EVOLUTION OF GEOTECHNICAL EARTHQUAKE ENGINEERING—FROM SPT TESTING TO PERFORMANCE BASED DESIGN

SP Gopal Madabhushi
Reader in Geotechnical Engineering, University of Cambridge, CB2 1PZ, United Kingdom. E-mail: mspg1@cam.ac.uk

ABSTRACT: The role of a geotechnical engineer has been evolving steadily, particularly in the area of geotechnical earthquake engineering. Increasingly the geotechnical engineer is required to design in difficult soil conditions that are stratified and/or sloping and may be susceptible to liquefaction. Vulnerability of soil layers to liquefaction may be established based on SPT or CPT testing but the geotechnical engineer is often required to predict the foundation and ground deformations that are likely in such soil strata. These are essential components to any Performance Based Design. This paper aims to present the modern tools such as dynamic centrifuge modelling that are available to a geotechnical engineer while designing complex foundation systems. It will focus on how the emphasis of the geotechnical engineer is shifting from characterising the soil strata and determining factors of safety against failure to a more exciting and complex task of establishing ground and foundation deformations. These points will be illustrated throughout by using design of pile foundations in liquefiable soils as an example.

1. INTRODUCTION

Traditionally the role of a geotechnical engineer involved in classifying the soils in the field and characterising the engineering properties of various soil strata encountered. These properties are then used in appropriate foundation design calculations to establish the factor of safety against a perceived mode of failure.

In earthquake geotechnical engineering the liquefaction potential of a given site is first established. The maximum credible earthquake that is possible at the site may be obtained from the seismic zonation maps for the region. The peak ground acceleration ($a_{\text{max}}$) that can occur may be determined for the maximum credible earthquake by using an appropriate attenuation relationship, for example, Ambraseys et al. (2005). The cyclic shear stress ($\tau_c$) generated by this earthquake can be calculated following Seed and Idriss (1971) as,

$$\tau_c = 0.65 \frac{a_{\text{max}}}{g} \sigma_0 r_d$$

where $\sigma_0$ is the total stress and $r_d$ is a stress reduction factor.

Field test data from either SPT or more preferably CPT tests can be used to establish the liquefaction potential of the site. The corrected SPT values ($N_{160}$) can be used, for example. In Figure 1 the liquefaction lines that demarcate ‘liquefaction’ and ‘no liquefaction’ are shown for three types of sands, following Eurocode 8 Part 5 (2004). It is preferable to obtain CPT data compared to SPT testing. If CPT data is available for the given site then, similar relationships can be found between CPT data and the liquefaction potential, for example as proposed by Robertson & Wride (1998). While progress has been made in the area of determining whether a given site is liquefiable there are several uncertainties associated with soil liquefaction. For example, if the cyclic shear stress value from maximum credible earthquake and the corrected SPT value for a layer give points very close to liquefaction potential lines, then it is not clear whether liquefaction should be expected or not. For example if the Cyclic Stress Ratio (CSR) is 0.2 and the $N_{160}$ is 20 for clean sand site, such a point will lie close to but below the liquefaction potential line. However, good engineering judgement requires that significant liquefaction be expected at such a site. So even when the $N_{160}$ data suggests that the site will

![Fig. 1: Liquefaction Potential Charts, Replotted Following EC8: Part 5 (2004), Madabhushi et al. (2009)](image-url)
fall into ‘no liquefaction’ category, significant excess pore pressures can be generated during sufficiently strong earthquake events. In such events, the structures founded on the site can suffer excessive settlement and/or rotations brought about by the lowering of the effective stress and a consequent degradation in soil stiffness.

Another effect of excess pore pressure generation and subsequent degradation of soil stiffness may be that the overall natural frequency of the soil-structure system can reduce bringing it closer to resonance. Such a lowering of natural frequency was documented for the case of tower structures on liquefiable soils by Madabhushi & Schofield (1993).

