

The Uplift Capacity of Under-reamed Piles
by
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“I hereby declare that, except where specifically indicated, the work submitted herein is my own original work.”

Signed:

Date:

Technical Abstract

Under certain conditions buildings and structures can exert uplift forces on their foundations. One method of dealing with this is through the use of under-reamed piled foundations, which have an enlarged base to increase the uplift capacity of the pile.

Currently most studies into piles with base enlargements have considered the case of piles with flat anchor plates attached to the base. Very little literature exists that considers the effect of the under-ream angle on the uplift capacity of the pile.

For this research a series of experiments were performed at 1-g in both loose and dense sand. The first series of experiments used anchor plates to examine the effect of varying the diameter of the base enlargement on the uplift capacity. The second series of tests looked at different under-ream pile shapes to examine the effect of varying the under-ream side angle on the uplift capacity. The aim of the experiment was not to use the model piles to predict the uplift capacity of full scale piles, but to carry out a comparative study between anchor plates and different under-ream shapes in order to determine whether the same manner of failure could be observed. The results were also compared to an existing theory for ultimate uplift capacity of anchor plates and a theoretical approach to predicting the load-displacement response.

It was found that the uplift capacity of the anchor plates increased with increasing base diameter and increased relative density of the sand. The stiffness of the response of the piles in dense sand was found to be far greater than in loose sands, with the uplift capacity being reached at lower displacements. The theory proposed for determining the ultimate uplift capacity was found to generally over-predict the resistance of the anchor plates in loose sand but provided a good estimate in dense sand. In contrast, the theoretical load-displacement response provided a good approximation for the loose sand but was not applicable to the response in dense sand.

The tests on the under-reamed pile bases found that in loose sand, the load-displacement response for bases with a large under-ream angle (measured with respect to the horizontal) was different to those with smaller under-ream angles, implying that a different mode of failure was taking place. In dense sand, the prediction for ultimate uplift capacity from the theory was found to correspond well with the experimental results, with no pattern for under-ream angle observed. However, the smaller the angle, the more gradual the drop in strength

after the maximum capacity had been reached. It was also observed that at low density, the greater the under-ream angle, the smaller the displacement to failure.

More experiments need to be carried out in order to examine the angles at which the change in failure mode occurs depending on the angle of friction of the soil and the pile-soil interface. Experiments using a half space to observe movement of the sand would provide insight into the mode of failure of the under-reamed piles using the modern visualisation techniques based on digital imagery.

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Nomenclature

Δ	Displacement at pile head [m]
α	Average angle of inclination of normal force on vertical failure surface through the edge of a footing [°]
δ	Interface angle of friction between the soil and the pile [°]
θ	Angle of pile under-ream surface to the horizontal [°]
τ_f	Average shear stress mobilised on a failure surface [Nm^{-2}]
ϕ_{crit}	Critical internal angle of friction of the soil [°]
ϕ_{max}	Maximum internal angle of friction of the soil [°]
ϕ_{mod}	Modified maximum internal angle of friction of the soil [°]
ψ	Angle of dilation of the soil during shearing [°]
B	Diameter of pile base enlargement [m]
D	Embedment of pile to pile base or to the bottom of the initial angled section of an under-reamed pile [m]
H	Vertical extent of failure surface above the base of the foundation [m]
K_p	Coefficient of passive earth pressure [-]
N_{qu}	Dimensionless breakout factor [-]
Q_u	Ultimate uplift capacity [N]

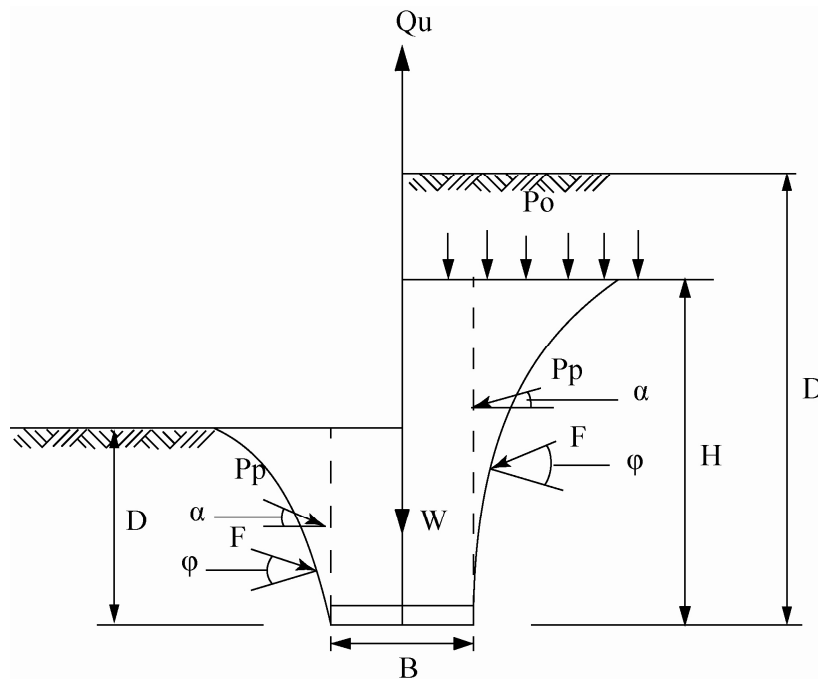


Figure A Failure surface for an anchor plate from Meyerhof & Adams (1968)

1 Introduction

1.1 Background

Buildings and structures exert loads on the ground, these loads are often transferred to the ground through deep foundations in the form of piles. The loads are usually compressive forces due to the mass of the structure. However, under certain conditions a foundation may experience an upwards tensile force and must therefore resist pull out. There are a number of situations in which these forces can occur. One example is wind forces applying a large overturning moment to the structure leading to compressive forces on the leeward side and tension forces on the windward side (see figure 1). Historically, large gravity base foundations have been used when large moments are expected on a foundation, however piled foundations using less concrete have become more economical. Uplift resistance is also required for offshore oil structures that are anchored to the ground to ensure that they remain in place under the dynamic loading conditions found at sea.



Figure 1 A transmission tower that could experience wind loading leading to uplift forces on the foundation, Savory (2009)

A special example where uplift resistance is required is where construction takes place on soils that experience large volumetric strains, for example Black Cotton Soil in India. Black Cotton Soil undergoes large changes in volume when swelling and contracting between

monsoon seasons, which lead to large displacements at ground level. In order to prevent these large displacements occurring in buildings at ground level, piles can have under-reams at a large depth to anchor the building where only small strains are occurring.

1.1.1 Under-reamed Piles

Under-reamed piles feature an enlarged base, or sometimes even a number of enlargements along their length. They are typically bored piles, with a special cutting tool used to ream out the base enlargement, as seen in figure 2. In the case of end bearing compressive piles, the base enlargement is used to increase the bearing capacity of the pile and for tension piles to provide added uplift resistance.



Figure 2 Under-ream excavator being emptied, [geoforum.com](http://www.geoforum.com) (2009)

1.2 Problem Definition

In the design of foundations both the Serviceability Limit State (SLS) and the Ultimate Limit State (ULS) must be considered. For anchored steel frames, small displacements are allowable, so the major consideration is the ultimate failure of the foundation. For buildings, the major problem occurs due to differential settlements leading to cracking in concrete or masonry elements of the buildings. The serviceability limit state aims to avoid the building becoming unusable and therefore is mainly concerned with displacements.

This research will examine under 1-g conditions the uplift capacity and the load-displacement response of a number of different anchor plates and under-reamed pile shapes in loose and dense sand at small scale. This research will form a comparative study between different shapes and will not aim to predict the exact settlement and load responses for full sized under-reamed piles.

1.3 Project Objectives

This project will aim to:

- Compare the uplift capacity of anchor plates of different diameters in loose and dense sand.
- Compare the uplift capacity of a number of different under-ream shapes with a fixed base and pile diameter.
- Review and discuss the analytical methods in the literature on the uplift capacity of under-reamed piles and anchor plates and then compare the experimental results to proposed theoretical models.

2 Literature Review

A large number of investigations into the uplift capacity of plate anchors have been published, however, only Balla (1961) considered “mushroomed” or under-reamed piles. All other investigations have looked at plate anchors as an approximation for under-reamed piles or as a direct estimate for the uplift capacity of plate anchors. The following papers are those that are most relevant to this project. Early experiments purely aimed to evaluate and produce models for the ultimate uplift resistance, observing but not aiming to quantify the displacements to failure. More recent papers also consider methods for predicting the ultimate displacements to failure.

2.1 Ultimate uplift capacity

In the early 1960s, a number of papers were published considering the uplift capacity of plate anchors related to the foundations of transmission towers. Meyerhof & Adams (1968) examined the uplift capacity of anchor plates from both field test data and laboratory experiments. They found that there was a complex relationship between the embedment depth and the uplift capacity but used simplifying assumptions to develop a generalized theory. They also categorized the foundations as being either deep or shallow and provided a method to estimate which type a foundation corresponded to. It was also noted that the lack of agreement in the theory at the time was due to difficulties in understanding the shape of the failure zone.

Geddes & Murray (1987) considered previous results and carried out tests in cohesionless soil. They ascertained that the theory put forward by Balla (1961) using Kotters equations was not valid as they ignored the vertical stresses and only considered shear stresses, which did not allow their result to satisfy equilibrium. 19 years after the theory proposed by Meyerhof & Adams (1968) had been published they still noted that there was no unanimity of view in this field. They attempted to describe their experimental results in terms of upper and lower bound solutions using equilibrium and limit state analysis approaches. From their experiments they found a good relationship with Meyerhof & Adams (1968) for tests in dense sands but not in loose sands. The surface roughness of the anchor plates at deep embedment was also found to

have a large effect on the uplift capacity. Geddes & Murray (1987) also presented the uplift results as a normalized resistance in terms of $Q_u/\gamma'AH$.

Sharma & Pise (1994) tested piles at varying embedment depths and with varying pile to base ratios for anchor plates in sand. However, they did not consider different soil densities. They compared the experimental data to theories proposed by Meyerhof & Adams (1968), Sharma *et al.* (1978) and one that they had developed. They found that the closest relationship between the test results and theory was found with their own and the model proposed by Meyerhof & Adams (1968).

2.2 Ultimate uplift capacity and load-displacement relationship

Ilamparuthi *et al.* (2002) carried out a comprehensive review of all previous literature on the subject. They also carried out relatively large-scale experiments using round anchor plates in both complete testing tubs and on a half-space to observe the movement of the soil during failure. They grouped previous trials related to their method for predicting the uplift resistance:

- Vertical Strip Surface Models (VSSM) – these were found to be conservative
- Truncated cone extending above the anchor plate, equating the uplift of the soil with the weight of the cone, ignoring friction on the cone failure surface. They found these to be conservative for shallow anchors and not conservative for deep anchors
- Truncated cone extending above the anchor plate including both weight of the soil and the shear force along the failure surface in the total uplift capacity.
- The use of a circular arc to predict the failure surface of the sand.

The different failure surfaces are presented in Figure 3.

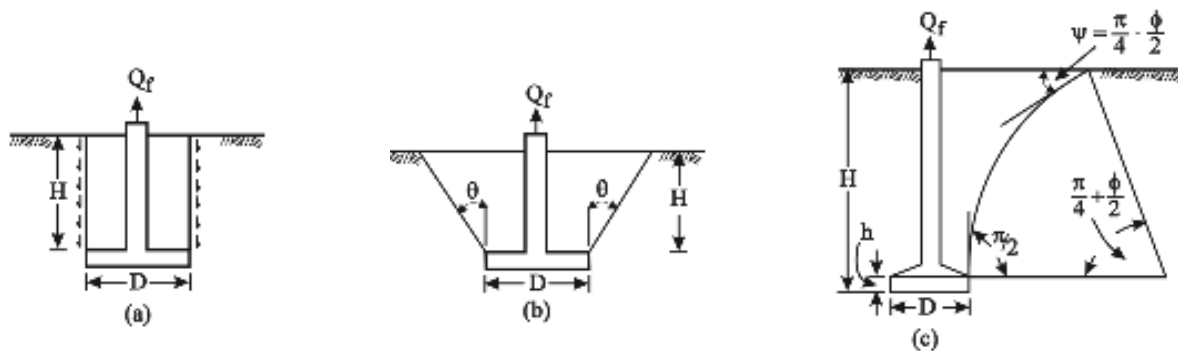


Figure 3 Assumed failure surface: (a) vertical slip surface model, (b) soil cone model, and (c) circular arc model (Ilamparuthi *et al.*, (2002))

They found that the relationship between displacement and embedment ratio was bilinear, allowing it to be used to distinguish between shallow and deep behaviour. They found that the Meyerhof & Adams (1968) values for critical embedment were in good agreement with their results. They also agreed that no entirely satisfactory theory for breakout capacity of anchors existed due to the difficulties in defining the failure surface.

Merifield & Sloan (2006) completed a rigorous numerical study using Finite Element modelling and the upper and lower bound theorems of limit analysis, rather than using conventional displacement finite element methods. They also compared their results to existing numerical and empirical solutions. However, their analysis only considered the plane strain case and therefore is not entirely applicable to round plate anchors and under-reamed piles. Their numerical study found a large dependence on interface angle between the pile and the ground, however this decreases for dense soils with high friction angles.

Vanitha *et al.* (2007) studied the load-displacement and ultimate response quantitatively for both single and group anchors. As with Ilamparuthi *et al.* (2002) they proposed an analytical model using the limit equilibrium method. They found that, using their analysis method and tests, the average of the maximum difference between the predicted and measured results was 25%.

Hanna *et al.* (2007) carried out similar experiments but on single flat plate and helical screw anchors at shallow depth. They also used a limit equilibrium approach to analyse their results. They used their results to estimate the zone of influence at the surface for determining pile spacing. The displacement of the pile was also determined to not have any effect on the displacement. In order to determine the failure load of dense sands under low confining pressure they used a modified mobilised friction angle proposed by Davis (1968), Drescher & Detourney (1993) and Michalowski (1997).

One of the latest papers to consider uplift of anchors by Bolton *et al.* (2008) again only considers plain strain but concludes that limit equilibrium solutions are preferred, although they over predict the area over which uplift occurs. Plasticity solutions are determined to be an inappropriate tool for analysing drained soil problems. It was explicitly stated that to apply the limit equilibrium approach the following input parameter would need to be evaluated:

$$Q = f(\gamma', \sigma_c, \phi_{crit}, I_D, H, B) \quad (2.1)$$

2.3 Conclusions

The experiments by Meyerhof & Adams (1968) have been found to still provide a useful method, with some adjustment for peak friction angle proposed by Hanna *et al.* (2007) . Therefore the Meyerhof and Adams approach combined with the modified friction angle will be used to compare to the experimental results in the research presented in this report. As the Hanna *et al.* (2007) is one of the latest papers, the theories on displacement will also be compared to the load displacement results from the experiments.

3 Experimental Equipment and Method

This section describes the equipment used to perform the testing along with the method for installing the piles and testing them in both loose and dense sand.

3.1 Test Container

The tests were performed in a large steel container 850 mm in diameter and 400 mm deep. The edge was pre-drilled containing holes to allow the installation of the actuator at almost any orientation, allowing a number of piles to be placed simultaneously for one test pour. The container was also chosen to provide adequate cover to the edges of the container and the bottom to avoid the container itself influencing the result.

