GEOTECHNICAL CENTRIFUGE MODEL TESTING FOR PILE FOUNDATION RE-USE

by

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DECLARATION

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ABSTRACT

In recent years development is at premium in many European cities. With life cycles of 25-30 years of buildings in financial cities and about 40 years in regional centres the ground is becoming more and more congested with redundant foundations. As the underground development of services and infrastructure already confines the location of building foundations, redundant foundations only add to this problem.

The research described in this thesis, using centrifuge model testing, describes how the existing pile foundations in overconsolidated clay are likely to behave when their loading conditions are changed by unloading caused by demolition and subsequent reloading. This is done with the view to re-use the existing pile foundations for the new redevelopments. The influence of the new foundations on the existing foundations is also described. By re-using the foundations, the use of raw materials is reduced, the energy consummation for construction is reduced, the volume of soil from foundation construction is eliminated and the construction time significantly reduced, consequently reducing the whole costing of a structure.

Experimental data was obtained from series of twenty one centrifuge model tests undertaken at 60g. The geometry of the model was such that it was possible to test two sets of foundations with each test. The performance of piles in overconsolidated clay when subjected to load/unload/reload cycles and the influence of supplementary piles used to achieve the required capacity were investigated. The model tests include comparison of the behaviour of bored piles when supplemented with mini piles of different length, number and spacing (centre to centre distance between the mini piles and the existing centre pile).

An increase in capacity was observed when single piles were subjected to load cycles. It was found that this increase in capacity is dependent on the previous loading conditions of the pile. The behaviour of enhanced piles was characterised using a single pile test as datum test. The influence of these novel pile groups on the existing pile was dependent on the number, length of the mini-piles in the group and centre to centre spacing between the existing and new pile foundation.

LIST OF SYMBOLS

a	Imposed radial acceleration
d_s	Pile shaft diameter
d_b	Pile base diameter
d_c	Depth factor applied to N_c
g	Acceleration due to gravity
h	Height
h_m	Depth in model
h_p	Depth in prototype
p_b	Pile base load
r	Radius
r_o	Maximum over-stress
r_u	Maximum under-stress
q_s	Ultimate shaft capacity
q_b	Ultimate end bearing
s_c	Shape factor applied to N_c
t	Time
t_m	Time in model
t_p	Time in prototype
u	Pore pressure
z	Depth
A	Acceleration
D	Model pile diameter
E_{25}	Young’s modulus at 25% of base failure stress
H	Height of pile foundation
M_s	Flexibility factor representing pile settelemnt caused by shaft friction
N_c	Bearing capacity for surface strip foundations
P_s	Ultimate pile shaft load
P_b	Ultimate pile base load at which the displacement is infinite
R_e	Effective centrifuge radius for the model
R_t	Radius at the top of the model

S_u	Undrained shear strength
S_{ub}	Undrained strength at the foundation base
K_0	Coefficient of earth pressure at rest
N	Gravity scaling factor
α	Adhesion factor
γ_s	Unit weight of soil
γ_w	Unit weight of water
γ	Unit weight
ν'	Poisson's ratio
ρ	Density
ρs	Pile shaft settlement
ρ_b	Pile base settlement
ϕ'	Friction angle
ω	Angular velocity (radians/second)
Γ	Specific volume on the critical state line when p'=1kPa
M	Stress ratio at critical state (q'/p')
σ'_h	Horizontal effective stress
σ'_v	Vertical effective stress
σ_{vm}	Vertical stress acting in model
σ_{vp}	Vertical stress acting in prototype

Abbreviations

CRP	Constant rate penetration test
ML	Maintained load test
PPT	Pore pressure transducer
LVDT	Linearly variable differential transformer (displacement transducer)
OCR	Overconsolidation ratio
RuFUS	Re-use of foundations for urbane site

CHAPTER 1

INTRODUCTION

With continuous development in the urban environment the ground is becoming more and more congested with redundant foundations. The underground development of services and infrastructure already restrict the locations of new building foundations and the redundant foundations only add to this problem. The research undertaken is an investigation into behaviour of bored piles in overconsolidated clay when subjected to load cycles with the view of the re-use of the existing piles for future redevelopments. If the existing piles are to be re-used, then by understanding the behaviour of pile foundations when subjected to load cycles, a decision can be made on the magnitude of the load to which the existing piles can be re-loaded.

If the capacity of the existing piles in not sufficient for the new development, their capacity will need ot be enhanced. Consequently, the research sought to explore the possibility of improving the capacity of the existing piles by placing a ring of new mini-pile foundations around the existing centre pile. This new mini-pile group was constructed around the existing pile that had previously been subjected to its working or failure load. The geometry of the group was varied, i.e. the number of the mini-piles, centre to centre distance between the existing and new pile foundation and length of these new foundations.

There are obvious advantages for redevelopment if as much as possible of the existing buildings can be reused to reduce the environmental impact, time and cost of the construction.

1.1 Background

Reuse of pile foundations is not new. In Elizabethan times it was a common practice to rebuild large structures supported by the old foundations. In recent times though, buildings and the expectations of their performance have changed as a result of the development of the industry. The acceptance of damage in structures has decreased. With construction techniques improving continuously and requirements for taller buildings increasing, thus dealing with greater loads, the existing foundations were ignored or removed and new foundations were preferred (Butcher et al., 2006).

