

## Understanding Seismic Behaviour of Pile Foundations

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### ABSTRACT

Pile foundations are widely used around the world to transfer loads from the superstructure into competent soil strata below the ground. Earthquakes can create additional loading scenarios on piles that need special attention. This is particularly true if the pile foundations pass through shallow, liquefiable layers such as saturated, fine sands or silts. There are many examples of distress suffered by pile foundations following soil liquefaction in recent earthquakes. In this paper the experiences from the New Zealand earthquake of 2011 and Haiti earthquake of 2010 will be presented. Several failure mechanisms of piles in liquefiable soils such as pile buckling and excessive settlement of piles in liquefied soil layers were developed at Cambridge University based on extensive centrifuge experiments. It will be shown that some of these anticipated failure mechanisms of pile foundations are confirmed by the observations in Haiti. The value of dynamic centrifuge modelling in deciphering the pile behaviour through observation of failure mechanisms is emphasised.

### 1. INTRODUCTION

Pile foundations are used extensively worldwide to transfer superstructure loads from ground surface to more competent soil strata. This is particularly true when loose, sandy/silty soil deposits or soft clay layers are present at the site underlain by stiffer soil deposits such as dense sand or stiff clay layers. Understanding the performance of piles during earthquake loading is important while designing in the seismic regions of the world.

Soil liquefaction can result in extensive damage to many man-made structures such as buildings, bridges, tunnels and other infrastructure. Pile foundations supporting these structures need to withstand additional seismic loading while continuing to carry the normal gravity loads. Soil liquefaction can pose particular problems to pile foundations as their load carrying capacity can change both due to degradation of the end bearing as well as the shaft friction. In the first half of this paper, examples of pile

behaviour in liquefied soils will be presented from two of the recent earthquakes namely New Zealand earthquake of 2011 and the Haiti earthquake of 2010. The plausible mechanisms of failure will be identified and comparisons will be made to failure mechanisms derived from centrifuge test results. In the second half of the paper, emphasis will be placed on dynamic centrifuge testing and how this modeling technique can help clarify the failure mechanisms at play in the case of pile foundations in liquefiable soils. Pile buckling failure and pile settlements following soil liquefaction will be considered in detail.

**2. Pile behaviour in liquefiable soils in recent earthquakes**

*2.1. New Zealand earthquake of 2011*

The city of Christchurch in New Zealand was rocked by a strong earthquake measuring 6.3 on Richter scale on 22 February 2011. This earthquake caused extensive damage to buildings in the Central Business District and eastern suburbs of the city. Extensive liquefaction was observed in areas close to the Avon river, that flows through Christchurch. Many bridge foundations were affected by liquefaction in this earthquake. A few examples are presented here.

In Figs. 1 and 2 the damage to a small bridge is presented. The bridge itself remained operational although there was some settlement of the approaches on either side. The abutments suffered rotation due to liquefaction induced lateral spreading of the soil. The abutments were supported on concrete square piles that were sheared off due to the tendency of lateral translation and rotation of the abutment wall as seen in Fig. 2.



Fig. 1 A highway bridge



Fig. 2 Sheared piles of the abutment

In Figs. 3 and 4, liquefaction induced damage to a second bridge is presented. Liquefaction induced lateral spreading has caused the slopes to move towards the river subjecting the abutments to additional forces that caused rotation. Again this bridge remained operational although the abutment has suffered rotation as seen in Fig.3. The abutment was supported on steel H piles. These piles did not shear off as the concrete

piles of the previous bridge, but instead formed plastic hinges close to the pile head to allow for the rotation of the abutment walls as seen in Fig.4.



Fig. 3 Rotation of the abutment



Fig. 4 Damage to steel H piles of the abutment

The examples presented in Figs. 1 to 4 illustrate the additional loading on pile foundations due to earthquake induced liquefaction and their subsequent failure.

## 2.2. Haiti earthquake of 2010

On 12th January 2010, a powerful earthquake with a moment magnitude  $M_w=7.2$  hit Haiti that has caused extensive damage to civil engineering infrastructure and led to a death toll of nearly 250,000. This earthquake led to an unprecedented effort of using satellite and pictometric imaging to evaluate both the scale of damage and in damage assessments, Saito et al (2010), Booth et al (2011), Madabhushi et al (2012). While the detailed analysis of such imagery is beyond the scope of this paper, specific use of high resolution pictometric and pre and post earthquake imaging to assess the liquefaction damage at the port in Port-au-Prince will be considered in this paper. More details of the EEFIT mission to Haiti that was undertaken with support from EPSRC, UK and support from Institution of Structural Engineers, London can be found at <http://www.istructe.org/knowledge/EEFIT>.

Soon after the earthquake, it became clear that extensive liquefaction has occurred in the vicinity of the main port in the capital city, Port-au-Prince. Excessive settlement of container crane structure as shown in Fig. 5 made the use of port to receive emergency relief from rest of the world impossible. This situation highlighted the importance of designing critical facilities such as ports and harbours to withstand earthquake loading.