It is both important and interesting to understand the behaviour of structures on liquefiable soils. With increased pressure on land, the geotechnical engineers have to look at designing foundations of structures on sites which may be susceptible to liquefaction. Also geotechnical engineers may have to design retrofitting measures to existing historic structures that may be located on liquefiable soils as is the case in southern European countries like Italy and Greece. A large EU funded project called NEMISREF that has concluded recently, was aimed at developing novel liquefaction resistance measures for existing foundations. (for further details http://www.soletanche-bachy.com/nemisref and Mitrani & Madabhushi (2008).

In order to investigate earthquake geotechnical engineering problems, it is necessary to conduct dynamic centrifuge tests. These tests can be conducted on small scale physical models in the enhanced gravity field of the centrifuge in which earthquake loading can be simulated. The technique is now well established and produces very valuable insights into failure mechanisms of foundations without the need to wait for large earthquake events to happen. Similarly non-linear FE analyses can be carried out to investigate these problems, but it is essential that such analyses include realistic soil models and incorporate the soil plasticity accurately.

In this paper the emphasis will be on dynamic centrifuge modelling. After a brief introduction to the technique itself, the paper will present the case of pile behaviour in liquefiable soils and how centrifuge modelling can help clarify the possible failure mechanisms.

2. DYNAMIC CENTRIFUGE MODELLING

Physical modelling in earthquake engineering with reduced scale experiments in the centrifuge is now widely considered as ‘the’ established experimental technique of obtaining data in controlled conditions to help engineers and researchers to understand the mechanisms involved in the response of soil-structure systems to seismic loading. This experimental approach recreates the stress state in soils which is a fundamental requirement to observe realistic soil behaviour. In Figure 2, a view of the 10 m diameter Turner beam centrifuge at Cambridge is presented.
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<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum g-level of operation</td>
<td>100 g</td>
</tr>
<tr>
<td>Dimension of the soil models</td>
<td>56 m (L) × 25 m (B) × 22 m (H)</td>
</tr>
<tr>
<td></td>
<td>80 m (L) × 25 m (B) × 40 m (H)</td>
</tr>
<tr>
<td>Earthquake strength of choice</td>
<td>Up to 0.4 g of bed rock acceleration</td>
</tr>
<tr>
<td>Earthquake duration of choice</td>
<td>From 0 s to 150 s</td>
</tr>
<tr>
<td>Earthquake frequency of choice</td>
<td>From 0.5 Hz to 5 Hz</td>
</tr>
</tbody>
</table>

Note: All parameters above are in prototype scale.

The SAM earthquake actuator is a mechanical device which stores the large amount of energy required for the model earthquake event in a set of flywheels. At the desired moment this energy is transferred to the soil model via a reciprocating rod and a fast acting clutch. When the clutch is closed through a high pressure system to start the earthquake, the clutch grabs the reciprocating rod and shakes with an amplitude of ± 2.5 mm. This is transferred to the soil model via a bell crank mechanism. The levering distance can be adjusted to vary the strength of the earthquake. The duration of the earthquake can be changed by determining the duration for which the clutch stays on. Earthquakes at different frequency tone bursts can be obtained by selecting the angular frequency of the flywheels.

With the capabilities of the SAM actuator and the geotechnical centrifuge facilities described above, it is possible to investigate a wide range of earthquake geotechnical engineering problems including soil liquefaction. In this paper, the particular example of pile behaviour in liquefiable soils will be considered. It will be demonstrated how dynamic centrifuge modelling can help understand the failure mechanisms involved. Further it will be shown how the thinking about pile behaviour in liquefiable soils has evolved as more information is deciphered from the valuable centrifuge test data, culminating in development of simplified procedures for estimating settlements of pile groups.

3. SINGLE PILES IN LIQUEFIABLE SOILS

Piles are long, slender members that carry large axial loads as they transfer super structure loads into competent soil strata. In normal soils the slenderness is not an issue, as lateral soil pressures support the pile in the radial directions. However in soft soils such as marine clays, piles can suffer buckling failure. This problem of pile buckling in soft clays has been investigated earlier by many researchers, for example, Siva Reddy & Valsangkar (1970).