The redevelopment of inner-city sites is at a premium in many world cities and the number of sites where construction requires a third set of deep foundations is increasing. In these urban environments, underground services and infrastructure already, to some extent, dictate the location of building foundations and by continuing to avoid the existing piles the problems is exacerbated (Chapman et al. 2001)[1]. If the foundations are not avoided, then the engineer is left with a choice of removing or re-using the existing foundations. Removal of piles is time consuming, costly (up to four or more times the cost of constructing new piles) and environmentally damaging and it seems logical that there may come a time when re-use of foundations will be the only practical and economical solution.

In 2003 the RuFUS (Reuse of pile Foundations for Urban Sites) project, funded by the European Union, was undertaken to provide ways to overcome technical and non-technical barriers to re-use of foundations for sustainable development. The project resulted in a “best practice handbook” (Butcher et al., 2006) on the re-use of foundations that included guidance on the remediation/upgrading of existing foundations, guidance on the measurement and analysis for testing of existing foundations beneath buildings to assess durability, integrity and geometrical shape and foundations loading performance, guidance for new foundations and documentation system to future proof new foundations.

The barriers for reuse of foundations were also recognised during the RuFUS study. The owners of the present generation of city buildings do not generally possess good records of their foundations and there is a critical lack of information on the extent, location and integrity of the existing foundations. Little is known about the changes of performance of pile foundations with time thus the load capacity of the foundations will generally not be known with confidence.

The research undertaken makes use of geotechnical centrifuge modelling to examine the behaviour of piled foundations in overconsolidated clays. Potentially physical model testing is a useful method of investigating a problem. There are sufficient data in the context of the large scale testing (trials at Chattenden) and a number of case histories with which the results from model testing can be compared.

1.2 Methodology

The aims of the research are to improve understanding of the pile soil interaction during load/unload/reload cycles, to investigate the influence of time on pile load carrying capacity and study the influence of new pile foundations on existing pile foundations during the life of the structure. The following are identified as the main objectives of the research and form the basis for the discussion and conclusions:

(i) Apparatus was developed to enable investigation of pile foundation behaviour with load changes associated with redevelopment of a site.

(ii) The effect of the new foundations on the existing foundations was assessed.

(iii) The results of the centrifuge model tests have been compared with data from field tests carried out at a test site in Chattenden, Kent by Cementation Foundations Skanska as part of the Re-use of Foundations for Urban Sites (RuFUS) project.

1.3 Experimental work

1.3.1 Centrifuge modelling

A model has been developed, suitable for testing in a geotechnical centrifuge, such that it is possible to simulate load/unload/reload cycles typical of those associated with redevelopment of a site, in order to examine the behaviour of pile foundations in overconsolidated clays. For the first model test apparatus developed by Morrison (1994) was used (see Figure 1.1). This loading device applies vertical load axially by dumping the water from the bucket suspended on the arm through remote controlled solenoid valve. The foundations were subsequently un-loaded by adding water, through hydraulic slip ring, to water reservoir. As this apparatus did not load the piles in a controlled way and the time required for setting up the model was long, a new loading apparatus was developed (see Figure 1.2).

Four series of tests were conducted:

1. The effect of load/unload/reload cycles on single pile foundations were investigated (see Figure 1.3).

2. The effects of supplementary piles were investigated by subjecting a single existing pile enhanced by additional mini-piles to loading cycles (see Figure 1.4).

3. Behaviour of a novel pile group was investigated by loading a new mini-pile group only, when the group was constructed around a previously loaded single pile (see Figure 1.5).

4. Loading the previously loaded single pile and mini-pile group together (old and new foundations), see Figure 1.6.

A total of twenty one tests were undertaken testing two different foundations or foundations groups with each test. In general, single piles were installed as soon as the model was removed from the consolidation press. The model was allowed to come into pore water pressure equilibrium after spin-up of the centrifuge. Foundations were then loaded to either working load or failure load. For working load a factor of safety of 2 was used on the ultimate load obtained from a series of centrifuge pile load tests to failure (load at displacement of 10% of the pile diameter). When the first loading cycle had finished the centrifuge was stopped and mini-pile groups were constructed around one ore both of the single pile foundations. The tests used a variety of foundation geometries modelling the effects of the length, number and spacing of the mini-piles in the group on the existing pile. The preconsolidation pressure, model soil type and the existing pile geometry were kept constant. Measurements were made of displacement of the foundations and the load applied.

1.4 Summary of thesis

The dissertation details the approach to the research and the reasons behind it. The model response in the series of tests conducted is explained and interpreted. Chapter 2 is a literature review of deep foundation design and behaviour during the first loading. Attention is given to case studies on the influence of changes in loading condition and time in the foundation performance. The chapter continues by covering the design and behaviour of model piles and the stress history of the centrifuge soil model. A number of case histories and tests carried out during the study at Chattenden as part of the RuFUS project are described.

The design development of the centrifuge testing apparatus is described in detail and any limitations in the testing procedure are discussed. Significant time was spent on experimental work to determine the performance of the apparatus and methods that were novel in terms of centrifuge testing. This work is described in detail in Chapter 3.

The centrifuge test results are presented in chapter 4. Model testing was carried out over a period of about 14 months with modifications to the apparatus becoming necessary during this time. Foundation load behaviour is reviewed for all load cycles to which they were subjected. The chapter finishes with the assessment of increase in foundation load capacity when subjected to load/unload/reload cycles and foundation improvement with mini-pile groups.

In Chapter 5 the results of the centrifuge tests are discussed and compared with field monitoring data and experimental data from the literature review and the study carried out at Chattenden by Cementation Skanska. Patterns of foundation behaviour are identified and analysed. Shear box tests undertaken are explained and interpreted. Significant aspects of the test results are highlighted.