3.2 Soil

The tests were performed in dry sand. Sand was chosen over clay for a number of reasons:

- Installation of the piles in sand was easier
- Pouring the sand and carrying out testing was achievable at a far quicker rate in sand than would have been possible in clay
- It was easier to achieve loose and dense soil states in a shorter amount of time.

Due to the time constraints imposed by 4th year projects, the option of doing a greater number of tests in sand was chosen. The sand used in the testing was Hostun HN31 sand. It is regularly used in geotechnical research as it is readily available in the lab, is uniform and is easy to handle. Properties for the Sand were taken from Mitrani (2006) (table 1). It is important to note however, that measurement of $e_{\min}$ and $e_{\max}$ is not exact, as the particle packing of the soil depends on its confining pressure and applied shear stresses. As a comparison, Flavigny *et al.* (1990) present another set of values listed in table 1.

Table 1 The properties of Hostun sand taken from a number of sources

	Matrini (2006)	Flavigny (1990) A	Flavigny (1990) B
$e_{\min}$	0.555	0.648	0.624
$e_{\max}$	1.01	1.041	0.961

For the calculations in this report, the most recent values from Matrini (2006) were used.

3.3 Model Piles, Plates and Under-reams

All of the model components were made from solid aluminium. The model piles were 400 mm in length and 12.7 mm in diameter. The anchor plates used were 6 mm in depth and had diameters of 36, 45 and 60 mm to achieve a range of embedment ratios D/B for the tests, where D is the embedment of the pile and B the anchor base diameter. Due to the thickness and scale of the experiments, it was assumed that the anchor plates were perfectly rigid throughout the tests and did not deform under loading. Similarly, any extension due to strains in the aluminium shaft was ignored, as they would be far smaller than the movement of the whole pile.

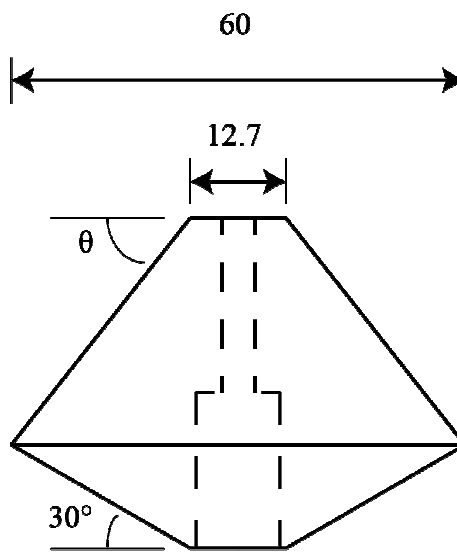


Figure 4 Model under-ream pile, with all dimensions in mm

The under-reamed piles used were all machined from solid blocks of Dural, an aluminium alloy as shown in figure 5. Figure 4 shows the dimensions of the under-ream pile bases. Each under-ream base had a diameter of 60 mm and a bottom angle of 30° , so that the main variable affecting the uplift resistance was the angle of the upwards face. The values for θ shown in figure 4 used were: 30° , 45° , 60° and 75° . An attempt was also made to produce one with a base angle of 15° , however this was difficult to machine. A slightly angled surface with the full angled base was produced. It was used to provide a comparison to the purely flat anchor plate.

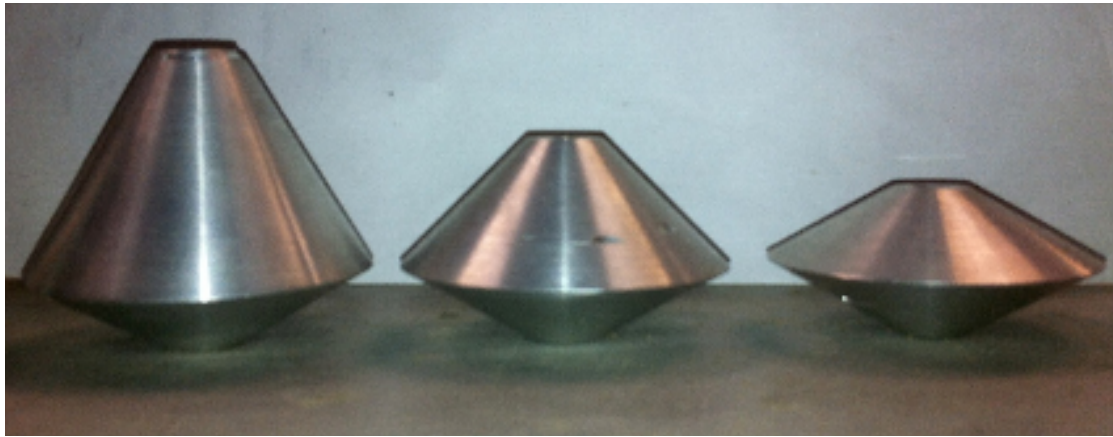


Figure 5 Model pile under-reams with angles of 60°, 45° and 30°

3.4 Actuator

An actuator, shown in figure 6, was used to apply a constant rate of displacement to the pile head in order to supply the required uplift force. The actuator was designed for use in centrifuge testing, and only required minor modifications to make it suitable for use in these tests. A shear box was used to control the rate of displacement of the piles. This was kept constant for each test, so as to ensure that rate of application of load was not a variable.

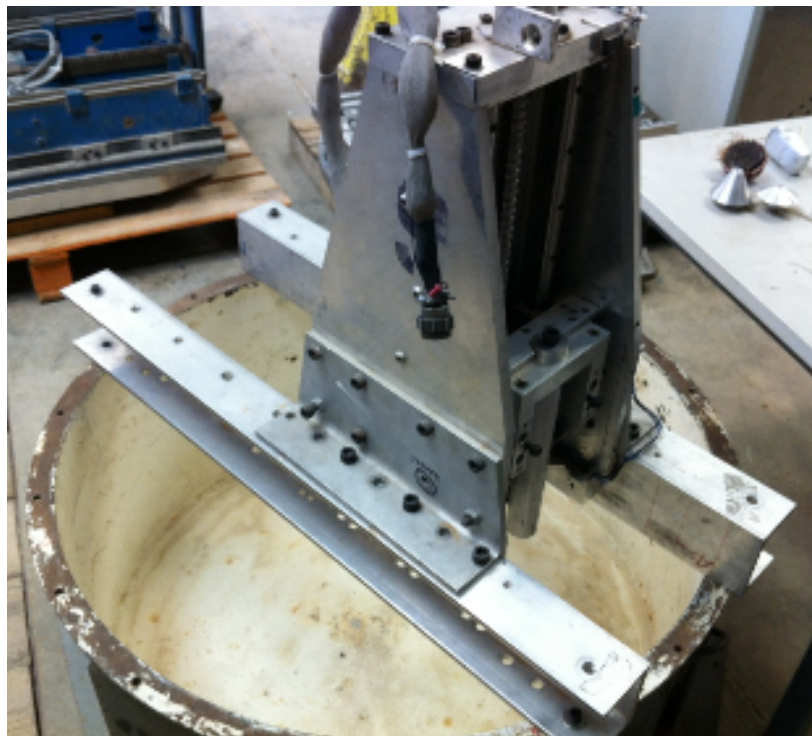


Figure 6 Actuator placed in position above empty test container

3.5 Instrumentation and Measurement

3.5.1 Load measurement

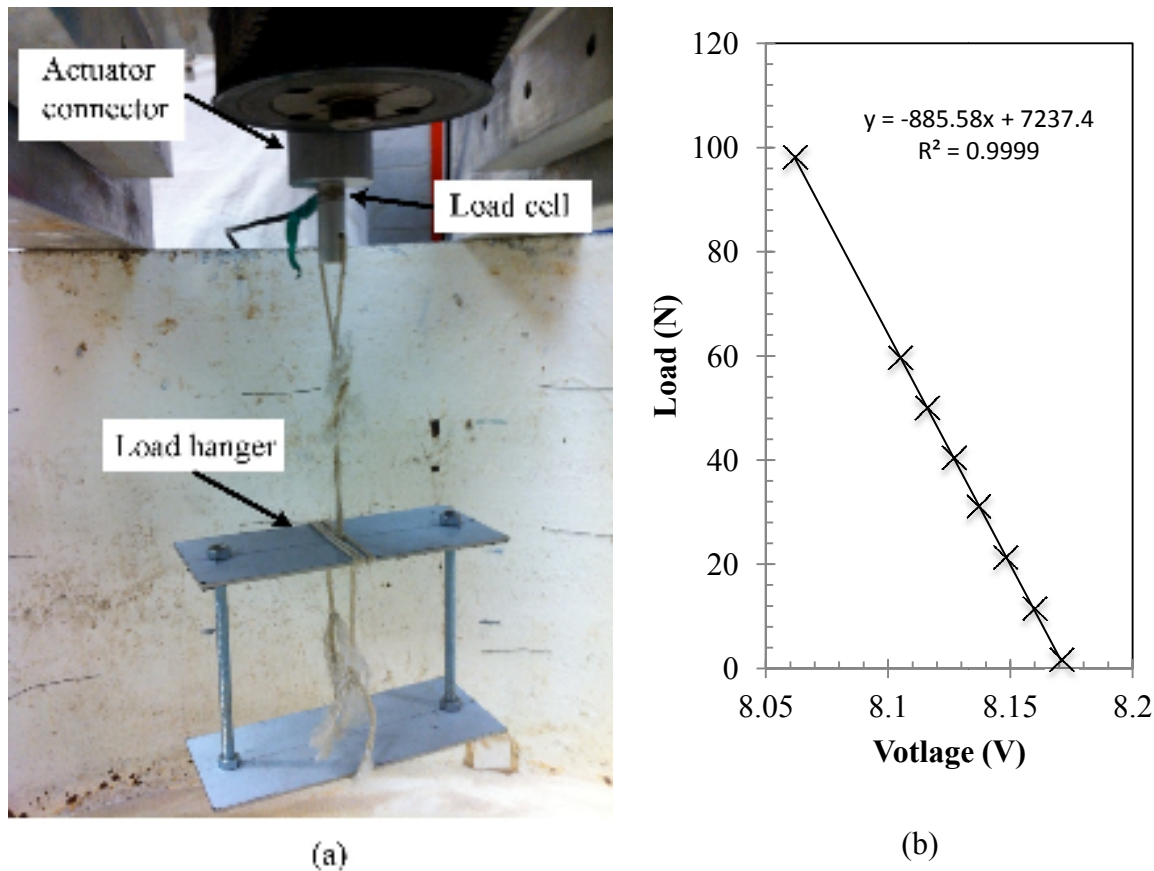


Figure 7 (a) Load cell calibration apparatus and (b) a typical calibration curve

Load measurement was carried out using a 1 kN Load cell placed between the actuator and the pile head. The load cell had to be calibrated to convert its voltage reading to force. In order to calibrate the load cell, the pile was replaced by a hanging carrier, on which the loading was carefully increased. The calibration equipment and a typical calibration curve for the load cell are shown in figure 7.

3.5.2 Displacement measurement

In order to accurately measure the displacement of the pile during testing, a Linear Variable Differential Transformer (LDVT) was used. The LDVT was placed above the 'carriage' of the actuator at the flat surface as shown in figure 8. In order to measure the displacement, the voltage output from the LDVT must be calibrated in order to gain the displacement in millimetres. The calibration rig used is shown in figure 8, with a similar calibration curve produced as that shown for the load cell, as both components maintained a linear relationship between voltage output and calibration values.

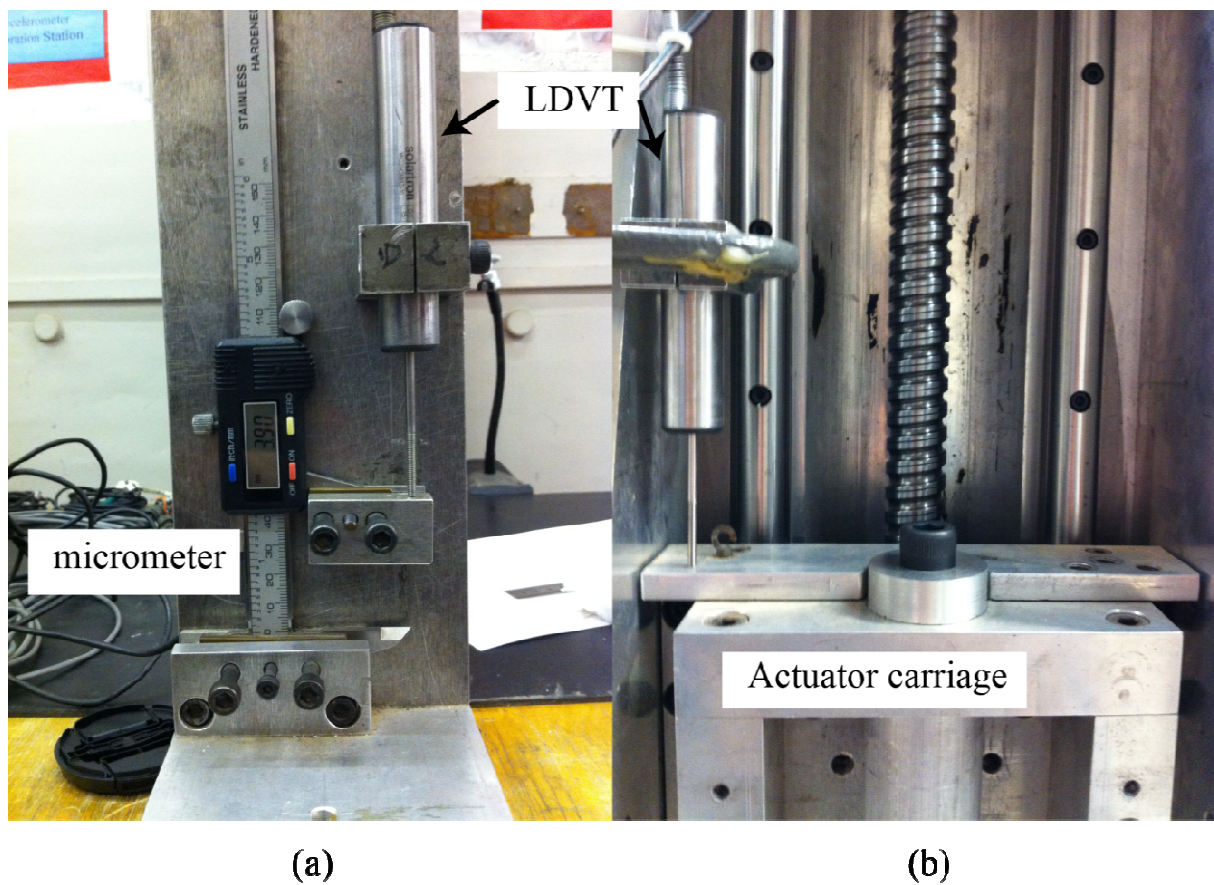


Figure 8 (a) LDVT calibration apparatus and (b) location on the actuator

3.6 Experimental Setup

3.6.1 Jig

A simple jig was manufactured to hold the piles in position during their placement and pouring of the sand, shown in figure 9. The jig was bolted into position using the holes provided for the attachment of the actuator to the test container. The holes were drilled such that the pile would line up below the actuator connection. It was important to ensure the jig was accurate, as the load cell used was very sensitive to applied moments, so all care had to be taken to ensure that the loading applied was directly through the load cell, without the connection being forced at any angles.

