tests have also been conducted to better characterise the liquefiable soils. Moreover, shake table results were also back analysed to show the application of post-liquefaction stress-strain curve of the soils. The summaries of the tests results and the major conclusions of this research study are explained as follows:

9.2 Conclusions

As explained, this study consisted of eight chapters with this chapter being the summary of the conclusions from the previous chapters. The major conclusions presented here are from Chapter 5 (shake table test results), Chapter 7 (advanced soil element test results), and Chapter 8 (the design of the pile foundations in seismic liquefaction conditions). The following section focuses on the main obtained conclusions.

9.2.1 Pile response during the transient phase before reaching full liquefaction

When an earthquake happens in loose to medium saturated sand conditions, the soil materials tend to be more compacted and as a consequence, the excess pore water pressure increases. This phenomenon may cause the soil materials to gradually or dramatically enter a more liquefied state and eventually lose its strength and stiffness. Therefore, the embedded pile foundation at this state loses its support from the surrounding soils and consequently shows a more flexible response. Thus, as pile becomes more flexible its time period increases from time period at pre-liquefaction \( T_{\text{pre-liq}} \) to time period at post-liquefaction \( T_{\text{post-liq}} \). This flexibility of pile foundation during earthquake may most likely affect by the time taken to reach liquefaction.
9.2.2 Time taken to reach full liquefaction

Pile foundation response and also its failure can be affected by time taken to reach liquefaction which is also considered as “speed of liquefaction”. This parameter is primarily depend on two other factors, the ground profile condition and the earthquake time history. This parameter, however, can be obtained either through the site response information or by performing finite element analysis. For example, in this study, the real site (Showa Bridge site) was analysed in Cyclic 1D (site response analysis program) and was subjected to some various earthquake time histories which were caused from different fault movement scenarios. The results (for a particular ground profile) showed, the time taken to reach liquefaction (i.e. speed of liquefaction) and also the maximum excess pore water pressure ratio obtained for different levels of ground can be affected by the earthquake magnitude and Peak Ground Acceleration.

9.2.3 Transience of bending moment during seismic liquefaction

As discussed earlier, pile foundation becomes more flexible during the liquefaction phenomenon. The response of pile foundations can be explained by presenting the bending moment along the pile foundation. As implied in Chapter 5, the bending moment of piles can be varied during liquefaction process. At the beginning of the input motion (pre liquefaction phase), the pile models bent with small amplitudes due to the inertia force from the superstructure. In the higher amplitudes of the input motion, as soil started to liquefy and stopped supporting the pile models, the bending moment of the pile reached the maximum value in the transient phase to reach liquefaction (i.e. \( r_u < 1 \)) and finally, at the full liquefaction state, the amplitude of the bending moment reduced. As bending moment was amplified and reached its maximum value during transient phase, a dynamic amplification factor was introduced which might be useful in designing pile foundations during the seismic liquefaction.

9.2.4 Dynamic amplification factor

From this study, two dynamic amplification factors (\( \eta_1 \) and \( \eta_2 \)) were derived; \( \eta_1 \) was defined by dividing the maximum bending moment measured in transient phase over the bending moment in pre liquefaction (\( \eta_1 = \frac{M_{\text{max,transient}}}{M_{\text{pre-\text{-}lq}}} \)) and \( \eta_2 \) was identified by dividing the maximum bending moment measured in transient phase over the bending moment in
post liquefaction \( \eta_2 = \frac{M_{\text{max transient}}}{M_{\text{post-liquefied}}} \). As were shown, these factors can be affected by parameters like speed of liquefaction and the time period elongation ratio. From the observation, the dynamic amplification factors reduced as the speed of liquefaction increased. However, by increasing the time period elongation, this factor also increased.