Chapter 6 summarises the main findings of this research project. Recommendations are made for further research that will enable a better understanding of the factors influencing foundation behaviour in load/unload/reload cycles. The implications of the results of this research on design issues related to a redevelopment without the need for additional or up-graded foundations are discussed.

CHAPTER 2

LITERATURE REVIEW

2.1 Re-use of pile foundations and novel pile groups

The presence of existing deep foundations is an increasing problem in the development of urban areas. Many modern buildings now leave behind a set of deep foundations and the foundations for new buildings have to be installed through and between this detritus. When other obstructions and resources in the ground are considered, such as tunnels or valuable archaeological deposits, on many sites the space for new foundations is scarce and diminishes with each wave of development. Geotechnical engineers have an increasingly difficult task finding locations for additional piles and providing sufficient foundation capacity (Chapman et al., 2001). Often, large transfer structures are needed to span over areas where no piles can be installed. On such sites in the future, reuse of old foundations may allow more versatile space planning for new developments. Reuse of foundations has been considered, and implemented, on several projects over the past decade where records of existing foundations have been available but the projects have been refurbishment rather than new build.

Developers need to make key strategic decisions early in any site development project in order to determine the financial viability of the proposed development. At such early stages, there are a number of risks that need to be considered. The risks particular to consideration of foundation reuse include both technical and non-technical issues that need to be overcome (Chapman et al., 2001; Fernie, 2002), such as lack of records, integrity, length and location of the existing pile foundations, as well as a lack of information on foundation performance with time and deterioration of the original foundation materials. There are also non-technical issues such as the insurance of buildings with re-used foundations and other aspects relating to liability (Fernie, 2002). It is easier to justify reuse of foundations to carry vertical loads because the capacity is dependent only on the pile dimensions, the geotechnical capacity and concrete strength and durability. These aspects are less likely to be misjudged then the parameters that control lateral capacity, such as provision of reinforcement (Fernie, 2002).

In this project the main interest is pile foundation performance with time and the effects, if there are any, of the new foundations on the existing piles when these foundations are subjected to axial load only. Generally, the existing piles in an urban environment are of the bored cast-in-place type as driven piles have seldom be used in urban situations (Chapman et al., 2001). The extent and efficiency with which those foundations can be reused depends on the amount of information that is available (Chapman et al., 2001). On this point St John (2000) states that "if a building has performed satisfactorily for many years, the foundations must be good for at least the loads that have already been applied, providing the materials are not deteriorating" suggesting that when an old structure's foundations have supported its imposed loads over many years without obvious structural distress having occurred, the old foundations have been proven sufficient to avoid breaching a serviceability limit state. Furthermore, Butcher et al (2006) state that:-

"for a foundation system that has already been tested and 'proved' by the application of the first building load, a lower factor of safety against failure may be acceptable compared to that for new foundations, provided that sufficient details are known".

This suggests that it may be reasonable to apply a greater load to an existing foundation than it was originally designed to carry.

If sufficient details are not known, Butcher et al (2006) continue by stating that:-

"if there are any uncertainties or there is lack of information of what is already in the ground, the design capacity of these foundations should be reduced".

The stress-strain behaviour of soil is typically non-linear and stiffer at small strains so designers prefer to use a small proportion of the available capacity of the existing piles in order to control the settlements. The existing foundations will usually therefore, have reserves of capacity as an enhanced factor of safety is used to account for a lack of information or understanding.

For successful reuse it is not only necessary to look at foundation performance during the first loading conditions, it is also necessary to understand how the old foundations are likely to behave when their loading condition has changed, especially in comparison to any additional foundations which may be required (St John, 2000).

There is increasing evidence that the ultimate capacity of piles increases with time. Wardle et al. (1992) carried out tests at a London Clay site on four instrumented 170mm diameter piles over a period of three years to investigate the effect of elapsed time and maintained load on the ultimate bearing capacity (see Figure 2.1). The piles tested were two driven piles, one jacked pile and one bored pile. An increase in the bored pile capacity of 47% was observed between the period of two months and three years as shown in the Figure 2.2. Powell et al. (2003) also observed changes in the vertical load capacity of piles with time. The data presented was on a number of steel driven and jacked piles in clay (see Figure 2.3). All piles showed increase in capacity with time, provided they were not retested immediately. Figures 2.4 and 2.5 show the load history for “pile D” tested at Canons Park and “pile A” tested at Cowden. “Pile D” was load tested three times initially, after 108 days, 496 days, 1130 days and 6200 days. No increase in capacity was observed during the first three load tests, as “pile D” was retested immediately. In the following load cycles, pile capacity ranges from 190kN to 290kN, an increase of 25% to 90% compared to the initial load tests. “Pile A” was load tested in compression after one month, 13 months and 25 years. After 25 years an increase in capacity of around 30% was observed. “Pile A” was also load tested in tension. Cooke et al. (1979) also observed an increase of the order of 60% in shaft resistance of jacked steel-tube piles in London Clay over a two to three year period. Powell et al. (2006) looked at the changes in capacity with time of bored pile foundations in heavily overconsolidated clay (London Clay). The piles were tested at a test site (see Figure 2.6) at Lodge Hill Camp, Chattenden, Kent UK. Some basic data found from characterisation of the site is shown in Figure 2.7. In this study a total of 12 piles with the same geometry were tested and the behaviour of two “virgin” piles and previously tested piles was analysed. The virgin piles were tested at two and a half months, a year and three and a half years after installation (see Figures 2.8, 2.9 and 2.10). Virgin piles tested three and a half years after installation showed an increase in capacity of 25% compared with load tests carried out on pile foundations immediately after installation. However, the virgin piles showed a tendency to increasingly brittle behaviour with age. The capacity of the "retested" piles was lower than the virgin piles, and these foundations showed no increase in capacity over the two year interval between tests (see Figures 2.11). In the two year period between tests the piles were not subjected to any loading.