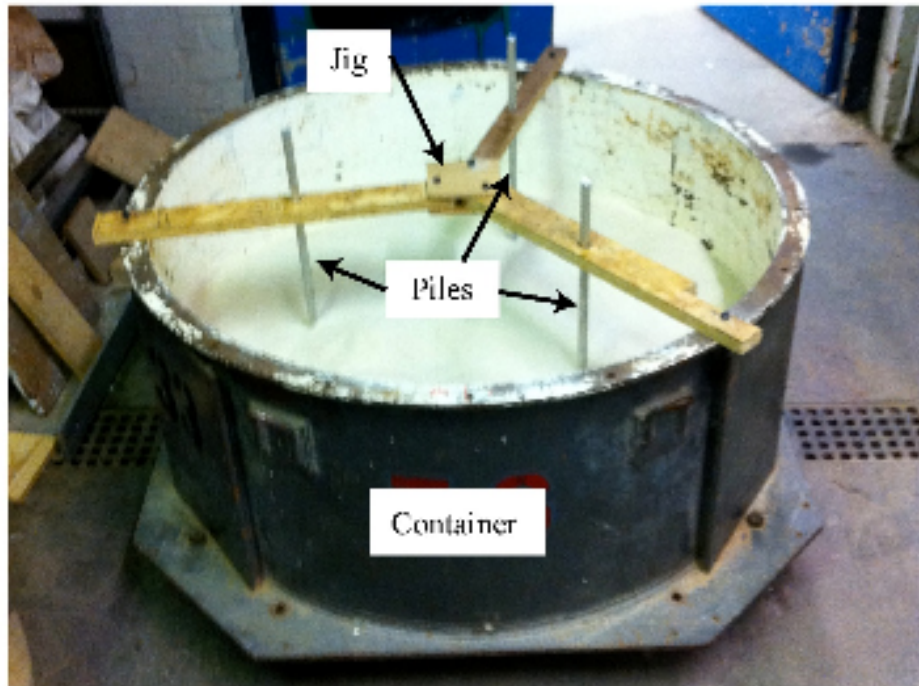


Figure 9 Test container showing jig used to secure piles during sand pouring and shaking

3.6.2 Sand pouring

In order to control the density of the sand during the test, the sand was poured using a hopper suspended above the container. A number of test pours were carried out into a cylinder of known volume that was then weighed to determine the density of the sand and then the relative density, I_D , of the soil. In order to achieve the desired I_D , the sand was poured at a constant rate from a height of 100 mm above the sand surface. In order to ensure uniformity of the sand, the initial pour took place in a crossed zig zag pattern shown in figure 10a. However, once the initial layer had been poured and the piles and jig placed into position, a new pattern had to be adopted to ensure the uniformity of the sand whilst working around the jig. This new pattern is shown in figure 10b only a small layer of sand was poured in each third at a time, with the pourer moving around the tub slowly to do each area in turn. Once the required mass of sand had been poured, the sand was levelled and the embedment of the piles recorded.

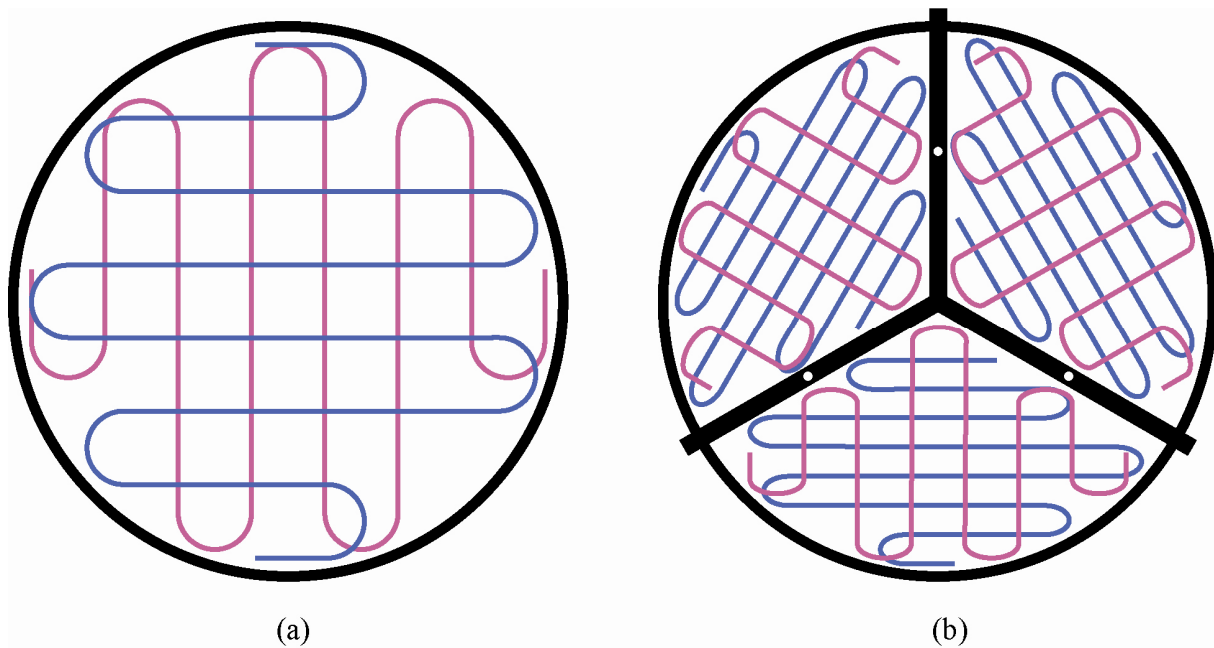


Figure 10 Sand pouring patterns for (a) the initial layer and (b) once the jig and piles were in place

For each test, the mass of sand placed in the hopper was recorded and the final depth of sand was measured to determine the volume. From this the voids ratio and relative density could be calculated.

3.6.3 Shaking

In order to increase the density of the sand the experiment was shaken on a shaking table until the level of the sand had reduced by the required amount. This saved time over adjusting the pouring rate and height to change the density, as the piles that had been tested in loose sand could be left in place, and then tested again at the increased density. The piles were held in position with the aforementioned jig while the shaking took place.

3.7 Experimental Procedure

The testing was carried out using the following procedure, which was repeated for each test that was carried out with the different pile bases:

- Pour initial 100 mm sand layer into container.
- Place jig and piles in correct position, ensuring that the piles are vertical, using a spirit level.
- Pour known mass of sand around installed piles up to the correct height.
- Remove the jig and place the actuator above the first pile.

- Attach the load cell and actuator to the pile and apply a constant rate of displacement, measuring the load and displacement throughout the test up to failure.
- Detach the pile and move the actuator round to test the next pile.
- Once all piles have been tested, remove the actuator and reattach the jig.
- Take test container to shaking table and shake until desired level change has been achieved. Repeat stages 4, 5 and 6 above until all the tests have been completed.
- Remove sand and clean tub in preparation for the next test.

3.8 Tests Performed

Table 2 details the tests that were performed:

Table 2 Details of tests performed

Name	Aim of test	Relative Density I_D	Base Diameter or Angle	D/B
ADH-1a	To compare the resistance of flat anchor plates in loose sand	0.17	36 mm	3.42
			45 mm	5.69
			60 mm	4.56
ADH-1b	To compare the resistance of flat anchor plates in dense sand	0.51	36 mm	3.00
			45 mm	4.78
			60 mm	4.86
ADH-2a	To compare the resistance of the under-reamed shape pile bases in loose sand	0.14	30°	3.50
			45°	3.50
			60°	3.50
ADH-2b	To compare the resistance of the under-reamed shape pile bases in dense sand	0.48	30°	3.07
			45°	3.07
			60°	3.18
ADH-3a	To provide tests for the remaining under-ream bases and a comparison to a flat anchor in loose sand	0.21	0°	3.42
			15°	3.42
			75°	3.42
ADH-3b	To provide tests for the remaining under-ream bases and a comparison to a flat anchor in dense sand	0.55	0°	2.58
			15°	2.77
			75°	3.08
ADH-4a	To carry out control tests for a number of the plates and an under-reamed base in loose sand	0.19	36 mm	5.56
			60 mm	3.33
			60°	3.42
ADH-4b	To carry out control tests for a number of the tests plates and an under-reamed base in dense sand	0.54	36 mm	3.00
			60 mm	5.00
			60°	2.77

4 Theoretical Model for Uplift Resistance

In this section two separate theories from the literature will be presented that will be used to compare the test results to analytical methods for pile design.

4.1 Meyerhof and Adams

Meyerhof & Adams (1968) proposed a model for the uplift capacity of under-reamed piles based on the friction on a cylinder of soil above the pile base, including the weight of the soil as shown in figure 11. The initial derivation is based on the plain strain case for a strip footing, which is then modified for circular foundations using a shape factor derived from empirical correlations. The equations taken from the paper below detail the stages in determining the final solution, but leave out the components concerning cohesion, as this testing is being carried out in dry sand which is cohesionless.

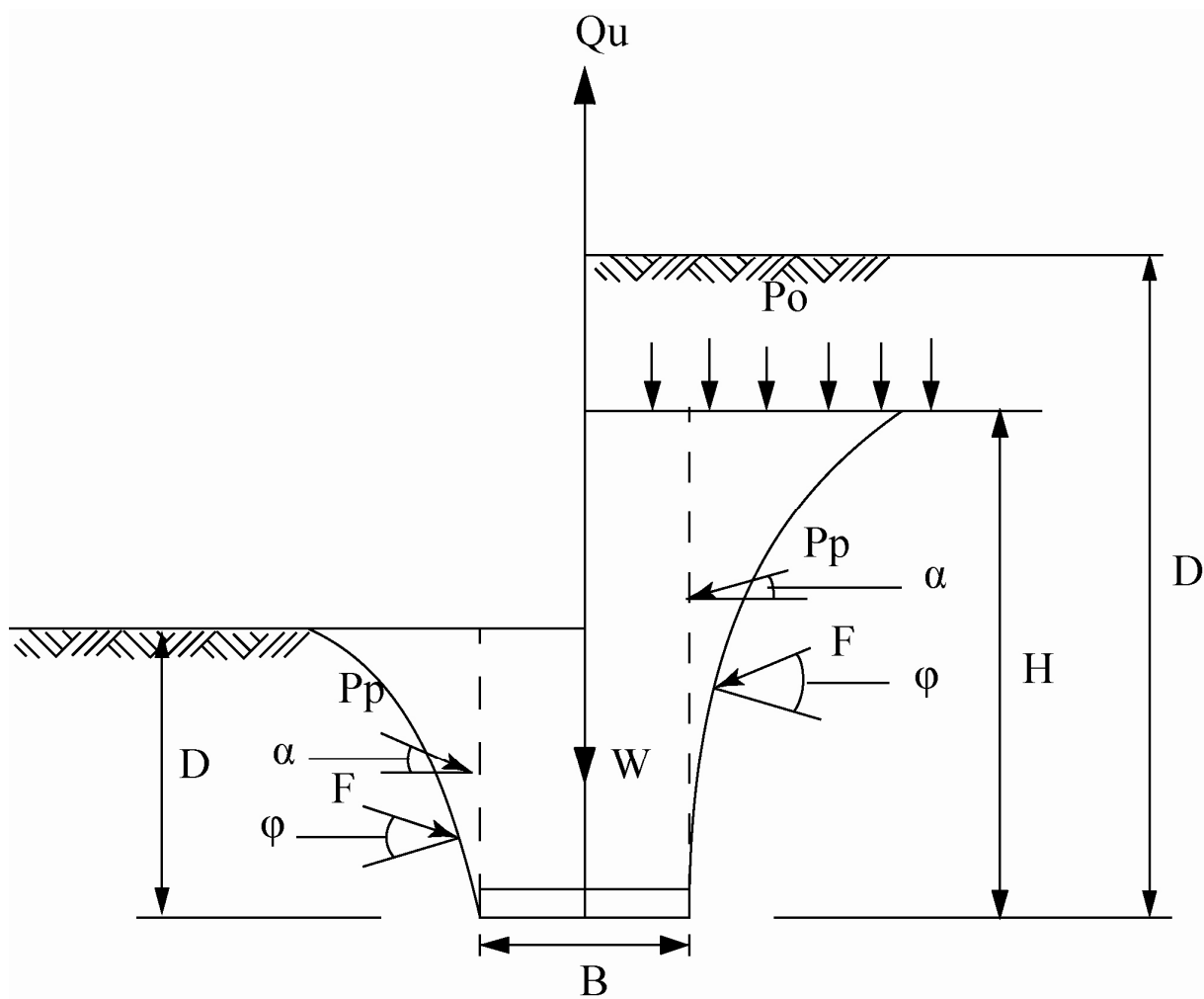


Figure 11 Geometry of an under-reamed pile foundation failure surface from (Meyerhof & Adams, 1968)

For a strip footing, at shallow depth:

$$\tau_f = \sigma \tan \phi \quad (4.1)$$

where σ is the normal stress on the failure surface and ϕ is the angle of internal friction of the soil. The ultimate load per unit length of footing therefore is:

$$Q_u = 2F \cos \beta + W \quad (4.2)$$

where W is the weight of lifted soil mass and weight of footing and β is the average inclination with vertical of force F . In the absence of a rigorous solution for the stresses on the failure surface one can assume:

$$Q_u = 2P_p \sin \delta + W \quad (4.3)$$

where P_p is the total passive earth pressure acting downwards at an average angle δ on a vertical plane through the footing edge. Expressing the normal component of P_p

$$P_p \cos \delta = \gamma \left(\frac{D^2}{2} \right) K_p \quad (4.4)$$

where K_p is the coefficient of passive earth pressure and γ is the unit weight of soil. Substituting into equation (4.3)

$$Q_u = \gamma D^2 K_p + W \quad (4.5)$$

where $K_{pv} = K_p \tan \delta$. Trial calculations have shown that δ is approximately $2\phi/3$. For K_p values on a curved failure surface, the value of K_{pv} has been evaluated and shown in figure 12.

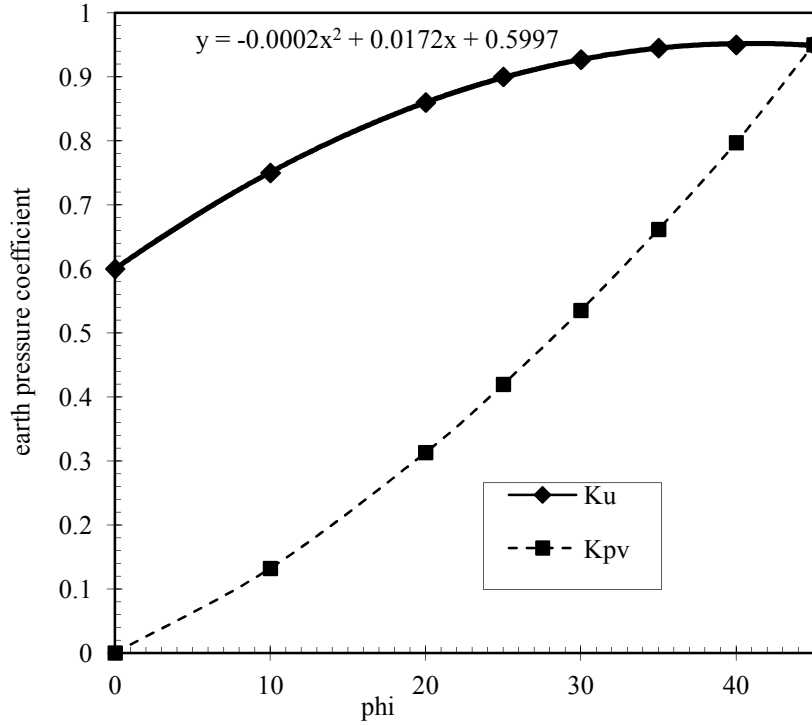


Figure 12 Coefficients of passive earth pressures (Meyerhof and Adams, (1968))

For convenience of analysis and in order to compare with the test results K_{pv} may be expressed as:

$$K_{pv} = K_u \tan \phi \quad (4.6)$$

where K_u is the nominal uplift coefficient of earth pressure on the vertical failure plane through the footing edge. A curve for K_u , along with an approximate polynomial of best fit to allow the value to be used in the analysis is shown in Figure 12. Equation (4.5) becomes

$$Q_u = \gamma D^2 K_u \tan \phi + W \quad (4.7)$$

For a footing at great depth, the extent of the local shear failure may be limited to the value H and utilising the surcharge pressure induced by the soil above the failure surface, $P_o = \gamma(D-H)$. Equation (7) then becomes

$$Q_u = \gamma(2D - H)H K_u \tan \phi + W \quad (4.8)$$

The magnitude H can only really be observed from the actual failure surface and is related to the internal angle of friction of the sand. An estimate of the ratio of H/B in terms of friction angle, determined from experimental measurements, is shown in table 3. Where the

experimental results in this project are not integer values, linear interpolation has been used between the values presented to provide a suitable estimate.