Pre and post earthquake images reveal the extent of damage to the port facilities as shown in Fig. 6.



Fig. 5 Settlement of container crane in Port-au-Prince

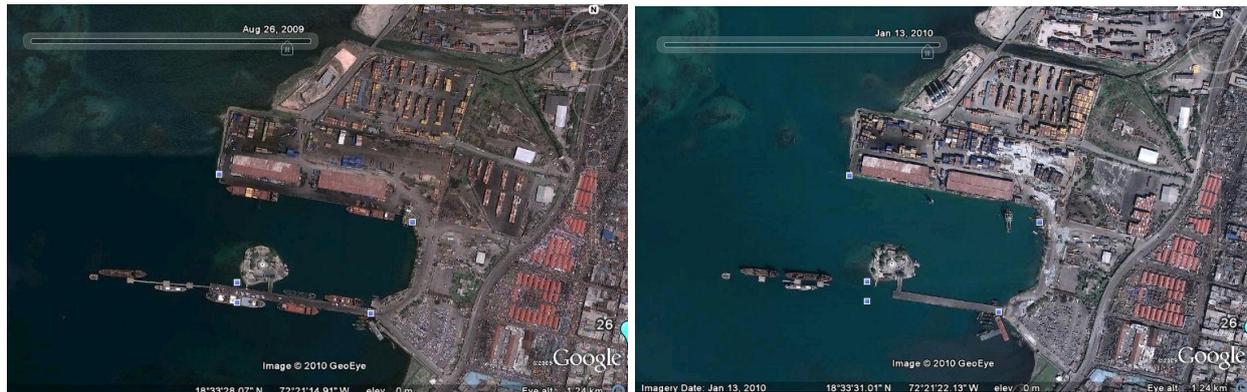


Fig. 6 Pre and Post-earthquake satellite images of the port

Comparing these two satellite images it can be seen that several sections of the south wharf have collapsed and the entire section of the north pier has collapsed. In addition to the satellite images, high resolution pictometric images were also available for the port. Pictometric images are aerial photographs taken in-flight akin to the street-view images of GoogleMaps. These offer an oblique view of the target and give a perspective of damage suffered by the structures. An example of these pictometric images is shown in Fig.7 that clearly depicts the damage to container crane seen in Fig.5 above.

In addition to the above aerial imagery, the EEFIT field mission was also able to obtain valuable data on performance of the port structures. The island seen near the top-left hand corner in Fig. 7 has suffered excessive settlement following liquefaction.

As a result, connecting bridge and wharf structures supported on pile foundations showed severe damage.

In Fig. 8 below the damage suffered by the bridge is shown. As the pier structure and the approach abutment on the island side settled, the bridge decks cracked to allow for these settlements. This caused the 'self-articulation' of the bridge.



Fig. 7 Pictometric image showing the damage to the container crane structure



Fig.8 Settlement of piers cause self-articulation of the bridge structure

Similarly a wharf structure suffered excessive damage as the pile foundations supporting the wharf settled into liquefied soil. This is shown in Fig. 9 which depicts the formation of plastic hinges at the pile heads. This type of failure mechanism was predicted by Madabhushi et al (2009a) based on a series of extensive centrifuge tests that were conducted to understand settlement of piles into liquefied sand overlying dense sand strata. More details of this study are discussed later in this paper.



Fig. 9 Hinging of piles supporting a wharf structure

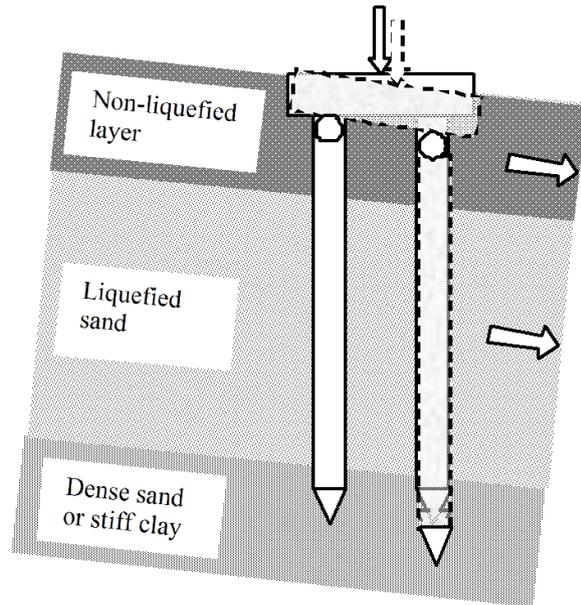


Fig. 10 Failure mechanism of hinging due to pile settlements, Madabhushi et al (2009a)

Comparing Figs. 9 and 10, it can be concluded that the failure mechanism of piles in liquefiable soils derived from dynamic centrifuge tests was validated from the observations following Haiti earthquake.