In sands the problem of pile buckling does not arise as large lateral pressures that offer lateral support to the pile can be easily generated. However when the sandy soils are loose and saturated, they can suffer liquefaction. Under those circumstances piles can lose all the lateral support and can suffer buckling failure as illustrated in Figure 5 by forming a plastic hinge close to the base of the pile. Madabhushi et al. (2009) describe several possible failure mechanisms for single piles. It must also be pointed out that the piles need to be ‘rock socketed’ at the base so that no settlement of the pile itself is possible. This type of base condition can occur when piles are transferring the load from the ground surface onto the bedrock.

This mechanism of failure for single piles was evaluated using dynamic centrifuge modelling by Bhattacharya et al. (2004). Tubular model piles made from aluminium alloy were placed in loose, saturated sand. The pile tips were fixed to base of the model container allowing no displacement or rotation. Axial load on these single piles was modelled using brass weights. These loads were calculated using Euler’s critical load formula for slender columns given by,

\[ P_{cr} = \frac{\pi^2 EI}{(\beta L)^2} \]  

where \( EI \) is the flexural rigidity of the pile, \( L \) is the length of the pile and \( \beta \) is a effective length factor that depends on the end fixity conditions. For the case of pile that is fixed at the base and free at the top, \( \beta = 2 \). For a pile group, where both ends of the pile may be considered as fixed, \( \beta = 0.5 \).

The centrifuge model was tested at 50 g’s and subjected to a base acceleration of 0.2 g at the bedrock level. This caused the loose sand to liquefy fully which was confirmed by excess pore pressure measurements as shown in Figure 6. The piles have suffered buckling failure as seen in Figure 7. After the test the model piles were extracted from the soil model to examine the location of plastic hinge formation. As seen in Figure 8, the plastic hinging occurred at approximately ¼ of the length below the pile head.
Based on these series of tests, it can be concluded that pile buckling occurs in liquefied soils for the case of single piles carrying axial loads close to the Euler critical loads. Any imperfections in the pile will of course bring down the Euler critical loads significantly. However single piles are very rarely employed in geotechnical design. One such case is the Showa Bridge foundations in which the bridge piers were extended below the water level and supported on rows of single piles. Based on the evidence of the centrifuge test data, Bhattacharya et al. (2005) argued that the failure of the Showa bridge during the 1964 Niigata earthquake could be attributed to buckling of the piles.

Interesting as the above results and their implications are, a few queries arise. Firstly, compared to Figure 5 in which the plastic hinging in the pile occurs close to the base of the pile, the location of the plastic hinge in the centrifuge test occurs quite a way up as seen in Figure 8. As plastic hinges form when the maximum bending moments exceed the plastic moment capacity of the pile section, clearly the liquefied soil below this level must be offering some support to the pile. This may be due to the monotonic shearing of the liquefied soil demanded by the deforming pile causes dilation and therefore a temporary reduction of the excess pore pressure. This can cause the liquefied soil to gain some strength temporarily and offer resistance to pile buckling. As a result the pile cannot buckle at the base but can only do so higher up, where the soil dilation is compromised due to inflow of water from surrounding soil. Secondly, would pile buckling be an issue for pile groups as the effective length of a pile in a group can be significantly smaller due to the end fixity conditions (fixed at base and top and therefore $\beta = 0.5$ in Eq. 2). This aspect is considered next.

### 4. PILE GROUPS IN LIQUEFIED SOILS

Piles are most often used in groups with a pile cap connecting all the pile heads. As a result there is considerable fixity of piles heads that provides resistance to rotation. In addition the pile heads are all forced to translate together. Madabhushi et al. (2009) describe several possible failure mechanisms for pile groups by forming plastic hinges. Typical examples for liquefiable level ground are shown in Figure 9 with four-hinge and three-hinge mechanisms.