Table 3 Limiting values for H/B in terms of ϕ

Friction Angle ϕ	20°	25°	30°	35°	40°	45°	48°
Depth H/B	2.5	3	4	5	7	9	11

To modify the equation for circular footing, the shearing resistance on a cylinder through the footing edge is evaluated. For shallow depths ($D < H$), equation (4.3) and (4.7) become

$$Q_u = s\pi B P_p \sin\delta + W \quad (4.9)$$

or

$$Q_u = s \frac{\pi}{2} \gamma B D^2 K_u \tan\phi + W \quad (4.10)$$

where s is the shape factor governing the passive earth pressure on a convex cylindrical wall. Similarly for great depths ($D > H$), equation (4.10) becomes

$$Q_u = s \frac{\pi}{2} \gamma B (2D - H) H K_u \tan\phi + W \quad (4.11)$$

For small ratios of footing depth to width, D/B , theoretical results for s have been determined from experimental measurements. S can be approximated by:

$$s = 1 + m \frac{D}{B} \quad (4.12)$$

with a maximum of

$$s = 1 + m \frac{H}{B} \quad (4.13)$$

where the coefficient m has the values given in Table 4. Using these equations the uplift resistance of the under-reamed piles can be estimated then compared to the theoretical value.

Table 4 Values of the coefficient m and maximum value of s for uplift of round anchor plates

Friction Angle ϕ	20°	25°	30°	35°	40°	45°	48°
Coefficient m	0.05	0.1	0.15	0.25	0.45	0.5	0.6
Max factor s	1.12	1.30	1.60	2.25	3.45	5.50	7.60

4.2 Soil friction angle ϕ

The soil friction angle is used in the equations to determine the uplift capacity of the pile. Part of the difficulty in finding the peak uplift is in determining the value of this friction angle. For loose soils undergoing shear deformations, the behaviour tends to be contractile, with the angle of friction between the soils approaching the critical value, ϕ_{crit} at which no volumetric strains occur. However, for dense soil, the initial behaviour is dilatant, with a peak angle of friction ϕ_{max} that needs to be overcome before steady state at ϕ_{crit} is reached.

$$\phi_{max} > \phi_{crit}$$

Bolton (1986) proposed a method for taking into account the dilatancy of the soil in calculating ϕ_{max} , using known properties of the soil. Firstly, the relative density I_R is defined as:

$$I_R = I_D I_C - 1 \quad (4.14)$$

Where I_D is the relative density and I_C is the relative crushability.

$$I_D = \frac{e_{max} - e_{min}}{e_{max} - e} \quad (4.15)$$

$$I_C = \ln \frac{\sigma_c}{p'} \quad (4.16)$$

The aggregate crushing stress for a quartz sand can be taken to be 20,000 kPa. Bolton's peak friction correlation states that the maximum difference between maximum and critical friction angles is

$$(\phi_{max} - \phi_{min}) = 3^\circ I_R \quad (4.17)$$

in axysymmetric conditions. We can clearly see that the peak angle of friction depends on the relative crushability that depends on the confining pressures. In these small scale tests the confining pressure is very small, and therefore the observed increase in friction angle will be predicted to be very large by Bolton's stress dilatancy method. However, as stated previously, Hanna, *et al.* (2007) used a method proposed by Davis (1968), Drescher & Detourney (1993) and Michalowski (1997) to find a modified mobilised friction angle for low confining pressures. This states that

$$\tan \phi_{mod} = \frac{\cos \psi \sin \phi_{max}}{1 - \sin \psi \sin \phi_{max}} \quad (4.18)$$

Where ϕ_{mod} is the modified angle of friction, ϕ_{max} is the maximum angle of friction and ψ is the angle of dilation. In order to predict the angle of dilation Schanz & Vermeer (1996) proposed the following equation:

$$\sin\psi = \frac{I_R}{6.7 + I_R} \quad (4.19)$$

Combining the above equations, and using the formula presented by Meyerhof, an estimate of the uplift capacity of the under reamed pile in dense sand can be achieved.

4.3 Displacement-Load predictions

Hanna *et al.* (2007) proposed a method for predicting the displacement-load relationship for an anchor plate, taking into account the embedment to base diameter ratio of the pile and the friction angle of the sand. They found that experimental pullout load-displacement curves could be well represented by a hyperbolic relationship. The following equation was developed:

$$\frac{\Delta}{Q} = a_1\delta + b_1 \quad (4.20)$$

Where Q is the applied pullout load on the anchor (N), δ the corresponding anchor displacement (mm), a_1 is the slope of the load-displacement of the anchor ($a_1 = 1/Q_u$) and b_1 is the intercept of the straight line with the (Y) coordinate axis. $B_1 = 1/E_i$ where E_i is the initial stiffness, which depends on ϕ and (H/B) as stated previously. This can therefore be written as

$$Q = Q_u \cdot \frac{\Delta}{\Delta + K} \quad (4.21)$$

where $K = Q_u/E_i$, a constant expressed in mm. Using experimental results, value for K can finally be presented as:

$$K = 0.018Q_u \left(\frac{H}{B}\right)^{-m} \quad (4.22)$$

where

$$m = 1.3\tan\phi + 0.93 \quad (4.23)$$

Combining these equations, a plot of the expected load-displacement performance of an anchored pile can be produced. As with the prediction of the maximum uplift capacity, in this case the modified angle of internal fracture presented above should be applied.

5 Performance of Anchor Plates

In this section the results observed in the experiments using flat plates in both loose and dense sand are presented. The normalised load-displacement curves are plotted to show the comparison between the different plate sizes. The dimensionless breakout capacity is then defined and the experimental values are compared to those calculated using the method from Meyerhof & Adams (1968) .

5.1 Observed Load-displacement curves

5.1.1 Comparison between plate sizes

In the first test, ADH-1a and ADH-1b, as detailed previously in table 2, three plates of different diameter were tested in sand under loose and dense condition. Figure 13 shows the plot of uplift resistance Q against displacement (mm) for the three piles. Clearly, under the same relative density and embedment, the uplift resistance of the pile increases with the size of the base plate. It is also interesting to note that all the piles reach their peak loads at around 8 mm. After the peak load has been reached and maintained for a short while there is a sudden drop in strength that is not recovered.

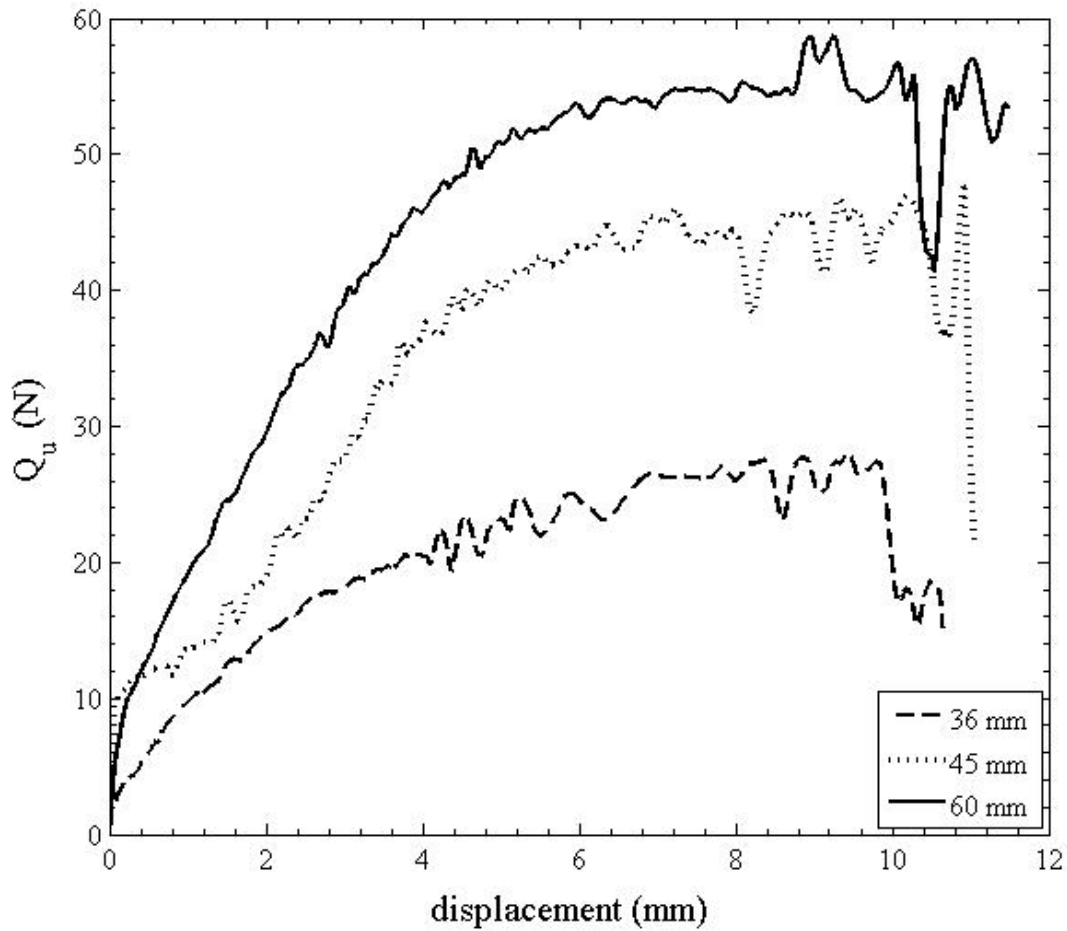


Figure 13 Load-displacement curves for anchor plates with $B=36, 45$ and 60 mm at 205 mm embedment and $I_D = 0.166$

The plot for the 45 mm base plate has a different initial shape, however this is more likely due to issues with the equipment and problems in setting up the experiment. As mentioned previously, any moments on the load cell influence the results so potentially the pile was not initially straight to the load cell.

The 60 mm base plate has an initially stiffer response than the smaller pile. Figure 14 shows a plot of the non-dimensionalised load N against the non-dimensionalised displacement A/B .

$$N = Q/YAD$$

where D is the embedment and A is the area of the pile anchor base. At peak uplift resistance, when $Q = Q_u$, the dimensionless breakout factor N_{qu} is defined as:

$$N_{qu} = Q_u/YAD$$

For the loose sand we can see that the initial sections for each curve are very similar. Ignoring the 45 mm plate, from the other two lines it would appear that the initial N against Δ/B plot for sands of the same density at the same embedment is almost identical. The 60 mm fails at a lower value of N_{qu} than the smaller piles, and also at a lower value of Δ/B .

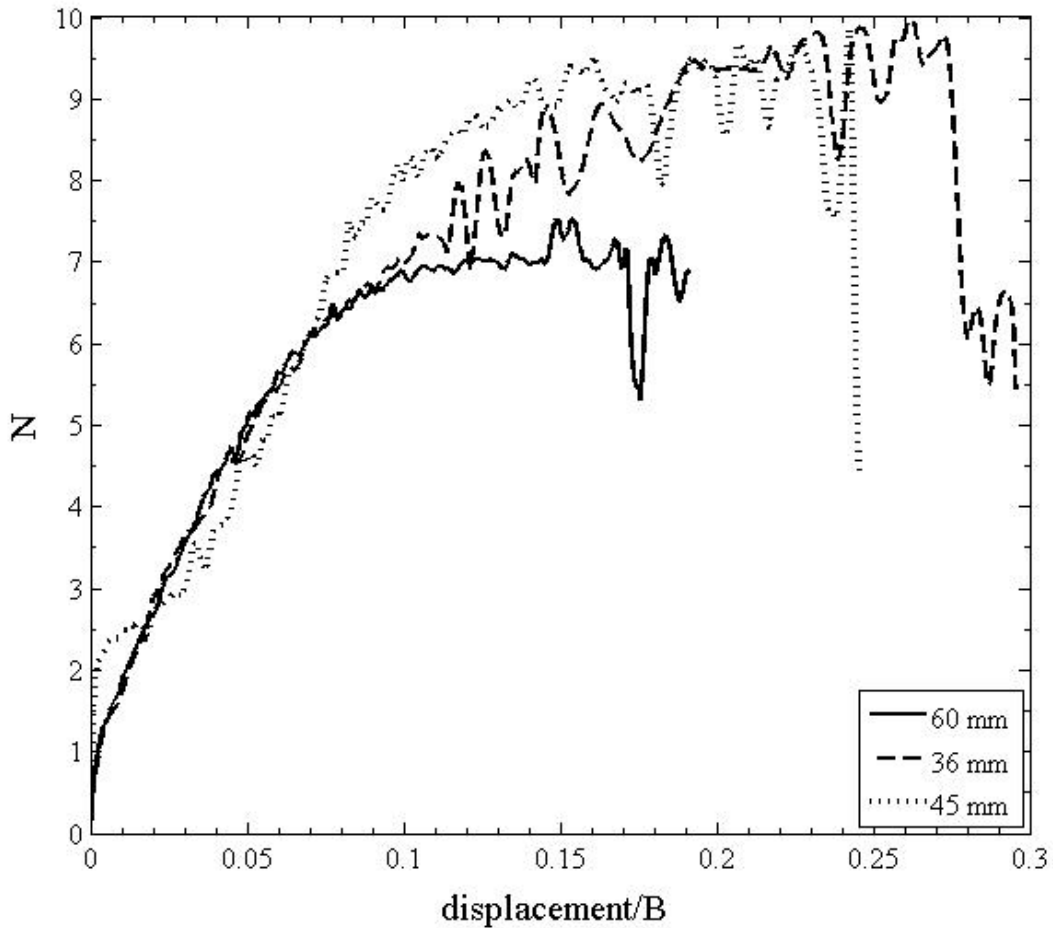


Figure 14 Non dimensionalised load-displacement plot for pile anchors with $B=36, 45$ and 60 mm and $I_D=0.166$

Figure 15 shows the non-dimensionalised plot for the piles in dense sand. Due to the method of setting up the experiment, the embedment depths for the three piles in dense sand vary by approximately 30 mm, but this should be taken into account in the non-dimensional plot. From figure 13, the unusual initial performance of the 60 mm plate can be noted. This, as with the slightly strange response observed for the 45 mm plate in loose sand, is likely due to problems with the equipment, rather than the pile. For the other tests of the flat 60 mm anchor plates performed at similar relative densities, the initial rise is similar to the other lines in

figure 13. The 36 mm and 45 mm once again show a very similar initial gradient on the non-dimensionalised graph. However, there is a greater spread in the value of N_{qu} reached between the three piles, compared to the results in the loose sand. The response is also less gradual, with the initial steep rise changing into an area at peak load where a constant rate of displacement occurs at a maintained load. This load drops as the overburden above the anchor plate reduces.