### 3. 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 Fig.11 a view of the 10 m diameter Turner beam centrifuge at Cambridge is presented.



Fig. 11 The 10m diameter Turner beam centrifuge

The success of earthquake geotechnical engineering at Cambridge depended to a large extent on the simple mechanical actuators that have been used for more than 30

years. First attempts of centrifuge shaking tables were mechanical 1D harmonic devices based on leaf spring device (Morris, 1979), bumpy road tracks (Kutter, 1982). The current earthquake actuator at Cambridge relies on Stored Angular Momentum (SAM) to deliver powerful earthquakes at high gravities was developed and is in operation for 14 years, Madabhushi et al (1998). In Fig. 12 the front view of the SAM actuator is shown while in Fig. 13 a view of the SAM actuator loaded onto the end of the centrifuge is presented. In Table 1 the specifications of the SAM actuator are presented. The model seen in Fig. 13 was from an investigation carried out by Haigh and Madabhushi (2005, 2011), on lateral spreading of liquefied ground past square and circular piles.



Fig. 12 A view of the SAM earthquake actuator

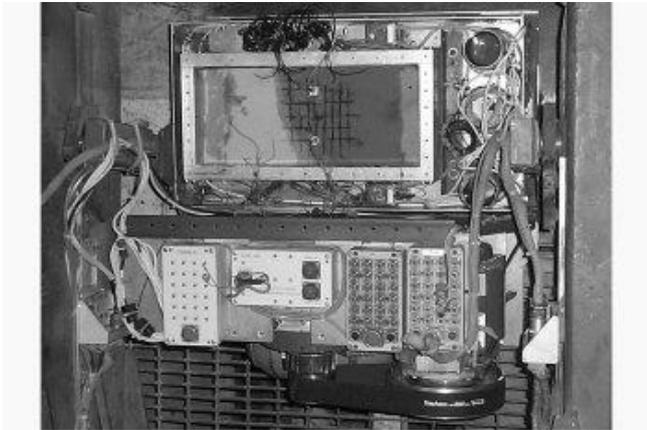


Fig. 13 Plan view of a centrifuge model of sloping ground with square and circular piles loaded on the centrifuge

Table 1 Specifications of SAM actuator

Parameter	Value
Maximum g-level of operation	100 g
Dimension of the soil models	56 m (L) × 25 m (B) × 22 m (H)
	80 m (L) × 25 m (B) × 40 m (H)
Earthquake strength of choice	Up to 0.4g of bed rock acceleration
Earthquake duration of choice	From 0 s to 150 s
Earthquake frequency of choice	From 0.5 Hz to 5 Hz
	Swept sine wave capability

*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  $\pm 2.5$  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.

More recently a new servo-hydraulic earthquake actuator was developed to enhance earthquake testing capabilities at Cambridge. This device is able to complement the capabilities of SAM actuator with the ability to simulate desired realistic earthquake motions such as a Kobe earthquake motion or Northridge earthquake motion. This device was custom-built to adapt to the existing services and configuration of the 10m Turner beam centrifuge shown in Fig. 9. Its maximum operational g level is limited to 80g. The specifications of this actuator are presented in Table 2.

An early example of the performance of the new servo-hydraulic earthquake actuator is presented in Fig. 14. The shaker was commanded to perform a Kobe motion and it responded satisfactorily as seen in Fig. 15. The higher frequency components still need further amplification as can be seen in the Fourier transform of the demand and achieved traces in Fig. 16. More details on the performance of this servo-hydraulic shaker are presented by Madabhushi et al (2012).