9.2.5 Undrained response of sandy soils

As known, the speed of earthquake is more than the speed of dissipation of excess pore water which is generated by the earthquake event and, therefore, the response of saturated soil during earthquake could considered as undrained response. Hence, the undrained behaviour of four types of liquefiable soil was studied in this research; Redhill-110 sand and Japanese silica sand No. 8 which were two commercial sands for research purposes and Assam sand and Ganga sand which were two real sands from India. These sand materials were subjected to undrained cyclic and monotonic loading (i.e. multi-stage test) in order to characterise their response. As observed, the undrained cyclic and monotonic response of sands were strongly dependant on some other factors such as the soil relative density \( (D_r) \), the effective confining stress \( (\sigma'_c) \), and the deviator stress \( (q) \).

9.2.6 Post-liquefaction response of sandy soils and its implication

The response of liquefiable sands in post liquefaction state was also investigated in this research by conducting a series of multi-stage tests on four different sand materials. Based on the results obtained, three parameters were involved in post liquefaction behaviour; initial shear modulus \( (G_i) \), post-dilation shear strain \( (\gamma_{\text{post-dilation}}) \), and the critical state of shear modulus \( (G_2) \). These parameters showed to be strongly dependant on relative density of the sand; by showing that as the relative density increased, \( G_i \) and \( G_2 \) increased, however, the value of \( \gamma_{\text{post-dilation}} \) decreased. In addition, based on the results obtained, the shear modulus decreased from the maximum shear modulus \( (G_0) \) to \( G_i \) to about 1/10000 during the undrained cyclic loading. During the post-liquefaction monotonic loading, however, the liquefied sand gradually recovered its stiffness \( (G_2) \) and was about 1/10 of the \( (G_0) \) value.

Furthermore, the post-liquefaction stress-strain curve of the soil can also be used in evaluating the lateral spreading of the liquefied soil, the liquefaction-induced settlements of
structures and other liquefaction-related ground deformations. This study showed that this curve can also be converted to the p-y curves and useful for the analyses of soil-structure interactions. Also, the back analysis of the shake table which was carried out by using proposed p-y curves, showed a good agreement with the measured data.

### 9.2.7 Proposed method for the design of pile foundations in the seismic liquefaction

Based on the results and the understanding developed, a new design of pile foundation for the seismic liquefaction state was proposed. In this research, the dynamic effects was considered in this proposed method as the existing design methods of pile foundations in liquefiable soils are based on bending mechanism failure without considering the dynamics of the structures. This method was based on some parameters such as depth of liquefaction, time period of the structure and time taken to reach liquefaction. As Winkler method was adopted to analyse pile foundation in this method, pile foundation was modelled as a beam and p-y springs were applied to represent pile-soil interaction. The base shear force acting on the top of the pile was computed by using response spectrum analysis based on the time period of the structure at pre and post liquefaction phases. The maximum bending moment obtained from the analyses was chosen and multiplied by the dynamic amplification factor (which was dependent on time taken to reach liquefaction) to estimate the design bending moment. In addition, an example was also presented in order to show the step-by-step proposed method.

### 9.3 Suggestions for further research

The focus of this research was to present the transience of bending moment along the pile foundation during liquefaction process. Specifically, the failure of pile foundation was examined, based on considering the dynamics effects involved in the process. Although many research studies have been carried out on this kind of research topic, however, there still remains many unknown issues in this topic that need to be better understood. Following are few items that can be suggested for further investigations:

- The effect of different dynamic parameters on dynamic amplification factor.
- The effect of significant and uniform durations of earthquake on pile response in the transient dynamics state.
• The effect of significant and uniform durations of earthquake on the time reaching the liquefaction state.
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Appendix A: Site response analysis results

In this section the results of site response analysis carried out using Cyclic 1D are presented. The ground profile (which has been already explained in Chapter 1) was subjected to different earthquakes to monitor the effect of different earthquakes on time to reach liquefaction.