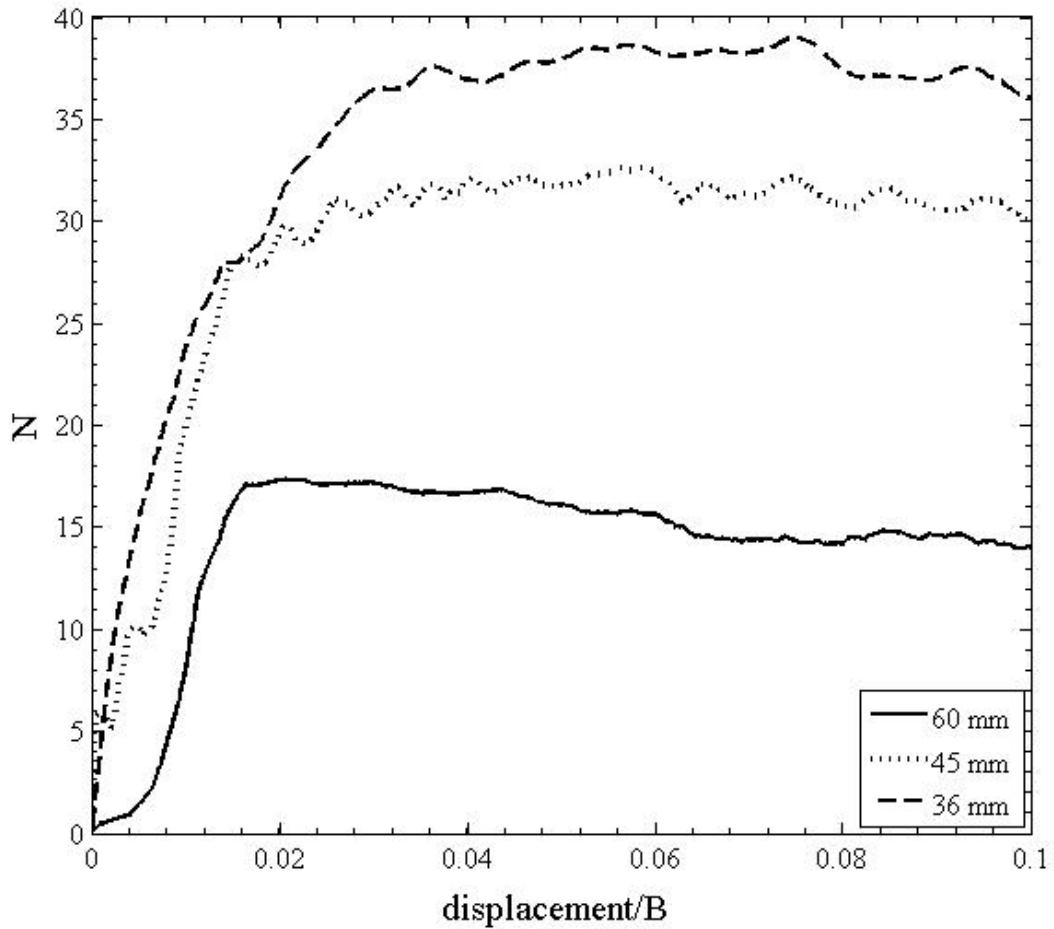


Figure 15 Non-dimensionalised load-displacement curve for anchor plates with $B=36, 45$ and 60mm and $I_D = 0.509$

5.1.2 Comparison between loose and dense soil

Figure 15 shows the plot of N against Δ/B for the 45 mm plate in both the loose sand and the dense sand. It is clear that the dense sand reaches a far higher load. Also, the maximum load is reached at a lower displacement. The loose sand has reached its maximum value after a displacement of 9 mm, whereas the dense sand has only displaced by around 2 mm at the peak value. This has a clear implication for design. For the loose sand, in order to reduce

excessive deformations in order to satisfy the serviceability limit state, the sand has to work at a very small percentage of its maximum uplift capacity. This is in contrast to the dense soil, where the soil can perform at values approaching its ultimate capacity without leading to excessive deformations.

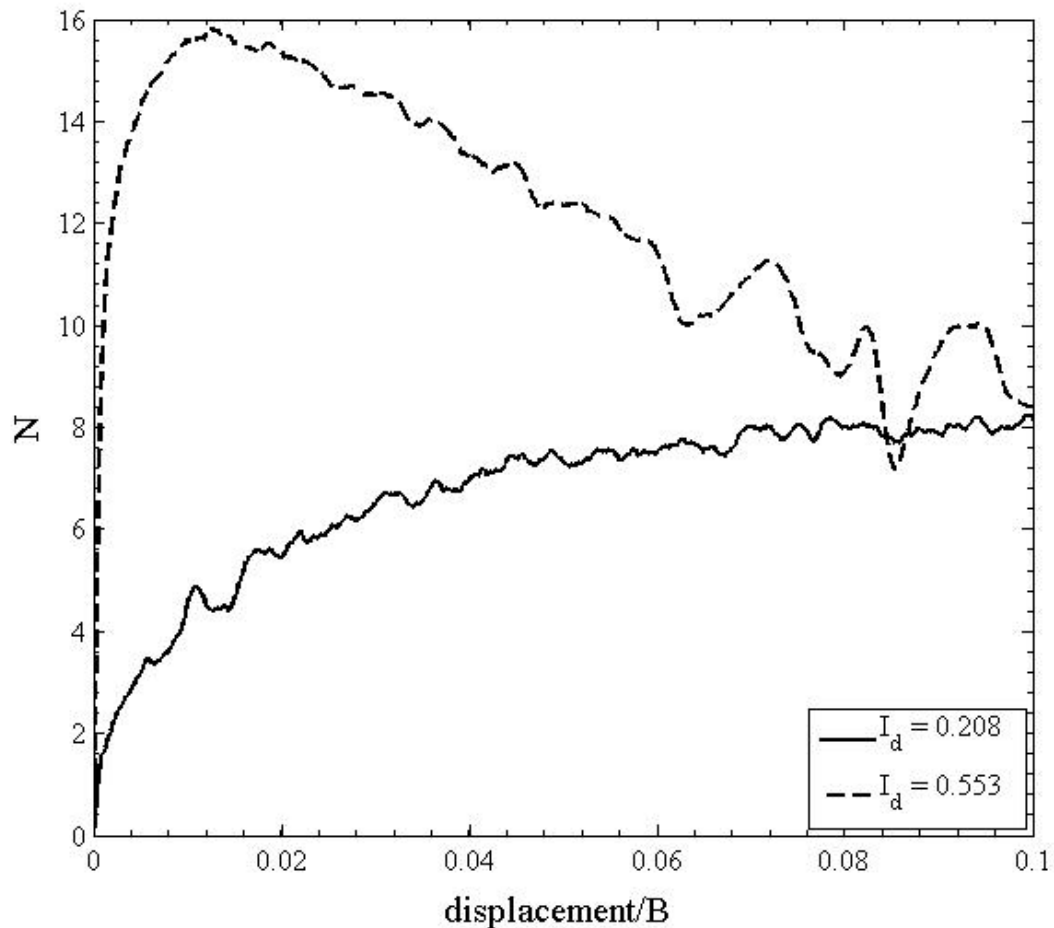


Figure 16 Non-dimensionalised load-displacement curve for a 60 mm anchor plate in loose and dense soil

The difference between the curves does not agree with that observed by Ilamparuthi, *et al.* (2002) for a comparison between loose and dense sands. The 36 mm pile tested at loose conditions is the only one that was not categorised as a shallow pile using Meyerhof & Adams (1968) method. Ilamparuthi, *et al.* (2002) observed that for piles at an embedment ratio of 3, at shallow embedment, the curves had a similar shape as seen in figure 17. There are large differences between their observations and the test results for this project. Firstly, Ilamparuthi, *et al.* (2002) find that the loose sand reaches its peak pullout at a lower

displacement than the dense sand, and secondly, that a high peak is also reached for a loose sand before a gradual drop.

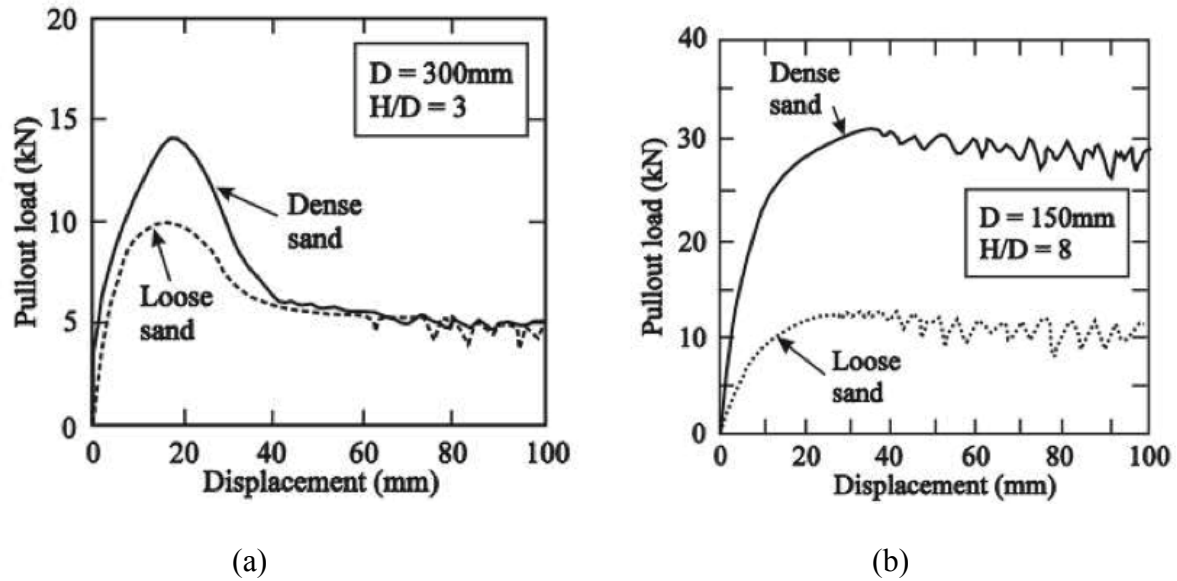


Figure 17 Ilamparuthi *et al.* (2002) observations of load and displacements for large scale tests at (a) shallow embedment and (b) deep embedment

The dense sand follows a similar shape to that observed in the tests shown in the previous figures, however including a far sharper peak. The peak is expected for the dense sand as volumetric dilation occurs as the sand is sheared to a peak friction angle, before dropping to shear at a critical state. This sort of shape has also been observed for their loose sand experiments, which was not observed in the tests presented in this report. The loose sand curves in the tests in this project followed a shape more like that seen for deep piles in Ilamparuthi *et al.* (2002) observations. However, they state in their paper that they generally agree with the Meyerhof & Adams (1968) boundary for defining deep and shallow ground anchors.

If one considers the shear stress/strain plot from a direct shear stress, the results similar to those observed in the experiments carried out in this project. At higher sand density, a peak stress is reached before the shear stress drops to a constant value, at the critical internal angle of friction of the sand. For the loose test the curve rises to the critical stress. The results from Ilamparuthi *et al.* (2002) are inconsistent with what would be expected and what has been observed for these experiments.

5.2 Comparisons with theory

5.2.1 Ultimate uplift resistance

Figures 18 and 19 show the results for flat anchors from, all the tests, plotted against the calculated values using the theory outlined in section 4. Figure 18 shows the plot for the results using the modified friction angle. The method appears to not be conservative for all the tests in loose sand at a low value of N_{qu} . There is a fairly large spread of the results. Figure 18 has lines plotted to show $\pm 20\%$ of the theoretical results, and some of the experimental results fall outside this line. At higher densities with high values of N_{qu} , ignoring the results for the test ADH-4b, as both results in dense sand lie far outside the $+20\%$ line, then the use of the modified friction angle appears to not be conservative.

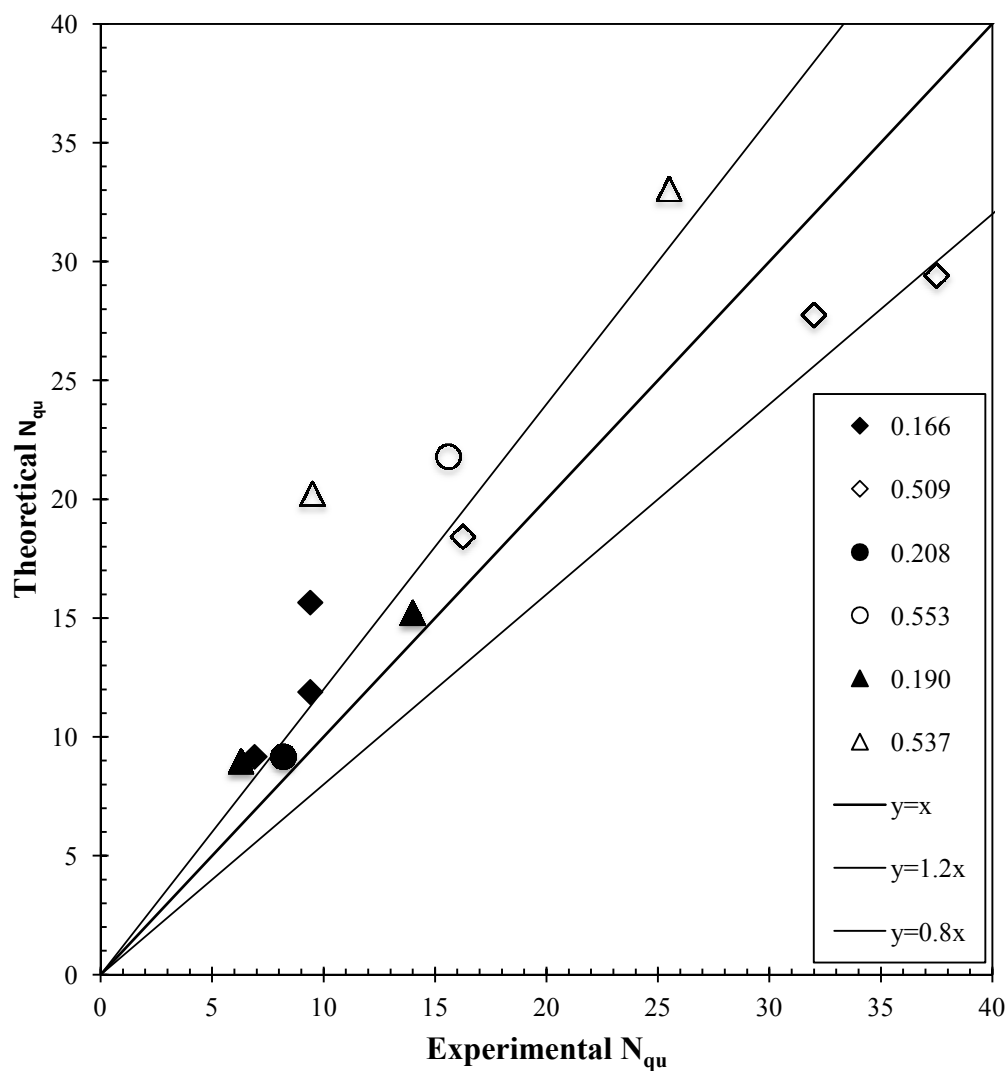


Figure 18 Comparison of theoretical and experimental N_{qu} results of flat anchor plates using the modified peak friction angle grouped by relative density

Figure 19 uses the maximum angle of friction predicted using Bolton's (1986) stress dilatancy theory for the test in dense soil. This clearly predicts far higher values than the actual results. The modified angle of friction therefore appears to be the suitable choice. Looking at the results for test ADH-2, then for higher values of N_{qu} a midway value between the modified angle and the maximum from Bolton (1986) would be most suitable.