Fig. 14 A view of the new Servo-Hydraulic earthquake actuator

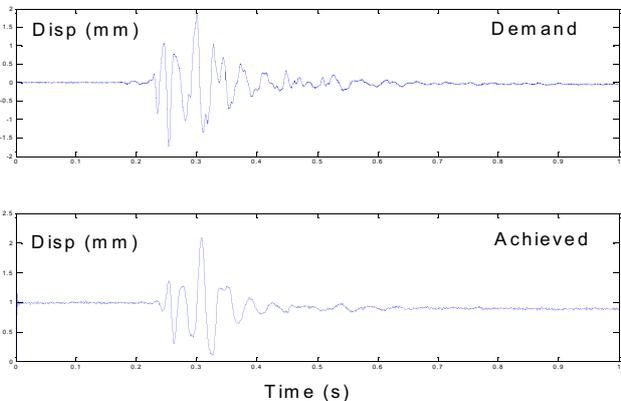


Fig. 15 Simulation of the Kobe earthquake motion

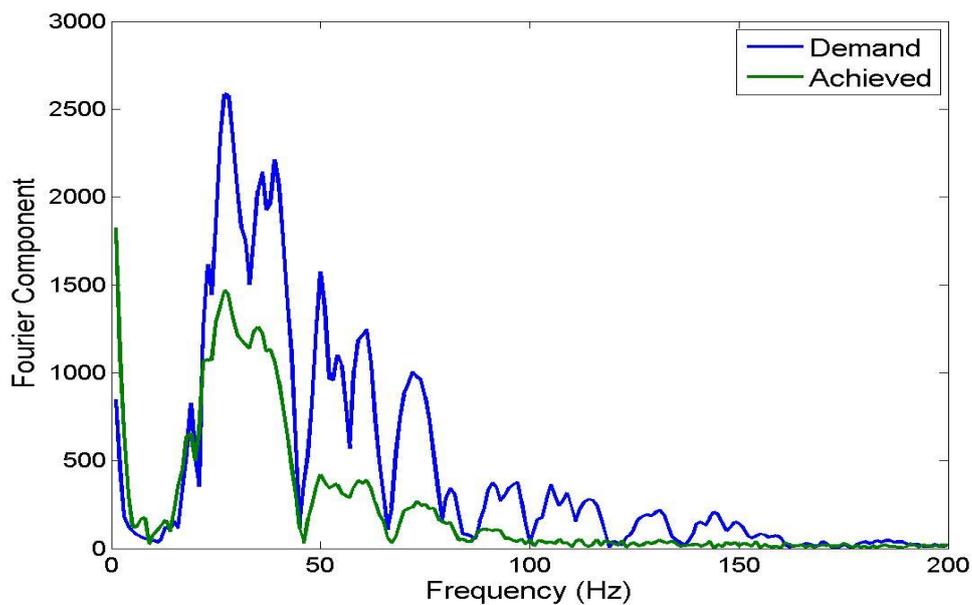


Fig. 16 Fourier spectra of the demand and achieved traces

Table 2 Specifications of the new Servo-Hydraulic actuator

Parameter	Value
Maximum g-level of operation	80 g
Dimension of the soil models	56 m (L) × 25 m (B) × 22 m (H)
	80 m (L) × 25 m (B) × 40 m (H)
Earthquake strength of choice	Up to 0.6g of bed rock acceleration
Earthquake duration of choice	From 0 s to 80 s
Earthquake frequency of choice	From 0.5 Hz to 5 Hz
	Realistic earthquake motion capability

*Note: All parameters above are in prototype scale*

#### 4. MODELLING OF 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, Reddy and 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 Fig. 17 by forming a plastic hinge close to the base of the pile. Madabhushi et al (2009a) 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 bed rock.

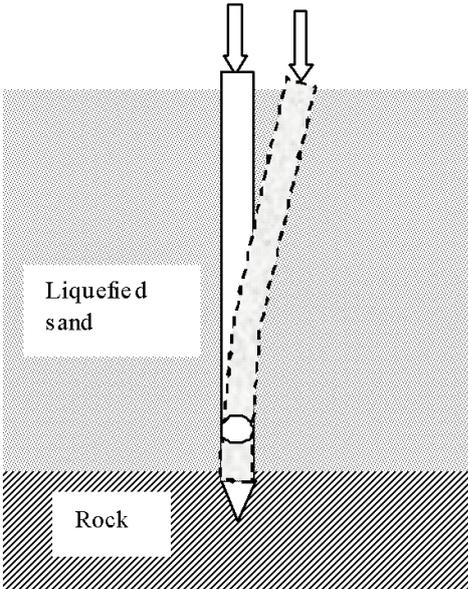


Fig. 17 Buckling of a pile in liquefied soil

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;

$$P_{cr} = \pi^2 \frac{EI}{(\beta L)^2} \tag{1}$$

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,  $\beta$  is 2. For a pile group, where both ends of the pile may be considered as fixed,  $\beta$  is 0.5.

The centrifuge model was tested at 50g’s and subjected to a base acceleration of 0.2g at the bedrock level. This caused the loose sand to liquefy fully which was confirmed by excess pore pressure measurements as shown in Fig. 18. Also sand boils can be seen on the soil surface in Fig. 19. The piles have suffered buckling failure as seen in Fig. 19. After the test the model piles were extracted from the soil model to

examine the location of plastic hinge formation. As seen in Fig. 20, the plastic hinging occurred at approximately 1/4 of the length below the pile head.