In order to investigate the failure mechanisms for pile groups, a series of dynamic centrifuge tests were undertaken as described by Knappett and Madabhushi (2009a). As with the single piles described in Section 3, $2 \times 2$ model pile groups made from aluminium alloy were tested in loose saturated sands. A typical cross-section of the centrifuge models is shown in Figure 10. It must be noted that the brass weights on the pile caps are restrained in the direction of earthquake shaking to remove the inertial effects i.e. the pile groups can only buckle in the transverse direction.

In Figure 11 the excess pore pressures in the free-field are presented. As before, the dashed lines indicate full liquefaction levels at the corresponding depths. In Figure 11 the excess pore pressures reach these horizontal dashed lines confirming that the saturated sand bed has completely
liquefied during earthquake shaking. The Pore Pressure Transducers (PPTs) are also able to pick up the pressure pulse when the pile group buckles and the pile head load collapses onto the soil surface. This is seen as a sharp pulse at about 210 seconds in Figure 11. It is also interesting to note that the pile group has buckled at about 200 seconds, well after the end of the shaking i.e. 60 seconds. By this time all the inertial effects of shaking are finished and the pile group buckling can only be attributed to the softening of the soil due to excess pore pressure and soil liquefaction.

The excess pore pressure traces close to pile group G4 are presented in Figure 12. In this figure again the generation of excess pore pressures to full liquefaction level is clearly visible. However during shaking cyclic variations of excess pore pressures are seen in Figure 12. This may be attributed to the shear stresses induced during the shaking when the liquefying soil tries to shear past the piles. These shear stresses imposed on liquefied soil cause it to dilate, which is manifested as a drop in excess pore pressure and a temporary stiffening of the soil. PPT 10952 shows a progressive dilation on top of the cyclic variations as the pile group suffers increased deformations until the end of the earthquake at 60 seconds. After this the excess pore pressures recover as the pore fluid migrates from the free-field into the zone next to the pile group. This softens the soil in the region next to the pile group and this softening continues until 200 seconds when the pile group finally buckles. Again the two PPT’s are able to pick up the impulse from the pile head load impacts onto the soil surface.

The lateral displacement of the pile group G4 is measured using long stroke LVDT’s. The pile head displacement (in the transverse direction) with time is presented in Figure 13. In this figure the evolution of lateral displacement of the pile group is clearly seen. However, the LVDT runs out of range just before the end of the earthquake shaking at about 58 seconds.

A view of the centrifuge model in plan is shown in Figure 14. In this figure it can be seen that the pile group G4 has fully buckled and collapsed onto the side of the model container. The other pile group G1, carrying smaller axial load has also suffered significant transverse displacements. The deformed shape of the pile group G4 that has been
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extracted after the centrifuge test is also shown as an inset in Figure 14. As in the case of the single pile, the pile group also suffered buckling failure due to liquefaction of the soil supporting the piles. However, the location of the plastic hinges is somewhat different from the anticipated positions for plastic hinges shown in Figure 9a that are based on sway-frame mechanisms. The location of the plastic hinges in the centrifuge model are much higher suggesting that the liquefied soil is able to offer significant resistance to pile group buckling. Due to this resistance, the plastic hinges form much higher compared to the fully-constrained pile tips. Knappett & Madabhushi (2009b) have analysed this problem using the finite element code ABAQUS, that can capture large strain deformations in the post-buckled phase using Ritz’s algorithm. In addition they used non-linear p-y soil springs to model the soil surrounding the pile. These FE analyses were able to capture the location of the plastic hinges in the pile group to match those seen in Figure 14.

5. SETTLEMENT OF PILE GROUPS

In Sections 3 and 4 above, the role of dynamic centrifuge modelling in investigating the novel failure mechanism for piles and pile groups in liquefied soils was presented. Clearly this modelling technique can be used to verify the perceived failure mechanisms. In both cases it has helped in understanding the role of liquefied soil in modifying the simple Eulerian buckling first anticipated and the importance of soil dilation in the post liquefied state. It must be noted that this failure mechanism can occur when the pile tips are rock-socketed into bedrock.