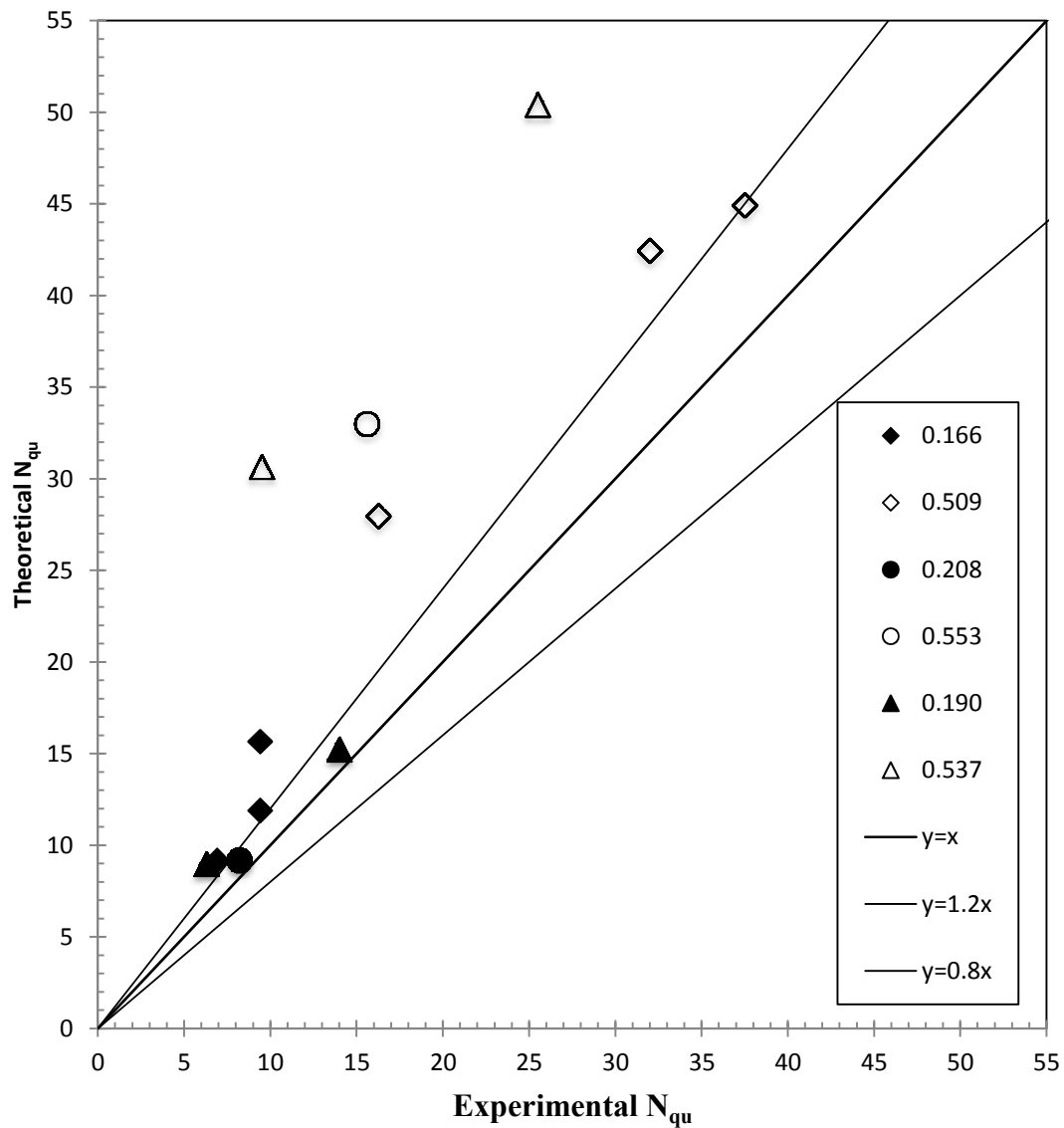


Figure 19 Comparison of theoretical and experimental results of flat anchor plates using the peak friction angle from Bolton's dilatancy principal an grouped by relative density

5.2.2 Load-displacement curve predictions

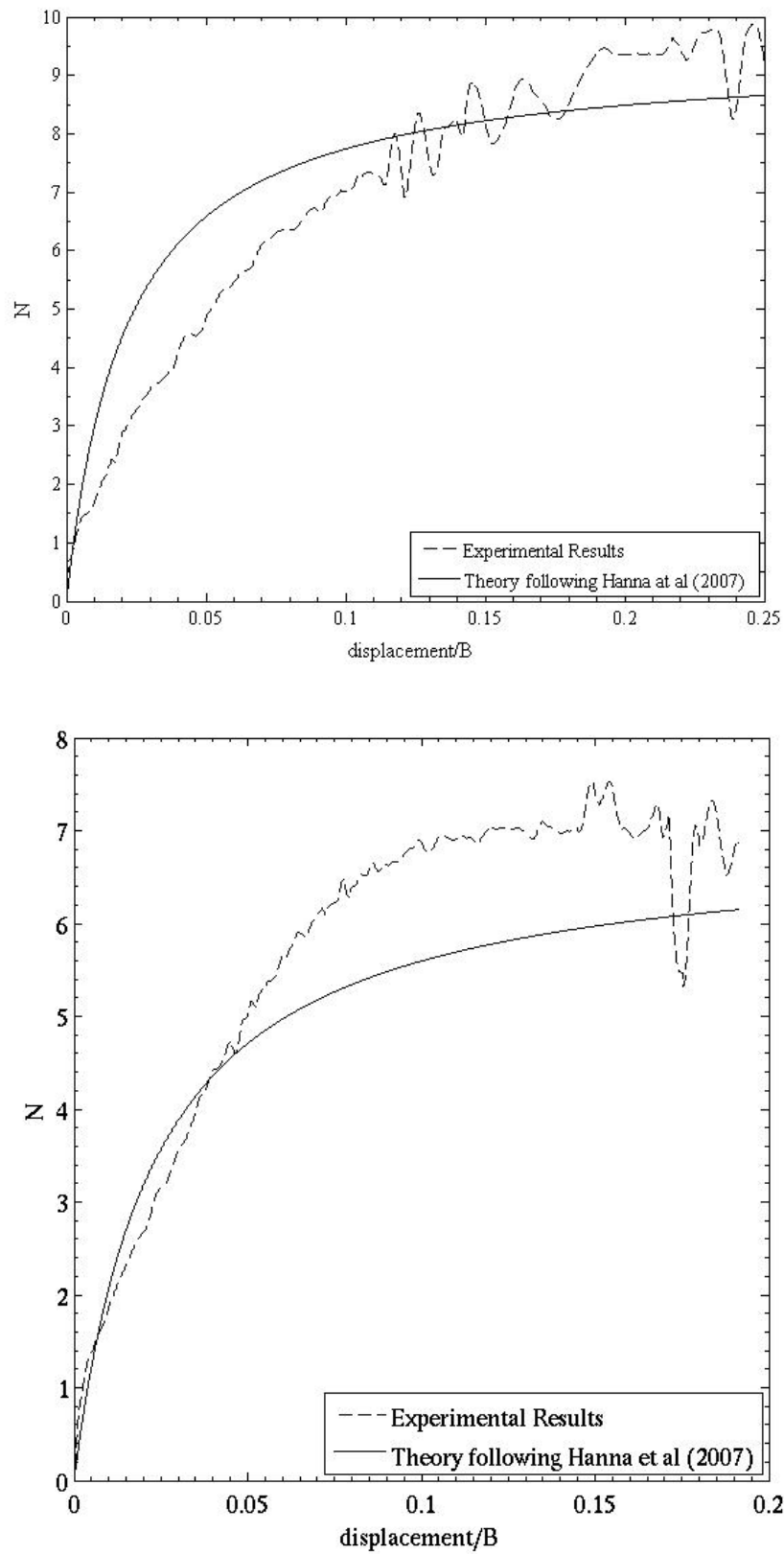


Figure 20 Comparison between theoretical and experimental results of the displacement response in at an $I_D = 0.166$ for $B = 30$ mm (top) and $B = 60$ mm (bottom)

Figure 20 shows a comparison between the test results for the 36 mm and 60 mm pile in test 1 and the displacement profile calculated using the equation from Hanna *et al.* (2007). In order to do a comparison using from (Hanna, et al., 2007) the value of Q_u observed in the actual test was used in equation (4.21), along with the modified angle of internal friction. Looking at the 36 mm test, it has a steeper initial curve then does not reach the full maximum load. This is not surprising as due to the form of the equation

$$Q = Q_u \cdot \frac{\Delta}{\Delta + K} \quad \text{and} \quad K = 0.018 Q_u \left(\frac{H}{B} \right)^{-m}$$

if K is not considerably smaller than Δ then the load will not be close to the asymptote of Q_u until a large displacement has been reached. Therefore it is not surprising that this method consistently under predicts. The equation also assumes that the loose and dense sand have the same response, with the variables adjusted for friction angle and embedment ratio. As we can see for the 36 mm plate, in the loose test the predicted curve is initially above the experimental result with a steeper gradient. However, the final value is close to that of the theoretical result at failure and overall the theory provides a fairly good match the observed results. For the 60 mm plate in loose sand there is a large difference between the final values reached as the response appears to be stiffer than predicted, but the general trend is the same as that observed for the 36 mm plate.

For the dense test, the theoretical curve is initially less steep and does not cross the experimental curve at all. The theoretical curve does not capture the actual load deflection shape of the dense sand. In terms of design, if this theoretical result were used to predict the deflections for a real building, in loose sand for a given load, the deflections would be underestimated, leading to serious consequences. In the dense sand, for a given load, smaller deflections than predicted would be observed. Clearly, the method for predicting displacements proposed by Hanna *et al.* (2007) is not suitable for the soil at this density and with a low embedment.

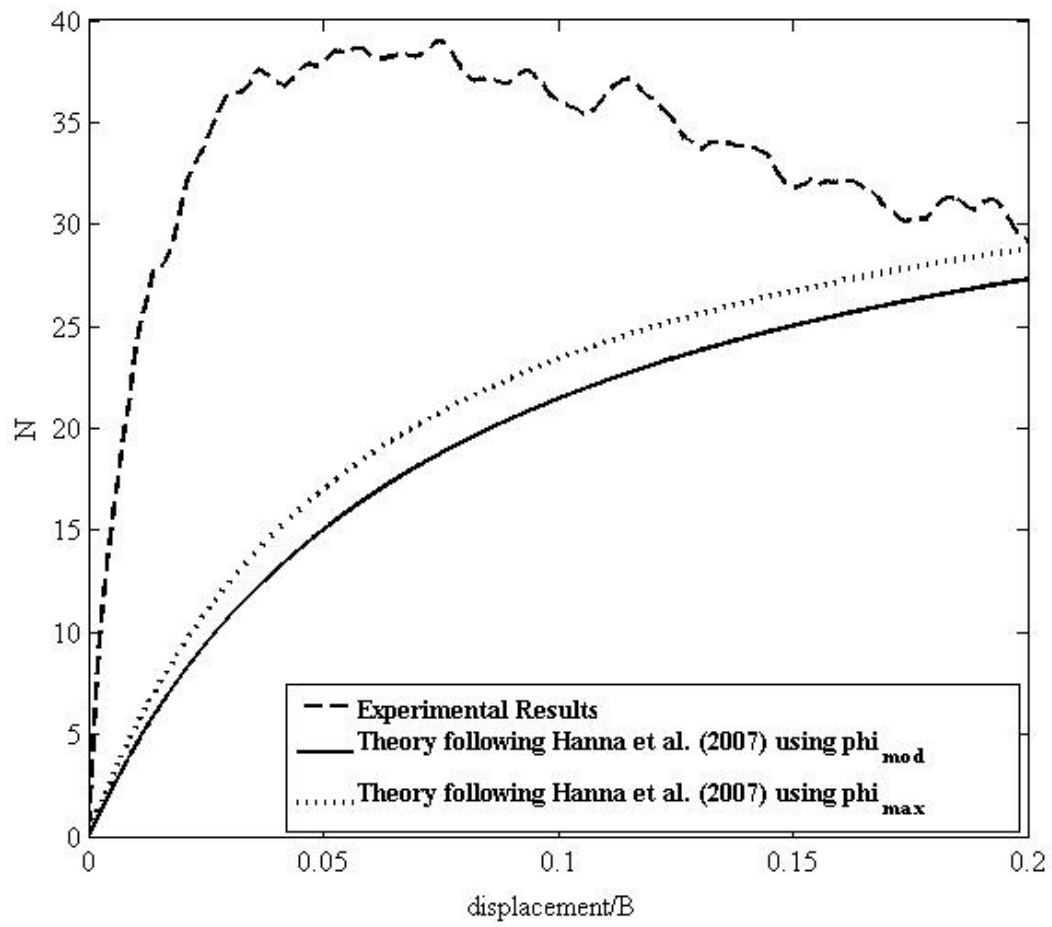


Figure 21 Comparison between theoretical and experimental results of the non-dimensionalised load-displacement response with $B = 36$ mm and $I_D = 0.509$

6 Comparison of Angled Under-ream Bases

In this section the results observed in the experiments using angled under-ream bases in both loose and dense sand are presented. The normalised load-displacement curves are plotted to show the comparison between the different under-ream angles. The results are then compared to the theory for flat plates, to see if it is still applicable at all angles of under-reamed base.

6.1 Observed Load-displacement curves

Figure 22 shows the non-dimensionalised load displacement curves for the under-reamed piles in loose sand. The two separate plots are for tests ADH-2a and ADH-3a. The highest value of N_{qu} observed was for the 45° base followed by the 30° and 60° bases test ADH-2a. The response curves for the three piles have similar shapes to that seen previously for flat anchor plates in loose sand, with a long initial rise before sudden failure. However, for the 75° under-reamed pile, a far lower load has been measured than for the flat plate and the other under-reamed piles. The shape of the curve is also quite different to that for the other bases in the loose sand. The initial gradient is far greater, before the graph levels out at a constant load as the displacement increases.

This implies that there is a completely different failure mode occurring for the 75° than for the other under-reamed shapes and therefore also different to the one observed by Ilamparuthi *et al.* (2002) in their tests. Due to the shape of the pile, it is possible that the soil does not move upwards as a wedge. Instead, some form of flow mechanism is potentially occurring. This indicates a relationship between the friction angle of the sand, the friction angle of the sand-pile interface and the uplift capacity. In the literature review it was stated that the uplift capacity was dependant on the following factors (equation (3.1)):

$$Q = f(\gamma', \sigma_c, \phi_{crit}, I_D, H, B)$$

For undreamed piles with a large angle between the horizontal and under-ream, the interface friction angle for the pile should be included. Tests on pile bases that have similar angles, but with different surface roughness would help to analyse this failure mode, as it could be observed at what relative interface-under-ream angle the new failure mode is observed.