Wardle et al. (1992) suggested that the increase in pile capacity with time is associated with a gradual "healing" of the failure surface in the soil, rather than a general increase in strength due to consolidation. Chapman et al. (2001) disagreed with this stating that the consolidation process that occurs around a pile after installation should cause capacity to increase with time. St John (2000) noted that in stiff overconsolidated clays undrained bearing capacity is greater after the ground beneath the foundation has fully consolidated although, as stated earlier, Powell et al. (2006) observed no increase in capacity on the piles retested after two years in heavy overconsolidated London Clay. Powell et al. (2006) suggest that no increase in capacity was observed due to the strain softening nature of London Clay. Tomlinson (1977) suggested that soil softening may occur as a result of surface water entering the gap between the upper part of the pile shaft and the surrounding soil, created due to pile installation, and that this could lead to a decrease in pile capacity with time. With driven piles, Bond et al. (1990) and Coop et al. (1989) observed that piles in heavily overconsolidated clay, such as London Clay, produced negative pore pressures and that the dissipation of these pressures would therefore lead to a reduction in the strength of the surrounding clay. Conversely Wardle et al (1992) observed no pore pressure changes in the surrounding soil during constant rate of penetration load testing on driven, jacked and bored piles. Figures 2.12, 2.13 and 2.14 show the trend of the pore pressure response during installation of pile A, D and B and the response up to 100 hours after installation. Piezometers were installed at a minimum radial distance of 0.3m from the centre line of the test piles as shown in Figure 2.1, implying that any changes in pore pressure were very small or confined to an area very close to the pile shaft. Chandler et al. (1982) agree that these locally generated pore pressures can dissipate rapidly as the shearing zone around the pile shaft is very narrow. The authors described a series of ten load tests, nine normally consolidated and one overconsolidated sample, on model piles installed in Speswhite kaolin clay. The piles were loaded under drained conditions.

In the literature there are numerous case histories for foundation performance with time in coarse grained soils, where an increase in bearing capacity of pile foundations over time was observed (e.g. Chow et al., 1997). One of these cases is a storehouse development in Hamburg harbour (König et al., 2006) where 42 year old piles were retested and the results were compared with original pile load test data from 1963 (see Figure 2.15). The pile foundations showed a significant increase in bearing capacity and stiffness. In general authors agree (e.g. St John, 2000; Chapman et al., 2001) that the consolidation of clay soil results in increasing capacity of piles, but in the case of coarse grained soil, as the consolidation process is probably finished prior to the first loading of the piles, the most plausible explanation for the increase in capacity is the relaxation of hoop stresses. The dynamic pile installation process results in high tangential hoop stresses around the pile shaft and this causes a reduction in radial stresses after pile installation (Chow et al., 1997). These stresses relax after a period of time that according to Schmertmann (1991) lead to an increase of radial stresses with time.

In order to understand foundation performance it is important to understand the behaviour of the soil, in this case overconsolidated clay, and the factors influencing this behaviour. It is well established that the behaviour of overconsolidated soil depends on the effective stress acting on it and its stress history. For clay soils the most important process representing the past loading history is overconsolidation. The deformation behaviour of clays including the general stress-strain relationship, shear strength and compressibility all depend on overconsolidation (Anandarajah, 2003). Therefore soil properties will vary with depth and this variation will need to be reproduced whenever a stratum of soil is to be modelled. An overconsolidated soil deposit is created geologically by a combination of swelling and recompression, which it undergoes to reach its current state. This loading is a result of processes of erosion of soil, re-deposition of alluvial deposits and changes in the sea level, and defines the stress history of the deposit (Stallebrass & Taylor, 1997). Many have attempted to relate the behaviour of piles to the effective stresses acting in the surrounding soil (Chandler, 1968; Burland, 1973; Meyerhof, 1976; Parry and Swain, 1977; Randolph et al., 1979; Kirby and Esrig, 1980). In order to use this approach, it is necessary to be able to understand the stress history of the soil around the pile. Once the initial in situ conditions have been determined, Wroth et al. (1980) state that the problem may be broken down and analysed in the following stages:-

1. Pile installation.

2. Dissipation of excess pore pressures generated by installation.

3. Pile loading.

When modelling foundations in clay it is considered important to install the foundations at the correct effective stress level. If the stress conditions acting around the pile immediately before loading may be predicted, then one can notice the changes of the effective stress as a result of loading the pile. Anderson et al. (1985) carried out laboratory tests on bored cast in situ piles in normally consolidated and overconsolidated clays and found that during excavation the horizontal effective stress (σ'_h) reduced dramatically but recovered 90% of the initial "at rest" effective stress after only 30 days (see Figure 2.16). The time required for stress recovery was dependent on the delay between boring and concreting, however, they concluded that it was probable that K_o values would eventually be re-established even if there was a considerable delay. The radial stresses vary during pile loading, depending on the properties of the surrounding soil. In overconsolidated soil the radial stresses will increase during pile loading (Chandler and Martins, 1982).