At many sites the bedrock may be quite deep and the liquefiable soil strata may not extend all the way to the bedrock. In such cases, the piles are usually driven through the loose, liquefiable sand layers into dense sand layers that lie below (or stiff clay layers) and are not considered to be at risk from a liquefaction point of view based on say SPT numbers and expected cyclic stress ratios as shown in Figure 1. This more common case of pile foundations was investigated using dynamic centrifuge modelling as described by Knappett & Madabhushi (2009c).

A series of centrifuge tests were carried out to investigate the settlement of pile groups in soil strata where a loose, saturated sand layer overlies a dense sand layer. Bedrock is considered to be below the dense sand layer. The piles are driven through the loose, sand layer into the dense sand layer. A pile length of about ten pile diameters (≈10 D,) is embedded into the dense layer to provide sufficient fixity length for the pile in dense sand. A schematic cross-section of the centrifuge model is shown in Figure 15. A typical data set from this series of centrifuge tests for the numbered locations in Figure 15, are presented in Figure 16.

![Fig. 14: Failure of Pile Group G4 (inset shows the extracted pile group after the dynamic centrifuge test)](image1)

![Fig. 15: Schematic Diagram of Centrifuge Models with Piles Founded in Layered Soils (liquefiable sand layer overlying dense sand layer)](image2)
quality data and the associated insights into soil behaviour can only be obtained from dynamic centrifuge tests.

Using the centrifuge test data as outlined above, Knappett & Madabhushi (2008 and 2009c) have proposed a simplified design procedure for pile groups in liquefiable soils. For example, if the designer wishes to limit the pile settlement to 0.1D_o (10% of pile diameter), then constants A and B are 5.5 and 3.5 respectively. The above equation is shown to provide a very satisfactory fit to the available experimental data. For example for the case of the pile group settlement to 0.1D_o, the available experimental data fits Equation 3 very well as shown in Figure 17.

More complex design cases, such as pile groups in liquefied and laterally spreading soils are considered extensively by Madabhushi et al. (2009).

The above methodology for design considers the pile group settlements that a designer can allow based on the superstructure tolerance for settlement and other criterion. Of course it is necessary to be able to estimate the amount of excess pore pressure that will be generated during the design earthquake at the base of the pile. This can be done from site response analysis programs such as CYCLIC-1D or more research based but generalised finite element codes like SWANDYNE, Chan (1988).

6. CONCLUSIONS

The role of geotechnical engineers is evolving and it is often necessary to estimate ground and foundation deformations. In geotechnical earthquake engineering, this is quite an important step as earthquake loading is an extreme event and adequate performance in terms of foundation deformations is the basis for the Performance based designs.

Dynamic centrifuge modelling offers a unique opportunity to gain insights into complex soil behaviour under earthquake loading. In this paper the problem of single piles and pile groups located in liquefiable soils is used to demonstrate the usefulness of centrifuge modelling. It was shown that using
this modelling technique, failure mechanisms can be identified correctly and as result established ideas can be changed. Buckling of single piles and pile groups was investigated but the centrifuge tests revealed changes from the anticipated classical buckling. As a result, the importance of dilation of liquefied soil on shearing was recognised and incorporated into appropriate numerical models.

The knowledge gained with single piles and pile groups located on bedrock was extended for the case of two layered soil strata. The centrifuge test data again provided valuable insights into the changes in the pile base capacity and pile shaft capacity as the surrounding soil suffered liquefaction. An example of simplified design procedure was presented. This procedure considers the requirement of limiting settlements to satisfy performance based criteria.

The important role of dynamic centrifuge modelling in earthquake geotechnical engineering has been emphasised in this paper. It is anticipated that with the growth in performance based designs the role played by this modelling technique will also increase, as geotechnical engineers seek a more thorough understanding of soil behaviour under cyclic loading.

REFERENCES