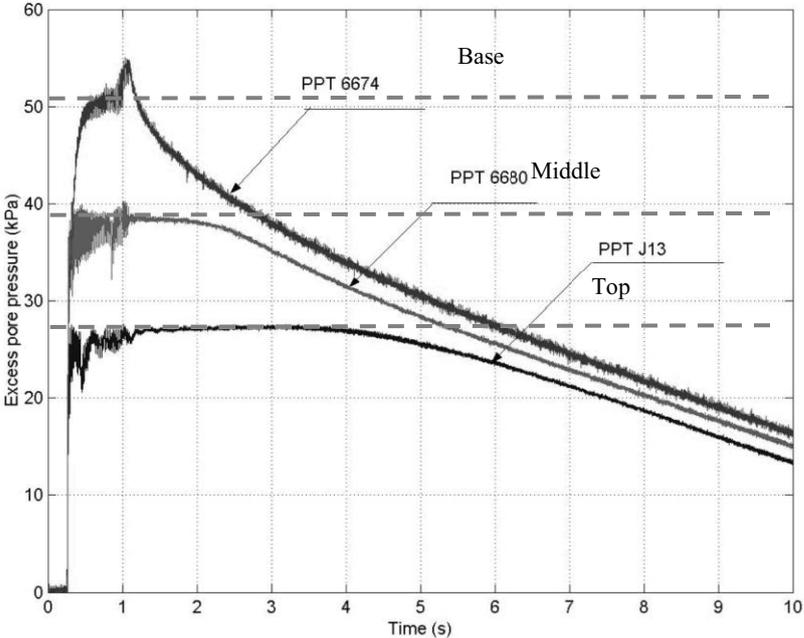


Fig. 18 Excess pore pressure time histories at base, middle and top of the sand layer (dashed line indicates full liquefaction)

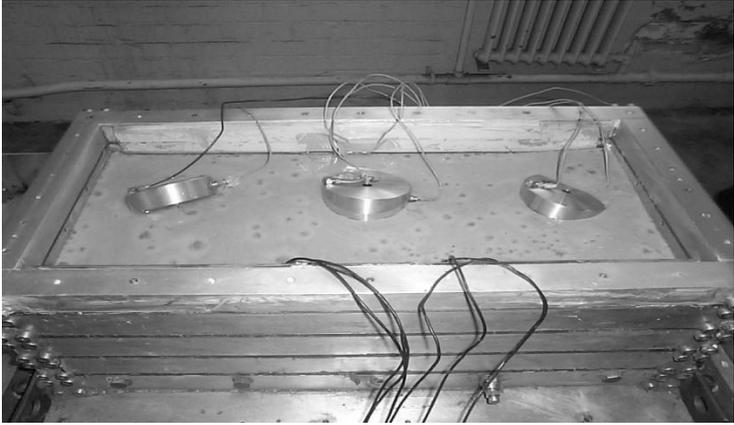


Fig. 19 Centrifuge model showing buckling of three piles in liquefied soils

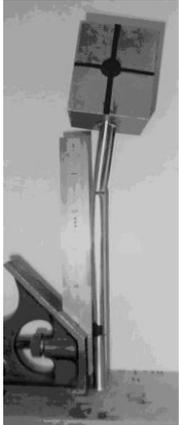


Fig. 20 A view of the buckled pile

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 Fig. 17 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 Fig. 20. 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? 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.1). This aspect is considered next.

## 5. 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 (2009a) describe several kinematically admissible failure mechanisms for pile groups by forming plastic hinges. Typical examples for liquefiable level ground are shown in Figs. 21a and b with four-hinge and three-hinge mechanisms respectively.

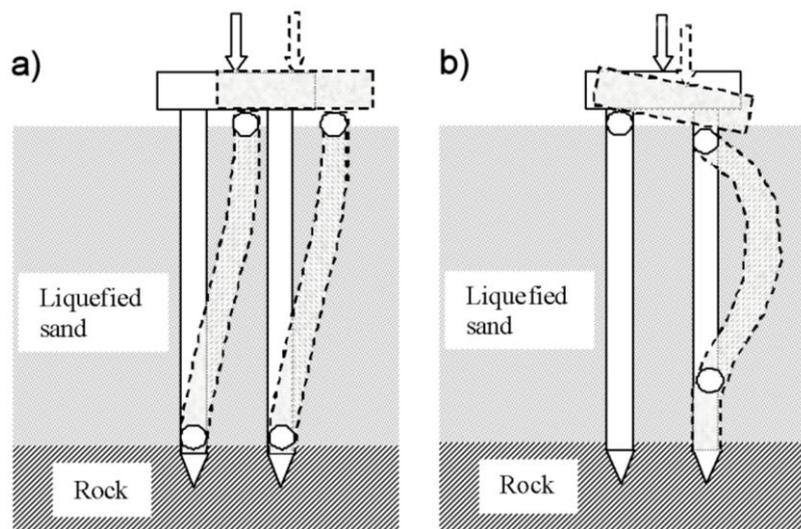


Fig. 21 Possible failure mechanisms for pile groups

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, 2x2 model pile groups made from aluminium alloy were tested in loose saturated sands. A typical cross-section of the

centrifuge models is shown in Fig. 22. It must be 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.