Hyde et al (1976) noticed the similarity in the behaviour of clay under repeated loading compared with that under creep loading. A series of triaxial tests with overconsolidation ratios of 4, 10 and 20 were investigated in this study. The samples were consolidated isotropically in the triaxial cell prior to being subjected to the various overconsolidated ratios. Figures 2.17 (a) and (b) show repeated load test stress path and stress path for creep load testing respectively. The repeated load tests involved sinusoidal variation of deviator stress at a frequency of 10Hz and constant confining stress. The results from the above tests were presented as a plot of strain rate against time using logarithmic scales. The sinusoidal stress pulse was approximated by a step function which allowed the creep test results to be used to successfully predict the behaviour under repeated loading. Figures 2.18, 2.19 and 2.20 show the results from the repeated load tests, and Figures 2.21, 2.22 and 2.23 shows the results for creep tests. Despite the scatter in the results for creep tests, a set of straight parallel lines were fitted to the points, and the mean slopes of these lines were found to be the same for both the creep and repeated load tests. It is also shown that the results are dependent on sample stress history.

There are several other papers discussing the repeated load properties of clay (Glynn et al, 1969; Murayama, 1970; Lashine, 1971). In all these papers it has been indicated that stress-strain-time behaviour under sustained (creep) loads follows a pattern similar to that experienced under cyclic loading. If repeated loading has a positive effect on pile capacity, which is also shown from the results of the preliminary centrifuge tests, the increase in pile capacity with time could also be associated with creep loading.

The requirements for position and capacities of a new building's piles can rarely coincide with what is available from existing pile locations. Unless there are particular site constraints which prohibit new foundation installation such as the presence of archaeology, tunnels or other obstructions, it will often be necessary for the existing foundations to be supplemented by new foundations to form the new foundation system (Butcher et al., 2006). Mixing old and new piles under new pile caps is possible but consideration should be given to possible differential settlement of the pile cap under loading (St John 2000). This is due to different pile types and also different stiffness response of similar pile types since the older piles have been pre-loaded. St John (2000) explains that in the case where the new building has locally higher loads than the original, an additional pile or piles can be installed through the slab, preferably directly under the column in question. Such a pile will work if sufficient load can be mobilised with a displacement which is compatible with the performance of the existing foundation under load.

The spacing of piles should be considered in relation to the nature of the ground, their behaviour in the group and the overall cost of foundations (BS 8004 cl. 7.3.4.2. – Spacing of Piles). There are generally three concerns regarding pile spacing:-

1. Stress changes in the soil adjacent to installed piles during construction of the new piles.

2. Provision needs to be given in case of unexpected pile inclination during drilling.

3. Design consideration of pile group effect.

According to BS 8004, for friction piles, the spacing centre to centre should not be less than three times the pile diameter. Fioravante (1998) observed increase in shaft resistance of a single pile belonging to a group compared to an isolated single pile due to the confinement offered by the surrounding piles. Jeong and Kim (1998) investigated the interaction factors due to group spacing using a three dimensional non-linear finite element approach. The response of groups were analysed by developing interaction factors. The major parameters that influence the interaction factors significantly are the group spacing, the total number of piles, and the relative position of piles within the group. For 1.2m diameter piles the interaction factors for centre to centre spacing of 2.5 and 5 diameters decreases as the number of piles in the group increases. This is particularly significant for groups of 9 to 25 piles. However, when a group of piles exceeds 25 piles in number, further reduction becomes small (Jeong and Kim, 1998).

2.2 Design and behaviour of deep foundations

Piles provide support by mobilising frictional forces on vertical shaft surfaces and bearing forces beneath the base. The predictions on the performance of pile foundations upon the application of ultimate axial loads are particular for soil conditions at any site where the piles are being installed and the behaviour of the soil at a relatively short time after installation. In clay soils the frictional forces are considerably greater than the bearing forces, unless the base is enlarged by underreaming. Thus the bearing capacity of most piles in clay soils is largely dependent on the area of the shaft surface and on the properties of the soil close to the shaft after the pile has been installed.

As a result of research in pile testing over the last fifty years the broad principles of the behaviour of bored piles in clay soils are widely understood. This understanding has influenced the appropriate sections of the "Code of Practice for Foundations" and has become embodied in the design philosophy of many consulting engineers and local authorities as noted by Burland et al., (1974).

Fleming (1992) states that when piles are subjected to gradual increasing loads, the pile-soil system behaves in a linear-elastic manner to a point. If the load is released at any stage up to this point the pile head returns to its original level. When the load is increased beyond this point slippage occurs between pile shaft and soil and skin friction on the pile shaft will be mobilised. When the friction has been mobilized fully the pile plunges downwards without any further increase in load, or a small increase in load produces large settlements (Fleming, 1992).

2.2.1 Pile capacity

The ultimate load on a pile can be defined as either the load at which settlement continues to increase without further additional loading or the load which causes a settlement of 10% of the foundation base diameter (Fleming, 1992). In this project the failure load has been deemed to be the load which gives a settlement of 10% pile base diameter.

For bored cast in situ piles in stiff overconsolidated clay the conventional method of estimating the load carrying capacity of a pile makes use of the undrained shear strength, S_u, of the clay in the calculation of both end bearing and shaft resistance (Patel, 1992).