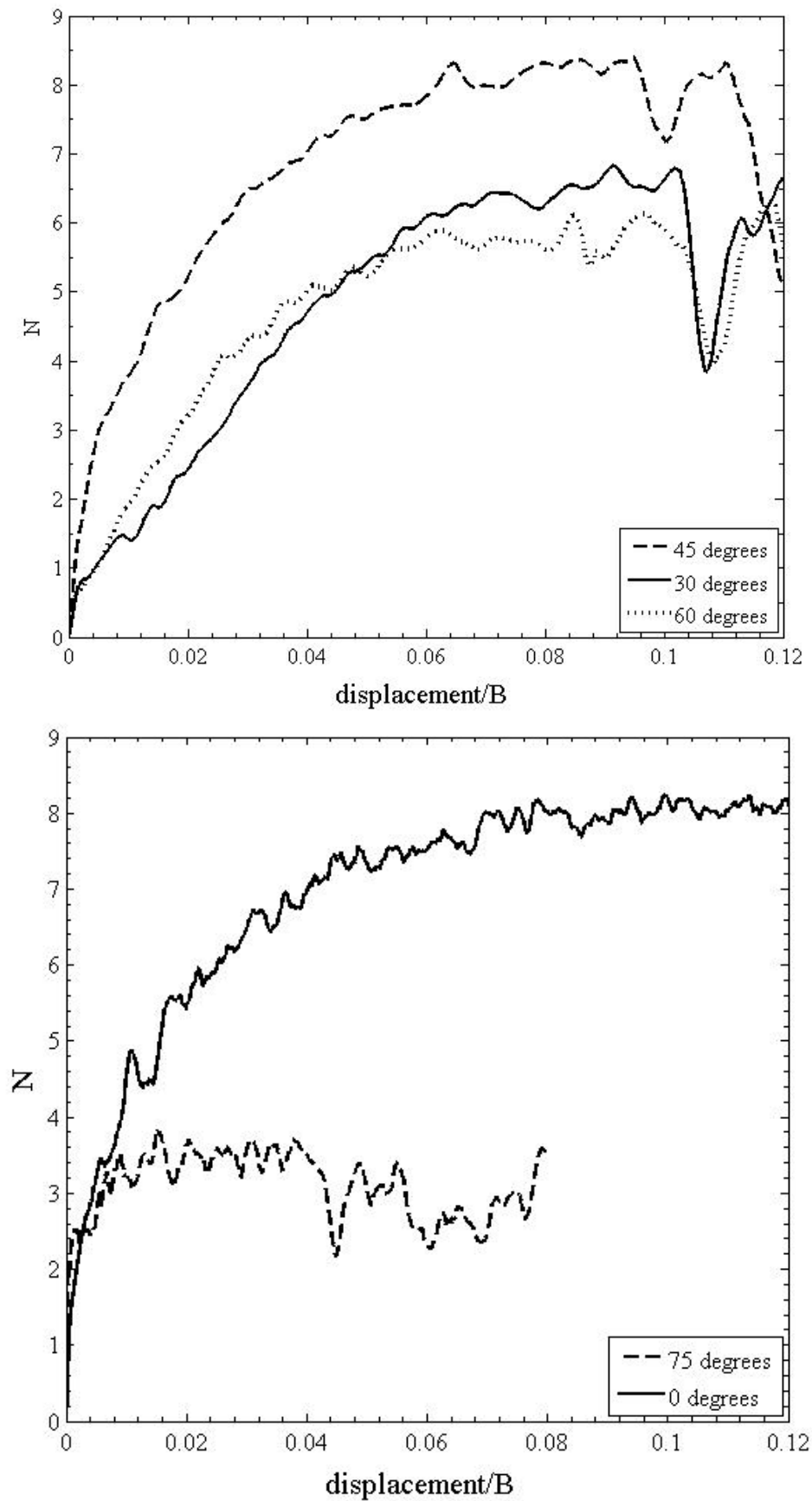


Figure 22 Non-dimensionalised load displacement curves for under-reamed piles in loose sand at $I_D = 0.139$ (top) and $I_D = 0.208$ (bottom)

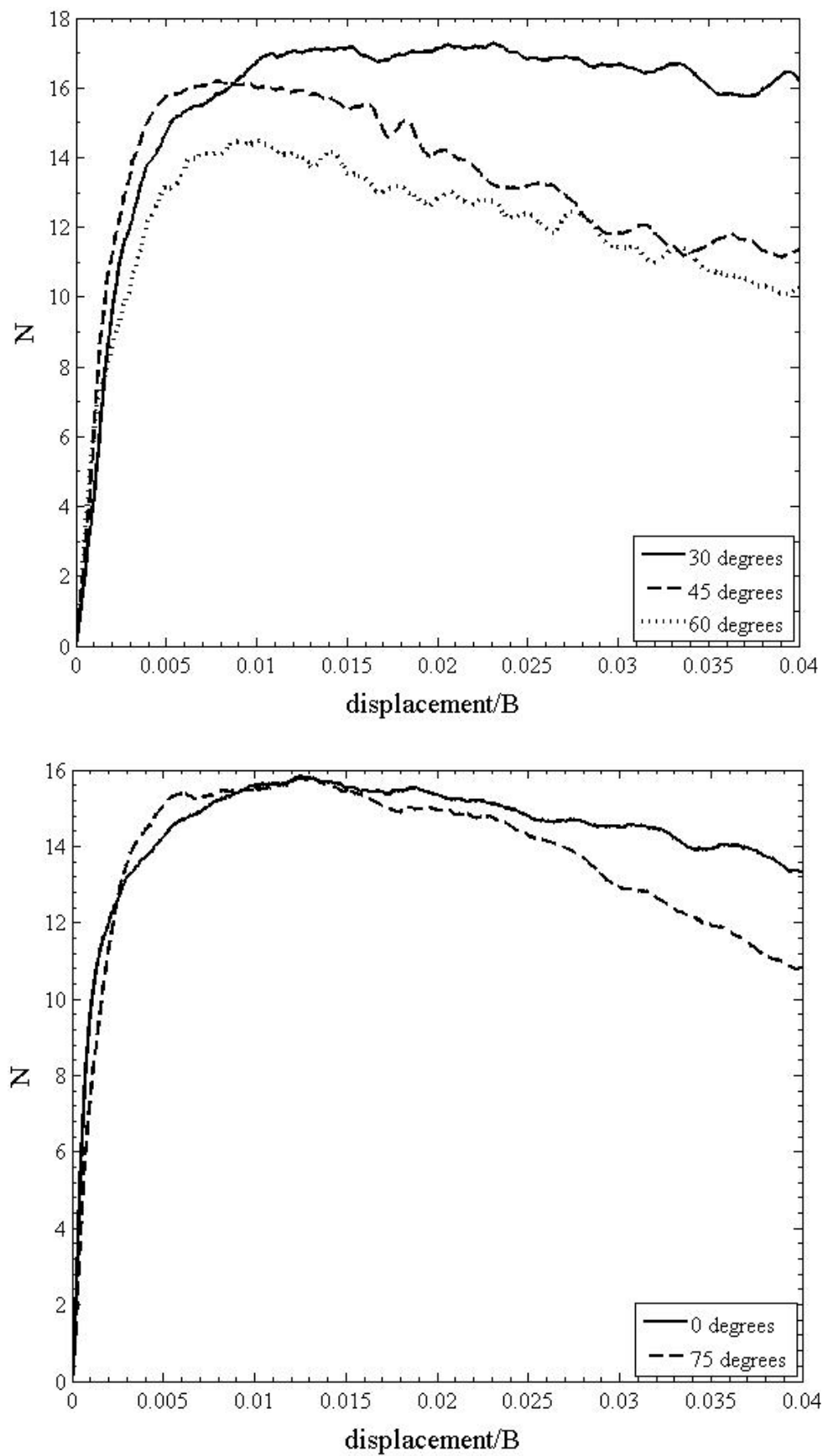


Figure 23 Non-dimensionalised load displacement curves for under-reamed piles in dense sand at $I_D = 0.484$ (top) and $I_D = 0.553$ (bottom)

Figure 23 shows the uplift resistance for the angled bases in the dense sand. Here, it is clear that the failure mode is very similar for all the piles, with a steep initial gradient before reaching the maximum load. However, the 30° under-ream load-displacement response has a different shape to the other two results in test ADH-2b once the peak load has been reached. For the other shapes the load decreases rapidly for the increase in displacement, however for the 30° base the maximum load is maintained for short while. Looking at a graph over a larger displacement, it is clear that for the greater angle of under-ream, relative to a flat plate, the peak section is far narrower and more pointed. The calculated peak friction angles are all greater than 30°, but less than 45°, so the observed difference may be related to the angle of the under-ream compared to the peak friction angle. The results for the 75° and flat plate are also shown in figure 22. This also shows that the higher angled base has a more rapid drop in strength post failure.

However, a structure should be designed to work below the peak resistance of the under-reamed pile so only the initial sections of the graph are of real interest, and these are very similar for all the piles tested.

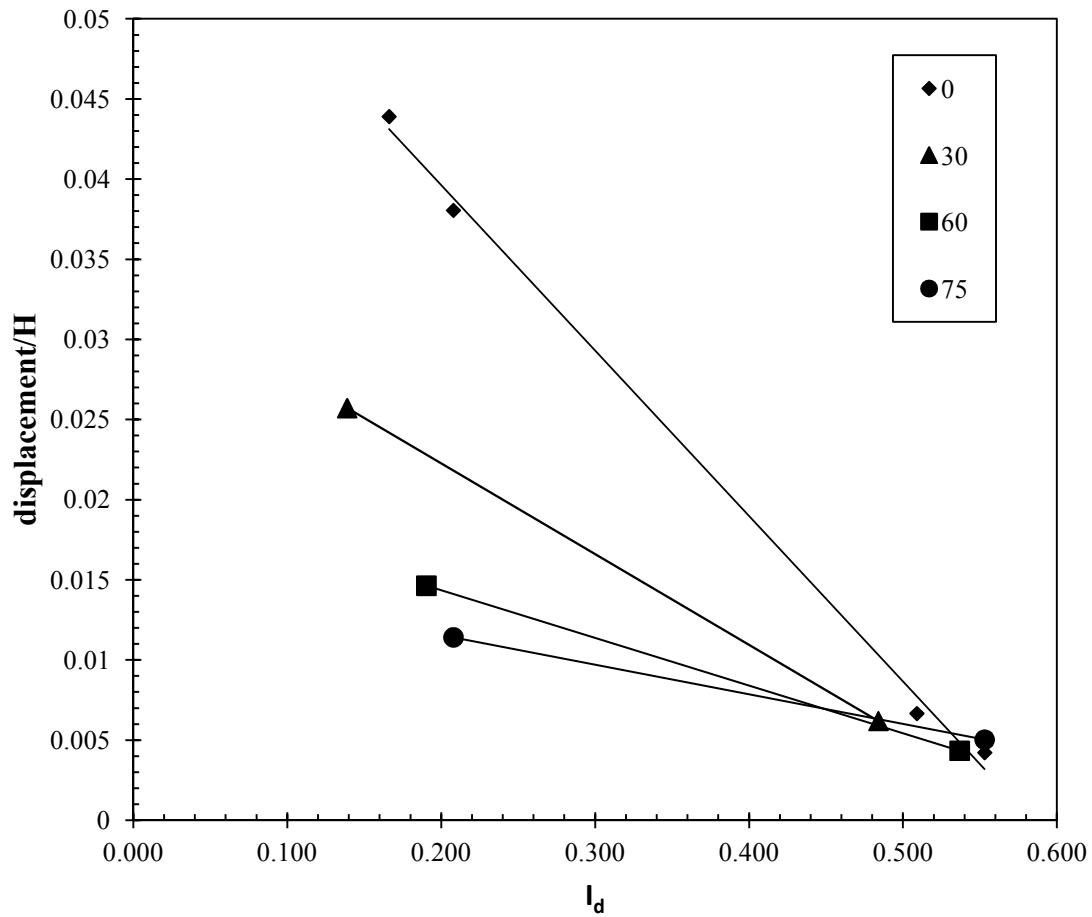


Figure 24 I_D against Δ/H for piles with a base diameter of 60 mm and various under-ream base angles

Figure 24 shows a plot of Δ/H against I_D for the flat 60 mm anchor plates and for the under-reamed pile bases. We can see that at lower relative densities, i.e. in loose sand, the non-dimensionalised displacement of the piles decreases for an increase in pile base angle. Therefore, in loose sands, the smaller the angle of the under-ream, the lower the displacement to failure of the pile for a given embedment. At the dense sand end of the spectrum, it is less clear, as the displacements are so small more tests would be needed to observe a clear difference, if any, between the piles.

6.2 Comparison with theory for a flat plate

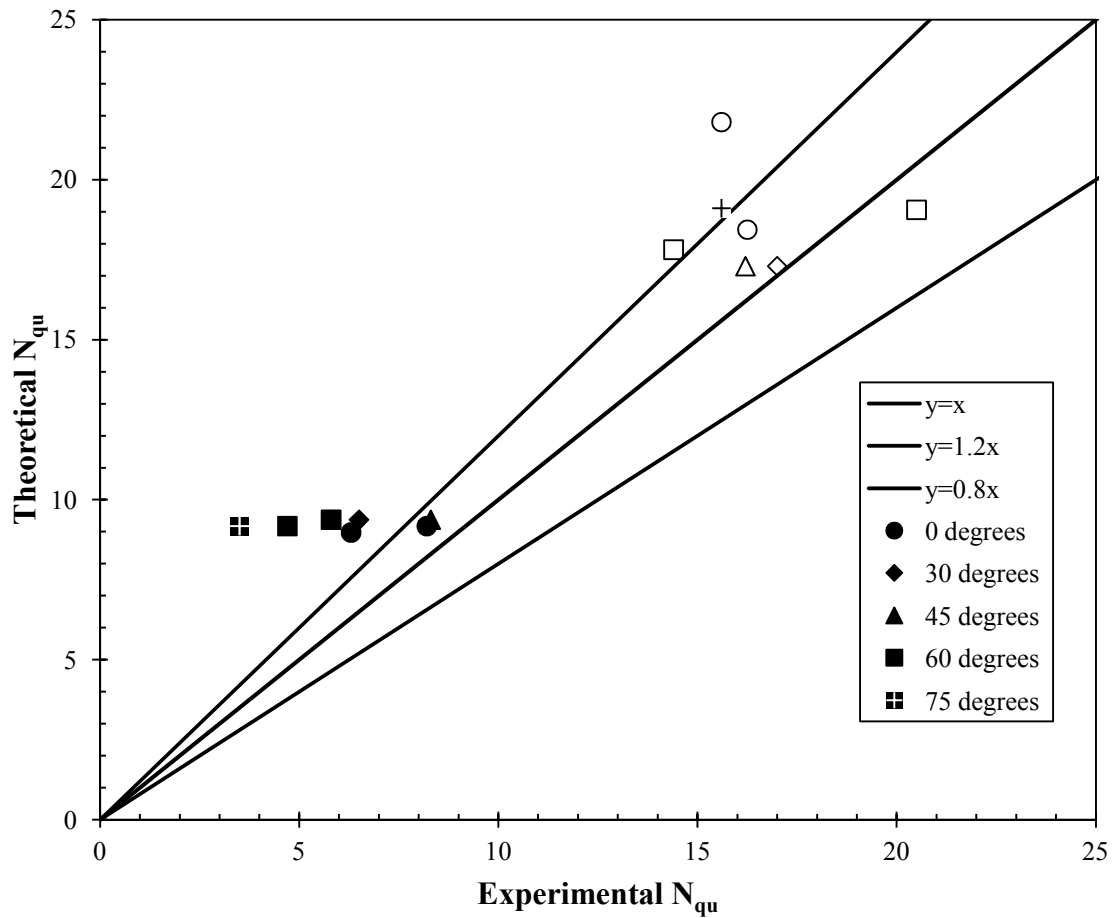


Figure 25 Comparison of theoretical and experimental values of N_{qu} for under-reamed piles in loose sand (solid data points) and dense sand (outline data points)

Figure 25 shows a comparison between the observed uplift resistances and those calculated for a flat plate using the equations outlined in section 4, including the modified friction angle. For dense sands, there is a scatter of results, with no pattern emerging for pile shape. However, results for the angled under-reams agree well with the theoretical predictions, with four of seven results falling within 20% of the predicted values.