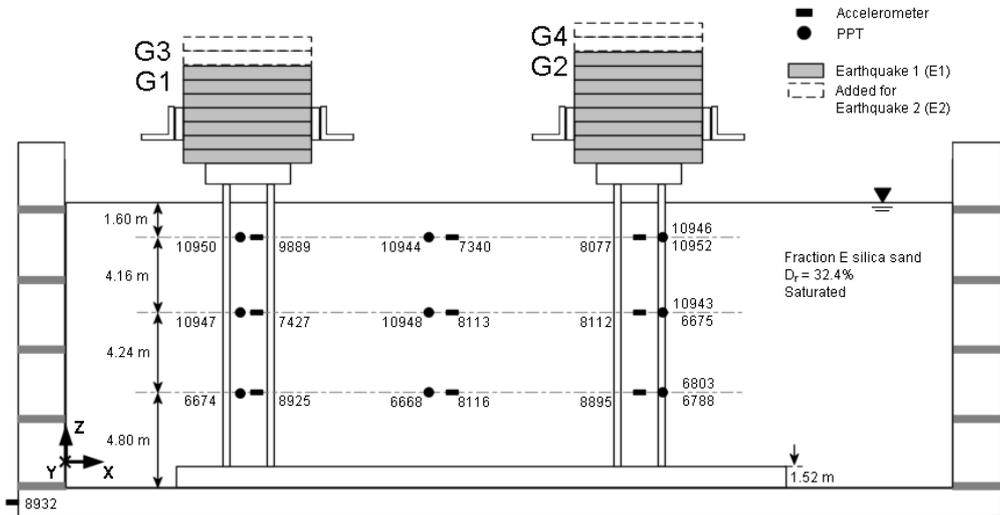


Fig. 22 Typical cross-section of the centrifuge model of the pile groups

In Fig. 23 the excess pore pressures in the free-field are presented. As before, the dashed lines indicate full liquefaction levels at the corresponding depths. In Fig. 23 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 Fig. 23. 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.

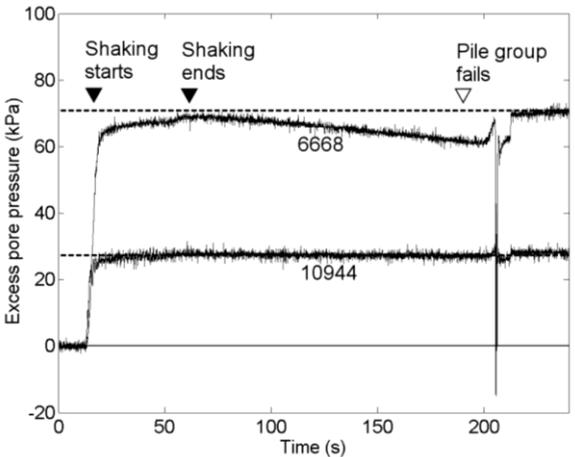


Fig. 23 Free-field excess pore pressures that confirm full liquefaction

The excess pore pressure traces close to pile group G4 are presented in Fig. 24. 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 Fig. 24. 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 Fig. 25. 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.

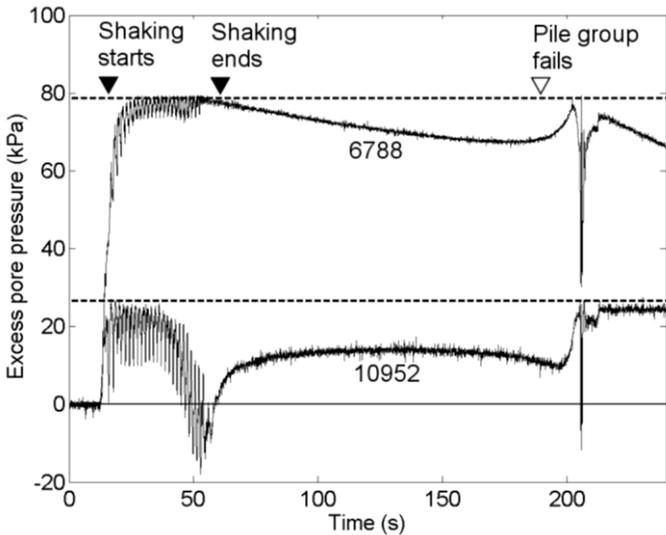


Fig. 24 Excess pore pressures close to the pile group G4

A view of the centrifuge model in plan after the test is shown in Fig. 26. 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 extracted after the centrifuge test is also shown as an inset in Fig. 26. 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 Fig. 21a 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 and 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 Fig. 26.