Shaft capacity of piles in clay is calculated in terms of undrained strength (S_u) measured from quick undrained triaxial tests on undisturbed samples and an empirical adhesion factor, α, back calculated from pile tests such that:-

$$q_s = \alpha \, S_u \qquad 2.1$$

Where: q_s – ultimate shaft capacity

The value of the empirical adhesion factor, α, depends on the strength, stiffness and plasticity of clay, the size and type of pile and the method of installation (Tomlinson, 1957; McClelland, 1972; Chandler et al, 1982; Patel, 1992). This factor accounts for unknowns such as the effects of disturbance caused by the pile installation process. For overconsolidated or stiff clay a value of $\alpha < 0.5$ would be commonly used. According to Skempton (1959) the adhesion developed on the shaft is less than unity, chiefly because the clay immediately adjacent to the pile shaft absorbs water during the drilling operations and from the concrete. The method of pile construction has much improved since 1959 and the above is probably more relevant to tripod method of construction. Patel (1992) collected results from forty five tests on straight shafted bored piles. Piles tested ranged in diameter from 0.35m to 1.2m and 6m to 32m length. From these analyses the most important conclusions were as follows:-

- The shaft adhesion factors α from Constant Rate of Penetration (CRP) tests were much higher than the values obtained from Maintained Load (ML) tests. This difference is considered to be due to the rate effects which develop in CRP tests.

- The results showed that:-
  α_{CRP} = 0.5 to 0.8, average α = 0.6
  α_{ML} = 0.4 to 0.5, average α = 0.45

Burland et al (1988) showed that CRP tests over-predict the actual ultimate bearing capacity of bored piles in London Clay. The authors indicated that this was due to the rapid nature of the test. The rapid undrained shearing of the clay generates negative pore pressures at the pile shaft/clay interface i.e. an increase in effective stress.

The base capacity for pile foundations in stiff clay is calculated in terms of undrained shear strength, S_u.

The end bearing capacity is:-

$$q_b = s_c \, d_c \, N_c \, S_{ub} + \gamma \, H \qquad 2.2$$

Where:

q_b	- ultimate end bearing
s_c	- shape factor applied to N_c
d_c	- depth factor applied to N_c
N_c	- bearing capacity for surface strip foundation applied to S_{ub}
S_{ub}	- undrained strength at the foundation base
γ H	- is often compensated for by the self-weight of the pile and therefore ignored

The product of $s_c \cdot d_c \cdot N_c$ is approximately 9.0 for circular footings where the depths exceed four base diameters (Skempton, 1959). Thus:-

$$q_b = 9.0 \, S_{ub} + \gamma \, H \qquad 2.3$$

2.2.2 Design and behaviour of pile groups

Piles are normally constructed in groups of vertical, battered or a combination of vertical and battered piles. The load applied to a pile group is transferred to the soil. The design methodology adopted when the pile group is subjected to vertical load, should provide calculations of the group capacity and displacement such that the forces are in equilibrium between the structure and the supporting piles and between the piles and soil supporting the piles. The allowable group capacity is the ultimate group capacity divided by a factor of safety. The ultimate capacity of pile groups in clay soil is the lesser of the sum of the capacities of the individual piles or the capacity by block failure (see Figure 2.24).

The up-to-date literature on the performance of pile groups is an investigation on experimental testing, design and performance of pile groups constructed at the same time and usually of piles with equal diameters. When improving an existing pile foundation with mini-piles that have a different diameter the group becomes a "composite" foundation. From now on this type of group will be referred to as a "novel" pile group. The behaviour of each pile and the pile group effect in such cases differs from pile groups in which piles of the same diameter and material are used (Itani et al. 1996).

In the case of novel pile groups, the difference between the mini-piles and the larger piles lies in the diameter and slenderness ratio, where slenderness ratio=L/d (L – pile length and d – pile diameter). Thus, when subjected to axial loading, the behaviour of a mini-pile is mainly governed by the shaft friction mobilised at the soil pile interface. The base resistance is generally negligible and if it is not, it is mobilised long after the shaft friction has been mobilised.

The behaviour of the mini-piles under loading was analysed using different testing methods in the FOREVER project (1993-2001). The National Project FOREVER (FOundation REinforced VERtically) is a French National Project on Micropiles. The project was operated by the Civil Engineering and Urban Network and took place from 1993 to 2001. The goal of the project was to promote the use of mini-piles, in particular in groups or networks, by establishing an experimental and theoretical basis for their specific characteristics and applications.

The mechanism of mobilising the shaft friction in an axial load test of a mini-pile was experimentally demonstrated during the FOREVER project, by measuring the distribution of the compressive strain between the head and the base of the mini-pile. It has been assumed that the mini-piles remain elastic during the entire loading process, an assumption which appears to be appropriate for most cases, thus the compressive strain was proportional to the applied load. As the strain was proportional to the load, the strain curves may therefore be correlated to those of the stress distribution in the mini-pile. For small loads the mini-pile is only stressed along a part of its length and as the applied load increases, the stressed part increases. The curves of the load distribution on the mini-pile tend to become linear and parallel in its upper part as the applied load increases (see Figure 2.25). In reality, the shaft friction mobilization is more complex, but these simplified assumptions are sufficient to illustrate the different phases of the behaviour of mini-piles under loading. The FOREVER project showed that shaft friction depends significantly on the installation technique of the mini-pile (drilling tool, injection pressure etc). The soil type and its properties play an important role, but the governing design factors are related predominantly to the interaction between the mini-pile and the surrounding soil.