Figure A.1: Site response analysis of Kern county earthquake (1952): (a) time history of real earthquake; (b) excess pore water pressure at 20m depth; (c) excess pore water pressure at 15m depth; (d) excess pore water pressure at 10m depth; and (e) excess pore water pressure at 5m depth.
Figure A.2: Site response analysis of San Fernando earthquake (1971): (a) time history of real earthquake; (b) excess pore water pressure at 20m depth; (c) excess pore water pressure at 15m depth; (d) excess pore water pressure at 10m depth; and (e) excess pore water pressure at 5m depth.
Figure A.3: Site response analysis of Loma Prieta earthquake (1989): (a) time history of real earthquake; (b) excess pore water pressure at 20m depth; (c) excess pore water pressure at 15m depth; (d) excess pore water pressure at 10m depth; and (e) excess pore water pressure at 5m depth.
Figure A.4: Site response analysis of Christchurch earthquake (2011): (a) time history of real earthquake; (b) excess pore water pressure at 20m depth; (c) excess pore water pressure at 15m depth; (d) excess pore water pressure at 10m depth; and (e) excess pore water pressure at 5m depth.
Figure A.5: Site response analysis of Borah Peak earthquake (1983): (a) time history of real earthquake; (b) excess pore water pressure at 20m depth; (c) excess pore water pressure at 15m depth; (d) excess pore water pressure at 10m depth; and (e) excess pore water pressure at 5m depth.
Figure A.6: Site response analysis of Kozani earthquake (1995): (a) time history of real earthquake; (b) excess pore water pressure at 20m depth; (c) excess pore water pressure at 15m depth; (d) excess pore water pressure at 10m depth; and (e) excess pore water pressure at 5m depth.
Figure A.7: Site response analysis of Dinar earthquake (1995): (a) time history of real earthquake; (b) excess pore water pressure at 20m depth; (c) excess pore water pressure at 15m depth; (d) excess pore water pressure at 10m depth; and (e) excess pore water pressure at 5m depth.
Figure A.8: Site response analysis of Umbria marche earthquake (1997): (a) time history of real earthquake; (b) excess pore water pressure at 20m depth; (c) excess pore water pressure at 15m depth; (d) excess pore water pressure at 10m depth; and (e) excess pore water pressure at 5m depth.
Figure A.9: Site response analysis of Southern California earthquake (1952): (a) time history of real earthquake; (b) excess pore water pressure at 20m depth; (c) excess pore water pressure at 15m depth; (d) excess pore water pressure at 10m depth; and (e) excess pore water pressure at 5m depth.
Appendix B:

B.1: Ground motion parameters

Some of the important earthquake parameters are defined here.

**Peak Ground Acceleration (PGA):** This is the maximum ground acceleration recorded at a given location during an earthquake. Figure 2.2a illustrates time history of Kobe earthquake, (Japan, 1995) at a location with the PGA of 0.34g.

**Earthquake magnitude:** This is used to measure the released energy from earthquake. There are several methods to measure earthquake magnitude and are listed below:

1) Local magnitude ($M_L$) (Richter, 1935)
2) Japanese Meteorological Agency (JMA) magnitude ($M_{JMA}$)
3) Kawasumi magnitude ($M_K$) (Kawasumi, 1951)
4) Surface wave magnitude ($M_s$) (Gutenberg, 1945a)
5) Body wave magnitude, ($m_b$) (Gutenberg, 1945b and 1945c)
6) Earthquake energy $E$ (erg = $10^{-7}$ J)
7) Moment magnitude ($M_w$)

Among these methods of magnitude measurement, moment magnitude ($M_w$) and surface wave magnitude ($M_s$) are normally used in practical analysis. For instance, in order to evaluate depth of liquefaction, $M_s$ is used based on the method proposed in Eurocode 8 (2003).

**Predominant period ($T_p$):** “The predominant period is the period at which the maximum spectral acceleration occurs in an acceleration response spectrum calculated at 5% damping” explains by Seismosignal help system (2013).

**Uniform duration:** Seismosignal help system defines as “The total time during which the acceleration is larger than a given threshold value (default is 5% of PGA)”. Figure B.1b demonstrates this duration for Kobe earthquake.

**Arias Intensity (AI):** shows the strength of a ground motion given by Equation B.1 following (Arias, 1970).
\[ AI = \frac{\pi}{2g} \int_0^t [a(t)]^2 dt \]  

(B.1)

where, \( a(t) \) is the acceleration time history, \( t_r \) is the total duration of the acceleration, and \( g \) is the acceleration due to the gravity.