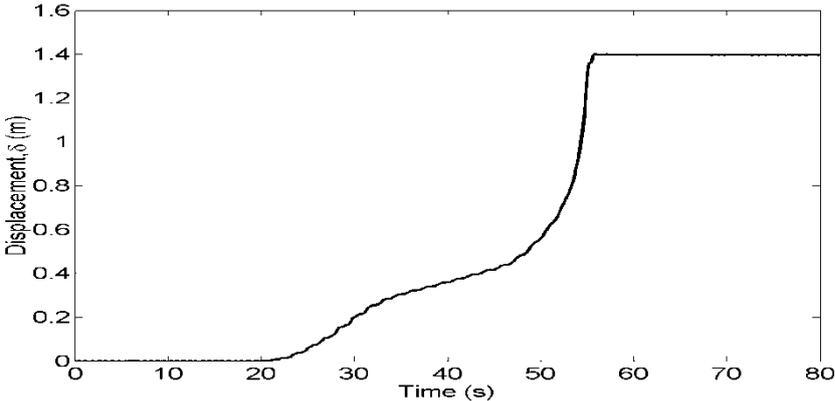


Fig. 25 Lateral displacement of the pile group G4

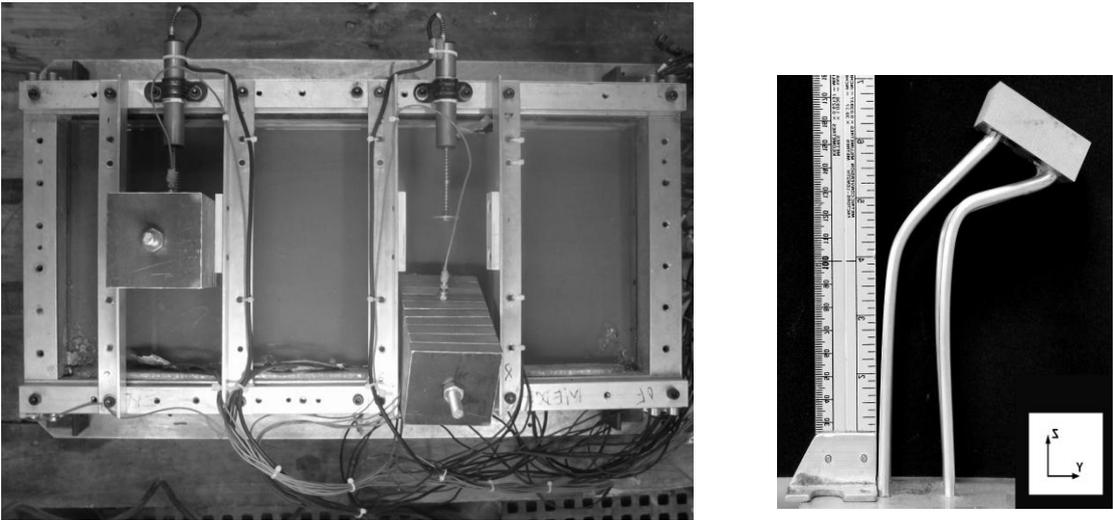


Fig. 26 Failure of pile group G4 (inset shows the extracted pile group after the dynamic centrifuge test)

**6. SETTLEMENT OF PILE GROUPS IN LIQUEFIED SOILS**

In Secs. 4 & 5 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. This more common case of pile foundations was investigated using dynamic centrifuge modelling as described by Knappett and 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 ( $\approx 10 D_0$ ) 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 Fig. 27. A typical data set from this series of centrifuge tests for the numbered locations in Fig. 27, are presented in Fig.28.

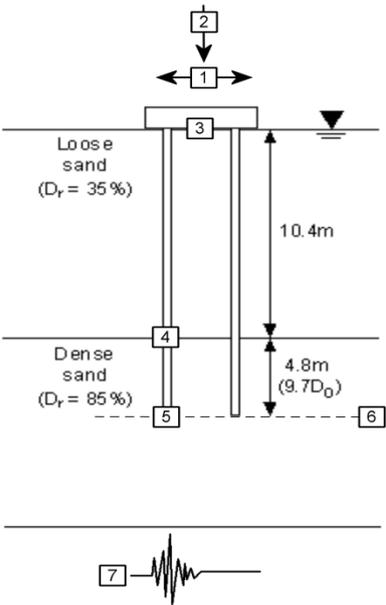


Fig. 27 Piles founded in layered soils (liquefiable sand layer overlying dense sand layer)

In Fig. 28, the input earthquake acceleration is given at the bottom as trace 7. The centrifuge test data shows that significant excess pore pressures are developed at the pile tip locations as shown by trace 6 and full liquefaction levels ( $ru = 1$ ) are reached quickly. It is interesting to note that such large excess pore pressures are generated even in the dense sand layer in which the pile tips rest. The horizontal response of the pile cap is given by trace 1 and this shows that after the first few cycles the pile cap oscillations are reduced and liquefaction sets in and the pile group is isolated from bedrock motion. The vertical settlement of the pile cap is given by trace 2, which shows

almost continuous settlement during the whole shaking with an ultimate settlement of about 0.5m. Traces 4 and 5 show the base capacity and shaft capacity respectively, for one of the pile. As the earthquake loading progresses and liquefaction sets in, it can be seen that the base capacity decreases significantly. The shaft capacity of the pile also reduces. As a result the bearing pressure of the pile cap increases as shown by trace 3. This type of high quality data and the associated insights into soil behaviour can only be obtained from dynamic centrifuge tests.