The experimental data from the FOREVER National Project has shown a positive group effect for groups comprised of a large number of mini-piles. It is believed that the group effect observed is most likely to be due to soil confinement between the mini-piles. The research on the behaviour of the mini-piles in the centre of the group also showed a similar confinement phenomenon.

2.2.3 Pile settlement

When the load is imposed on foundations installed in overconsolidated clays, it is initially taken by the pore water (i.e. the pore water pressure increases). Due to this, the pore pressures around the loaded foundation are not in equilibrium with the surrounding pore water, and these generated pore pressures will dissipate at a rate which is governed by the permeability of the soil. As the water pressure dissipates, the load will be taken by the soil skeleton and the soil particles will move slightly among each other and the soil compresses (reduces in volume), causing the settlement of the structure. Settlement and differential settlements are perhaps the most important features in pile design, and the problem is complicated by structural stiffness, pile load redistribution, construction technique and group effects (Fleming, 1992). From a series of pile load tests in London Clay, Skempton (1959) drew the following conclusions:

- Settlement at failure load is approximately 8.5% of pile base diameter.

- The shaft adhesion is fully mobilised at smaller settlements than the base resistance.

The results presented by Whitaker et al. (1966) show that the shaft and base resistance are mobilised at entirely different rates of settlements. Burland et al. (1974) showed that shaft friction develops rapidly and linearly with settlement and is generally fully mobilised when the settlement is about 0.5% of the shaft diameter. Hereafter it remains relatively constant irrespective of the pile settlement. On the other hand the base resistance is seldom fully mobilised until the pile settlement reaches 10-20% of the base diameter.

The shaft transfers the load to the surrounding soil by means of shear stress, which decreases in magnitude inversely with distance from the pile (Fleming et al., 1992). On the other hand the base load displacement response requires relatively large displacements (10% of pile base diameter or larger) to fully mobilise the ultimate capacity.

Fleming (1992) derived a pile settlement analysis using a composite approach incorporating both pile shaft and base components with elastic soil parameters and ultimate loads to describe the total pile response to maintained loading. The analysis is based on the use of hyperbolic functions as these more accurately represent load/settlement behaviour of a pile. The equations below are typical expressions for base and shaft settlement calculations:

Pile base response:-

$$\rho_b = (0.6\, P_b\, p_b) / (E_{25}\, d_b\, (P_b - p_b)) \qquad 2.4$$

Where: ρ_b - pile base settlement

P_b – ultimate pile base load at which the load displacement is infinite

p_b – pile base load

E_{25} – Young's modulus at 25% of base failure stress

d_b – pile base diameter

Pile shaft response:-

$$\rho_s = (Ms\ d_s\ p_s) / (P_s - p_s) \qquad 2.5$$

Where: ρ_s - pile shaft settlement

P_s – ultimate pile shaft load

p_s – pile shaft load

M_s – flexibility factor representing pile settlement caused by shaft friction

d_s – pile shaft diameter

When these functions are combined and elastic pile shortening is added by a relatively simple procedure, an accurate model is obtained for prediction of foundation performance and analysis.

When using new foundations to improve capacity of the existing piles, the settlement of old and new foundations have to be compatible. The load supported previously by the existing pile and the time this load was supported has to be considered and compared with the new loads to which the foundation will be subjected to in the future. Based on the information available, it is possible to design the new foundations to behave compatibly with the existing piles.

2.3 Case histories of the behaviour of re-used foundations

Reuse of foundations has been considered, and implemented, on several projects over the past decade where records of the existing foundations have been available and the projects have been refurbishment rather than new build.

In 2003, a major refurbishment of an office building at 13 Fitzroy Street in central London was completed (Anderson et al, 2006). It was intended to re-develop the building by re-using the existing reinforced concrete frame structure and adding a new extension to the existing building. The new loading conditions from the refurbishment resulted in 92% of the existing pile groups to be subjected to lower loads compared to the original design rated value. There was some localised overloading of the existing foundations by up to 31% when compared with the original pile group capacity. For the foundation layout see Figure 2.26.

The ground investigation carried out in 1956, for the development of No. 13 Fitzroy Street and the adjacent development sites, indicated Made Ground overlying Terrace Gravel and London Clay. As none of the available exploratory holes were within the footprint of the building the shallow strata levels beneath the structure could not be confirmed. Results of undrained shear strength from the 1956 tests were found by the authors to be comparable with more recent data from nearby sites (see Figure 2.27). There was no information available on the detailed design and construction of the existing foundations. However, information was available from intrusive investigation which determined material condition, pile geometry and load carrying capacity.

Anderson et al. (2006) suggested that as the application of load over the first building life was successful i.e. the building performed with no evidence of structural damage over its designed life, a reduced factor of safety for the existing foundations could be used demonstrating that the existing foundations would be sufficient to support the proposed redevelopment loading.

Vaziri et al. (2006) describe the reuse of piled foundations at Belgrave House in London. The project involved demolition of a seven-storey building followed by the construction of a new six-storey building. The ground conditions comprised Made Ground underlain by River Terrace Deposits and London Clay with the groundwater level one metre above the top of London Clay. The original design drawings were available but the existing column grid did not coincide with the new column grid (see Figure 2.28). New structures (e.g. transfer slab, transfer beams, capping beams, etc.) were needed in order to distribute the loads onto the existing piles. Vaziri et al. (2006) suggested that if the new loads exceeded the original pile loads then there would be a net permanent settlement, and reduction in pile stiffness. This would become progressively worse as the pile ultimate load is approached. If this happened the stiffness of the pile would reduce rapidly and excessive settlement would be observed which would subsequently lead to pile failure. Thus, where it was not possible or if there was a risk in overloading the existing piles, isolated new piles were installed.