**Significant duration:** Seismosignal help system defines as “The interval of time over which a proportion (percentage) of the total Arias Intensity is accumulated (default is the interval between the 5% and 95% thresholds)”. Figure B.1c illustrates this duration for Kobe earthquake.

**Fourier amplitude:** a parameter that shows the distribution of ground motion amplitude with respect to time period or frequency.

**Power spectrum density:** this parameter is used to understand the strength distribution of the motion in the frequency domain.

The above parameters can be obtained by standard ground motions analysis programs such as “SeismoSignal”, (Seismosignal manual, 2013).

![Kobe earthquake parameter](image)

Figure B.1: Kobe earthquake parameter: (a) time history; (b) uniform duration; and (c) significant duration.
B.2: Significant and uniform duration of the applied input motions in shake table test

Figure B.2: Friuli earthquake parameter (a) time history, (b) uniform duration, and (c) significant duration.
Figure B.3: Irpinia earthquake parameter (a) time history, (b) uniform duration, and (c) significant duration.

Irpinia earthquake, (1980)

Figure B.4 Northridge earthquake parameter (a) time history, (b) uniform duration, and (c) significant duration.

Northridge earthquake, (1994)
Figure B.5: L’Aquila earthquake parameter (a) time history, (b) uniform duration, and (c) significant duration.

Christchurch earthquake, (2011)

Figure B.6: Christchurch earthquake parameter (a) time history, (b) uniform duration, and (c) significant duration.
Figure B.7: Scaled (by factor of 0.7) Christchurch earthquake parameter (a) time history, (b) uniform duration, and (c) significant duration.

Christchurch earthquake (2011)- scaled by 0.7

5% of Arias Intensity

95% of Arias Intensity

Significant duration~7 sec

$A_o = 0.05 \times 0.92g = 0.05g$

Uniform duration=$\Sigma t$ when $a > (0.05)^2$

Uniform duration~8 sec

Figure B.8: Scaled (by factor of 0.5) Christchurch earthquake parameter (a) time history, (b) uniform duration, and (c) significant duration.

Christchurch earthquake (2011)- scaled by 0.5

5% of Arias Intensity

95% of Arias Intensity

Significant duration~6.5 sec

$A_o = 0.05 \times 0.64g = 0.03g$

Uniform duration=$\Sigma t$ when $a > (0.03)^2$

Uniform duration~9 sec

Figure B.8: Scaled (by factor of 0.5) Christchurch earthquake parameter (a) time history, (b) uniform duration, and (c) significant duration.
Figure B.9: Sine-dwell motion parameter (a) time history, (b) uniform duration, and (c) significant duration.
Appendix C: Shake table test results

C.1: Bending moment of pile models

The results of the shake table test are presented in this appendix. The bending moment of pile models due to the applied input motion were plotted along the pile models.

**GP22 (Test MR-3)**

![Figure C.1: Measured bending moment along the GP22 (Test MR-3).](image)

For the instrumental layout please refer to Chapter 4, Figure 4.10b.)
Figure C.2: Measured bending moment along the GP12 (Test MR-5).  
(For the instrumental layout please refer to Chapter 4, Figure 4.10c).
SP22 (Test MR-7 to MR-12)

Figure C.3: Measured bending moment along the SP22 (Test MR-7).
(For the instrumental layout please refer to Chapter 4, Figure 4.10c).
Figure C.4: Measured bending moment along the SP22 (Test MR-8).
(For the instrumental layout please refer to Chapter 4, Figure 4.10c).
Figure C.5: Measured bending moment along the SP22 (Test MR-9). (For the instrumental layout please refer to Chapter 4, Figure 4.10c).
Figure C.6: Measured bending moment along the SP22 (Test MR-10).
(For the instrumental layout please refer to Chapter 4, Figure 4.10c).
Figure C.7: Measured bending moment along the SP22 (Test MR-11).
(For the instrumental layout please refer to Chapter 4, Figure 4.10c).
Figure C.8: Measured bending moment along the SP22 (Test MR-12).
(For the instrumental layout please refer to Chapter 4, Figure 4.10c).
C.2: Dynamic bending amplification factors