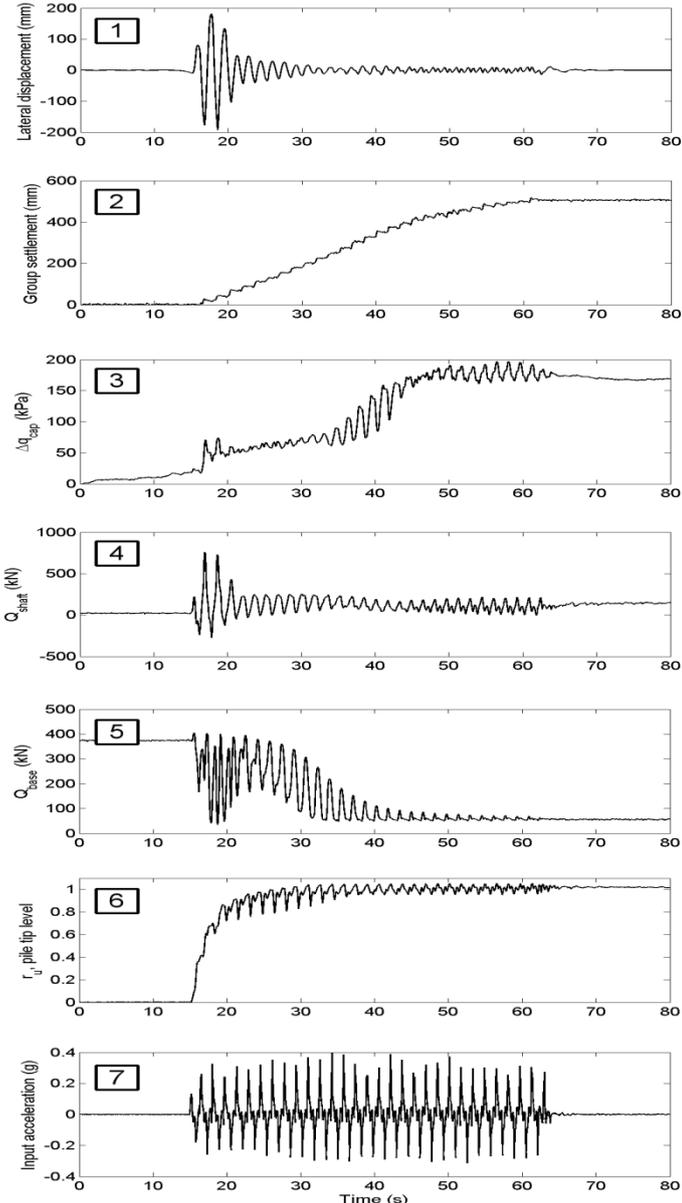


Fig. 28 Typical centrifuge test data from the pile group in two layered model

Using the centrifuge test data as outlined above, Knappett and 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 in this type of layered soil strata to a given value, they suggest that a minimum factor of safety can be estimated as,

$$FOS = 1 + A(r_{u,base})^B \quad (2)$$

where  $r_{u,base}$  is the excess pore pressure ratio at the tip of the pile,  $A$  and  $B$  are constants. 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 Eq. 2 very well as shown in Fig. 29.

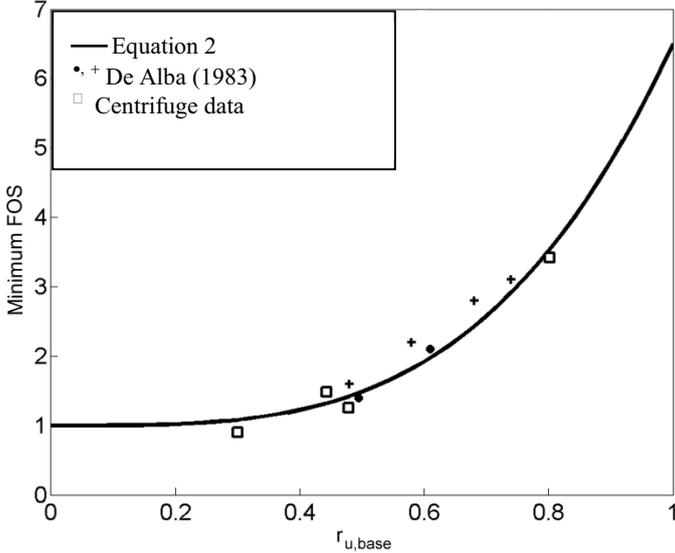


Fig. 29 Minimum factor of safety for a given liquefaction ratio at the base of the pile group

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

## CONCLUSION

Powerful earthquakes continue to cause severe damage to pile foundations, particularly when they are located in liquefiable soil strata. The Haiti earthquake of 2010 provided its share of examples of liquefaction damage. Port structures were found to be particularly vulnerable to liquefaction damage. In this paper examples of use of satellite and pictometric images were presented that can be used in rapid damage assessment immediately after the earthquake. In addition ground based investigations were very useful in recording the damage to pile foundations supporting bridge piers and wharf structures.

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 a 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 assisting earthquake geotechnical engineering has been emphasised in this paper.

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