For the loose sand, there is a clear straight horizontal line. The result recorded for the 45° degree under-ream was closest to the predicted value. However, from there, with increasing under-ream angle, the lower the observed experimental result compared to the theoretical predictions for a flat plate. The largest angle, 75°, is furthest from the predicted value. This is related to the fact that as mentioned previously regarding the load-displacement curves, the 75° under-ream failed by a different mechanism than that used to predict the uplift capacity for a flat plate.

6.3 Post failure behaviour

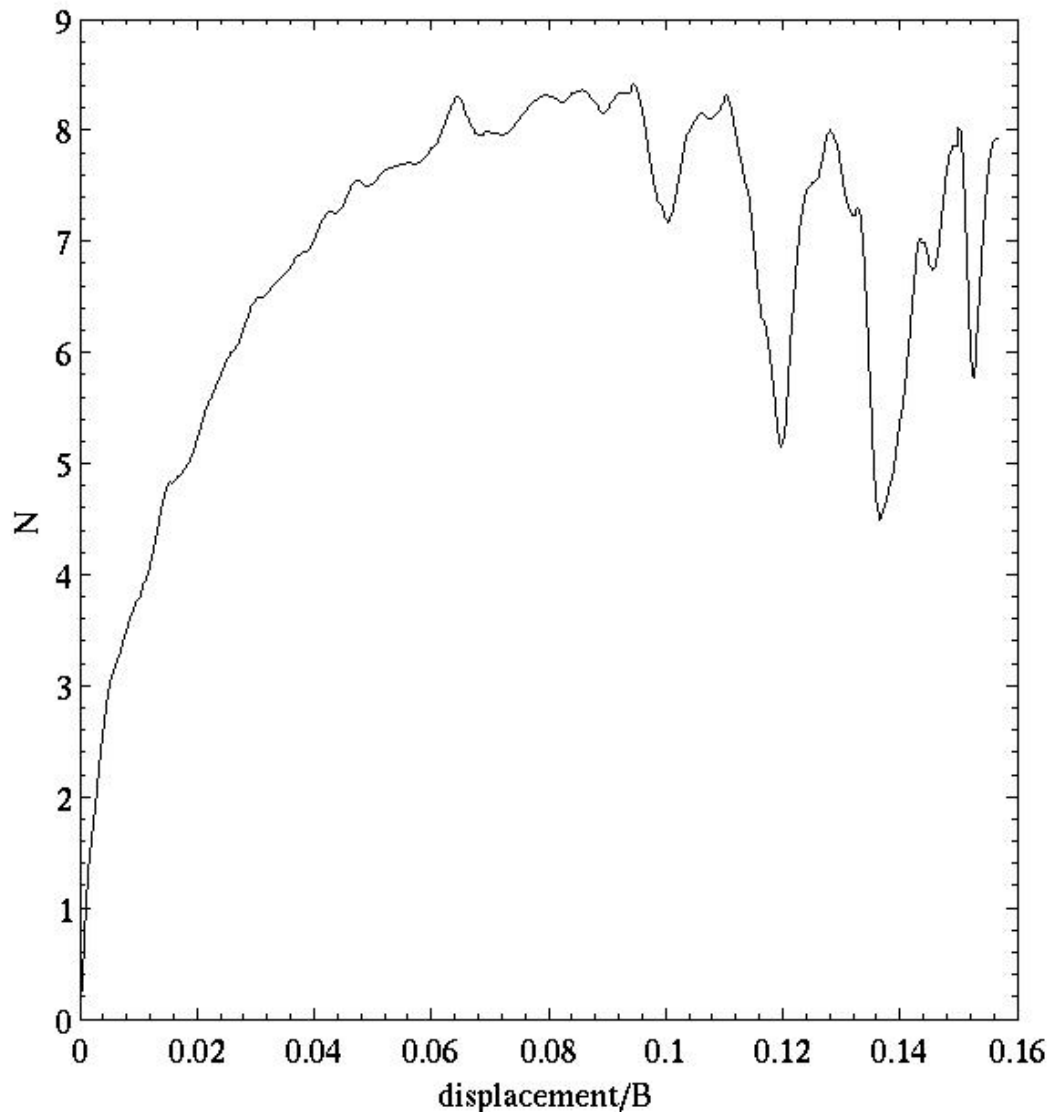


Figure 26 Graph highlighting the post buckling behaviour for the under-ream base angle of 30° and $I_D = 0.205$

Figure 26 shows the post failure buckling behaviour for the 30° undreamed pile. Once the peak has been reached, there is a sudden drop in strength, which is recovered very quickly, before dropping again. These oscillations continue on as the pile displacements increase. The reason for this is due to arching in the soil. As the peak strength is reached, the soil arch is broken. As soon as this has been allowed to occur, the peak strength of the sand is fully recovered, until the same process occurs again. The same post peak strength oscillations were also observed by Ilamparuthi *et al.* (2002) in their experiments.

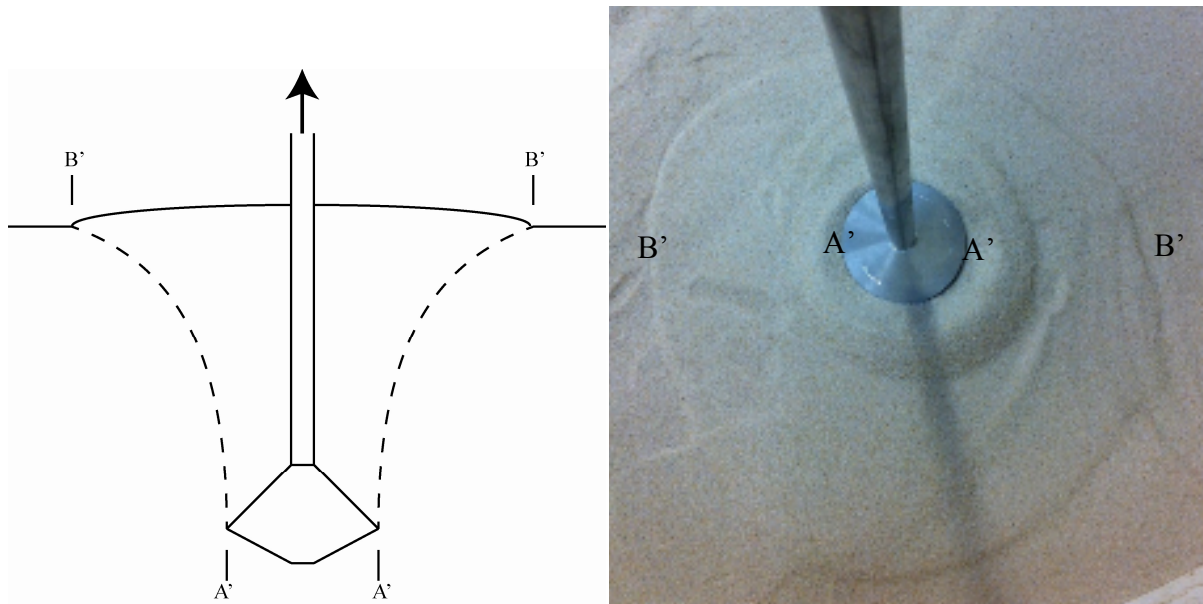


Figure 27 Observation of heave at the surface shown diagrammatically in (a) and the actual surface observed in (b) for a 45° under-ream

Figure 27 shows the pile when it has been fully pulled from the ground. Visible in the picture is a large diameter circle in the sand around the pile. The pile in this area has been raised, heaving above the surface of the rest of the sand. The illustration in figure 27 indicates how this relates to the failure surface in the sand. This surface heave has also been documented by Ilamparuthi *et al.* (2002) when they carried out half space studies using plate anchors, observing the movement of the sand layers during a test. Clearly, the angled under-ream shapes also create a similar failure mechanisms in dense sand to those observed for a flat plate.

7 Conclusions and Recommendations for Further Work

7.1 Performance of anchor plates

The simple conclusions are that increasing the plate diameter leads to an increase in uplift resistance. Also, an increase in sand density leads to an increase in uplift resistance. However, a different load-displacement curve shape was observed for the loose and dense sand. Dense sands have a high initial stiffness, but drop slowly once the peak capacity has been reached. For loose sand there is a gradual increase in displacement up to failure.

The theoretical predictions proposed by Meyerhof & Adams (1968) in the dense sand were greater than the observed results, so its use in design would not be conservative. At high relative density, and using Bolton's stress dilatancy theory to determine the peak friction angle the theoretical results were also found to be far greater than those observed. Using the modified friction angle at high density achieved agreement to within 20% between the theoretical and experimental results.

The hyperbolic equation proposed by Hanna *et al.* (2007) for predicting settlements was found to not agree well with results for either loose or dense sand. The shape of the hyperbola is similar to the results in loose sand, however it underestimates initial displacements, then does not reach the maximum load at the expected displacement, however, the general shape is acceptable and fairly similar. For the dense sand, the shape was not similar, displacements are over-predicted greatly, with the hyperbolic curve not rising steeply enough. This method for predicting displacements is not suitable for the dense sand. Although it does not match the results, if it were used to design a pile of this size then the actual displacements would be smaller than predicted so it would err on the safe side.

7.2 Comparison of Angled Under-reamed Pile Bases

Tests were carried out using the under-reamed pile shapes in loose and dense sand. In the loose sand, a similar failure curve was observed to that of a flat plate for all angles, except for the 75° angled under-ream. This displayed a far lower load and displacement capacity. This implies that a different failure mode is occurring and therefore in practice under-reamed piles should avoid large side angles.

For dense sand, a similar curve for all foundation angles was observed compared to the curve for a flat plate. The peak became narrower for greater base angles. The 30° pile could sustain

the maximum load for a greater displacement than the other under-ream shapes, exhibiting a similar response than that for a flat plate. It was also the only pile with a base angle less than the internal angle of friction of the sand.

In dense sands, for the larger values of I_d , the relative displacement Δ/H at which failure occurs was very similar for all angles. However, as the density of the sand decreases, a larger displacement can be tolerated the smaller the base angles.

The comparison of the anchor plates in sand against the theory for the uplift capacity of a flat plate only showed that the theory continued to over predict the ultimate capacity of the under-reamed piles as it did for the flat plates. The higher the angle above 45° , the lower the experimental result compared to the theoretical predicted value.

In the dense sand, the predictions for a flat plate were found to agree fairly well with the observed results for the under-ream pile shapes.

Finally, a few terms should be added to the terms that form the function for the ultimate uplift capacity for the under-reamed pile:

$$Q = f(\gamma', \sigma_c, \phi_{crit}, I_D, H, B, \delta, \theta) \quad (7.1)$$

where δ is the interface friction angle between the sand and the pile and θ is the angle of the under-reamed base side with respect to the horizontal. This is clearly a very in depth problem, where changing one parameter has an affect on the others. Therefore, the more tests that are performed, changing one variable at a time, the better the understanding of the uplift capacity of under-reamed piles will become.

7.3 Future Work

7.3.1 Small scale modelling

It is clear from the number of tests that have been performed, that a larger amount of data would contribute to the understanding of the results. Investigation into piles with a greater embedment ratio would lead to a better understanding for the definition of shallow and deep foundations. This has been investigated in depth previously by Meyerhof & Adams (1968) and Ilamparuthi *et al.* (2002), therefore it would be important to carry out these tests on the under-reamed pile bases with angled sides in order to see if they follow the same mechanism.

Another useful set of investigation that could be carried out at small scale and 1-g is where the transition to the failure mechanism of the 75° pile takes place, and how the surface

roughness of the pile, which has not been varied in this experiment, would affect the results. So using the same scale of experiments more tests could be carried out with:

- A greater range of base angles
- A range of under-reams of varying base diameter
- Rough and smooth piles
- Greater embedment to investigate the deep pile response

However, these would still only be comparative, due to the problems noted earlier with testing at 1-g. Due to the small confining pressures, far larger dilation angles are observed than would be observed at full scale. In order to replicate the actual ground conditions at small scale, the testing could take place in a centrifuge. There are issues with the amount of data that one is able to collect per test, however the information recorded would be of greater value as the test would be dimensionally similar to full scale piles.

Particle Imaging Velocimetry (PIV) is a technique that could be employed in connection with centrifuge testing to observe the failure surface in the sand. Ilamparuthi *et al.* (2002) have already carried out tests in layered sand to observe arching, however PIV is rather more subtle. The model would be set up in a half plane, with the section through the pile being a clear face. High definition photos of the soil are taken during testing, and the individual sand grain movement relative to marker points is observed. This technique has already been employed in numerous areas such as displacements around piles, driven punches, model tunnels and pipelines within the university, so this would be a perfect application to observe the failure surface and help in the understanding of the failure mechanism.

7.3.2 Large scale modelling

Small scale testing is useful; however nothing is as good as full scale testing or centrifuge modelling at high g values. Data already exists for plate anchors and mushroom piles for transmission tower foundations. However, the construction of test piles in known soil conditions would help to get real information regarding the response of under-reamed piles.

This project has only considered the uplift capacity in dry sands. These are not the only conditions to be found in nature, so testing in sand and clay would be useful to gain a comparison between different materials. When testing in saturated soil under undrained conditions more factors will need to be considered, such as the negative pore water pressures on the base of the pile and the effects this would have on the ultimate capacity.

8 References

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APPENDIX – Risk Assessment Retrospective

The risks identified before the experiments were undertaken were the use of the actuator with its electrical power supply and moving parts. Throughout all the experiments no injuries were sustained due to use of the actuator and associated equipment.

Some hazards were not identified before the experiment started, but dealt with once the situation arose:

- Dust hazard. When pouring the sand there was the potential for dust to be given off which could be inhaled and cause respiratory issues. This was recognised before pouring took place, and therefore all pouring was carried out in a sealed room and using dust extractors to avoid harm to others, and wearing a respirator helmet to avoid dust inhalation by the person carrying out the sand pouring.
- Heavy lifting. Bags of Hostun sand weighing 25kg were initially used to fill the tub before filling the hopper for pouring. In order to avoid any injury the correct heavy lifting procedures were employed. No injuries occurred due to heavy lifting.
- Transportation of test container. In order to move the tub to the shaking table, it was transported using a fork lift truck around the laboratory floor. This was done by a trained operator. No incidents occurred as a result of moving the tub.