Table C.1: Dynamic amplification factors for all the structures (Test MR-1)

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<th>Test ID</th>
<th>Structure</th>
<th>Strain Gauge level (m)</th>
<th>Bending moment before liquefaction (Nm) ((M_1))</th>
<th>Bending moment at full liquefaction (Nm) ((M_2))</th>
<th>Maximum bending moment at transient phase (Nm) ((M_3))</th>
<th>Experimental dynamic amplification factor ((\eta_1)) (\frac{M_3}{M_1})</th>
<th>Experimental dynamic amplification factor ((\eta_2)) (\frac{M_3}{M_2})</th>
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Table C.2: Dynamic amplification factors for SP12 (Test MR-2)

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<th>Maximum bending moment at transient phase (Nm) ((M_3))</th>
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### Table C.4: Dynamic amplification factors for GP22 (Test MR-4)

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Table C.6: Dynamic amplification factors for GP12 (Test MR-6)

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<th>Maximum bending moment at transient phase (Nm) ($M_3$)</th>
<th>Experimental dynamic amplification factor ($\frac{M_3}{M_1}$)</th>
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### Table C.11: Dynamic amplification factors for SP22 (Test MR-11)

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Table C.12: Dynamic amplification factors for SP22 (Test MR-12)

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<th>Maximum bending moment at transient phase (Nm) ( (M_3) )</th>
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Table C.13: Dynamic amplification factors for SP22 (Test MR-13)

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<th>Maximum bending moment at transient phase (Nm) ( (M_3) )</th>
<th>Experimental dynamic amplification factor ( (\eta_1) ) ( \frac{M_3}{M_1} )</th>
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C.3: Soil response

Figures C.9 to C.20 show the soil response for all the tests. These plots have four subplots of input motion time history, excess pore water pressure ratio at five levels of soil, and soil acceleration time history in two different levels of -0.6 and -1m below the soil surface. As can be seen soil shows a higher acceleration in a deeper depth as soil is denser.

Figure C.9: Measured soil response at -0.6 and -1m below soil surface (Test MR-2). (For the instrumental layout please refer to Chapter 4, Figure 4.10b).
Figure C.10: Measured soil response at -0.6 and -1m below soil surface (Test MR-3). (For the instrumental layout please refer to Chapter 4, Figure 4.10b).
Figure C.11: Measured soil response at -0.6 and -1m below soil surface (Test MR-4). (For the instrumental layout please refer to Chapter 4, Figure 4.10b).
Figure C.12: Measured soil response at -0.6 and -1m below soil surface (Test MR-5). (For the instrumental layout please refer to Chapter 4, Figure 4.10c).
Figure C.13: Measured soil response at -0.6 and -1m below soil surface (Test MR-6). (For the instrumental layout please refer to Chapter 4, Figure 4.10c).
Figure C.14: Measured soil response at -0.6 and -1m below soil surface (Test MR-7). (For the instrumental layout please refer to Chapter 4, Figure 4.10c).
Figure C.15: Measured soil response at -0.6 and -1m below soil surface (Test MR-8). (For the instrumental layout please refer to Chapter 4, Figure 4.10c).
Figure C.16: Measured soil response at -0.6 and -1m below soil surface (Test MR-9). (For the instrumental layout please refer to Chapter 4, Figure 4.10c).
Figure C.17: Measured soil response at -0.6 and -1m below soil surface (Test MR-10). (For the instrumental layout please refer to Chapter 4, Figure 4.10c).
Figure C.18: Measured soil response at -0.6 and -1m below soil surface (Test MR-11). (For the instrumental layout please refer to Chapter 4, Figure 4.10c).
Figure C.19: Measured soil response at -0.6 and -1m below soil surface (Test MR-12). (For the instrumental layout please refer to Chapter 4, Figure 4.10c).
Figure C.20: Measured soil response at -0.6 and -1m below soil surface (Test MR-13). (For the instrumental layout please refer to Chapter 4, Figure 4.10c).