The original design drawing showed that four types of piles were used to support the existing building, three of which were under-reamed (see Figure 2.29). The pile size ranged from 600mm diameter straight shafted piles to 1200mm diameter piles with 2800mm under-reams. Piles were generally shown to be 20m in length.

The authors concluded that pile reuse is a viable foundation solution if approached in the correct way. They emphasised the importance of understanding how the existing piles are likely to behave when subjected to unloading by demolition of the present building and reloading by the development. The authors suggested that the effect of the construction process taking place for the building redevelopment on the existing piles should be compared to a loading test on an under-reamed pile and the demolition stage should be compared to the unloading cycle of a pile in a maintained load test. The construction is analogous to the second loading cycle of a pile load.

St John and Chow (2006) describe two case studies of reuse of piled foundations which were successfully completed. The first project, The Empress State Building, involved refurbishing (use of the basic framing and fabric of the existing building with, possibly, some additional structure) and the second project, Juxon House, foundation reuse.

The Empress State Building was a 28 storey high concrete frame building founded on under-reamed piles. For the plan of the original foundations and original pile design see Figures 2.30 and 2.31. There were no pile test results found in archive data and there was no information to suggest that any piles were tested. Although no pile load test results were found, in effect, the building loads were imposed on the existing piles for over forty years constituting a long-term pile load test. The authors suggested a load take down analysis for the existing building to assess the previous loads applied to the pile foundation in order to determine the load capacity that the existing foundations would be reloaded to. In order that the existing piles remained stiff on reloading, the authors describe that the foundation design was based on the principle of limiting the new pile loads to below the maximum load that they have experienced previously and the excess of these loads to be transferred into new straight shafted bored piles. The plan of the new foundation layout is shown in Figure 2.32. The authors conclude that it is very rare that the opportunity exists to determine what the actual capacity or stiffness of existing piles is but in reality the performance of the piles is likely to be better compared to when the piles were first tested.

The redevelopment of Juxon House site involved the demolition of a ten storey office building and construction of a new eight storey building. The existing structure was founded on a number of under-reamed and straight shafted piles (see Figure 2.33). Prior to their reuse, all of the accessible existing piles were exposed and inspected for signs of concrete deterioration. During demolition, the piles were unloaded and underwent some elastic upward rebound. This was measured during demolition on four under-reamed piles and information was gained on unloading (and hence reloading) stiffness of the existing piles. The new additional piles that were added, their location and number were chosen following an iterative process of modifications between the structural and geotechnical engineers. In cases where the new piles were combined in the group with the existing piles, they were designed to be relatively stiff (i.e. mobilise load under small settlements) to be compatible with the stiff reloading response of the old under-reamed piles. Due to this and space restrictions, the new pile foundations were able to be constructed as straight shafted piles. The plan showing the outline of the new pile caps is shown in Figure 2.34.

Continuous Flight Auger (CFA) piles were used except on the east side of the site where rotary bored piles were used. These new foundations extended below the existing under-reamed piles by 20m to 25m depth and were designed using an overall factor of safety of 3.0. It was thought that possible interaction may exist between old and new foundations; only the pile length below the under-ream was considered as load carrying, with a factor of safety of 2.0. This working load was compared with the working load obtained taking into consideration the whole pile shaft with a factor of safety of 3.0 and the lowest value was adopted. The first criteria (FOS=3.0) always dictated the pile length.

The Leigh Mills Car Park redevelopment provided an “ideal” foundations reuse case (Tester and Fernie, 2006). The existing pile installation and the redevelopment were both carried out by Cementation Skanska, and extensive records of the original project had been retained. The reserve capacity of the existing piles was utilised and the existing pile groups were supplemented with mini-piles leading to a less complex foundation and superstructure construction and also significantly reducing programme and cost. The mini-piles were chosen as a supplement as they could be installed in low headroom, a frequent problem in foundation reuse situations. For the geometry of the composite pile groups see Figure 2.35.

The composite pile group performance prediction was approached in two ways, the strain based and stress based approach. The individual mini-pile settlement at working load was predicted and compared with the recorded settlement of the existing piles under load test. As the new and old foundation had a different stiffness, a difference in predicted settlement between old and new foundations was observed. This difference was very small, and it was considered insignificant. The load would initially be taken by the stiffer elements in the group leading to a further settlement of the existing piles. The load would then be distributed to less stiff elements, the new mini-pile foundations. Checks to ensure that the existing piles were not overstressed were carried out by Cementation Skanska.

The stress based approach looked at the capacity of the group to ensure that the spacing of the piles did not result in reduction of capacity and large settlement of the composite group. The group was considered as a large "mono-pile" with equivalent diameter and founded at an equivalent toe level. The factor of safety used in the ultimate capacity calculations was 3.0 and the settlement of the equivalent "mono-pile" was within allowable limits.

2.4 Trials at Chattenden

The effect of new foundations on existing foundations was studied by Cementation Skanska at a test site in Chattenden, Kent (see Figure 2.36). The site is a well calibrated London Clay site (Figure 2.37) and the tests were carried out as part of the Re-Use of Foundations for Urban Sites project, RUFUS. The RUFUS project was funded by the European Union aiming to provide ways to overcome barriers, both technical and non-technical, to the re-use of foundations for sustainable development. The geometry of the model for centrifuge testing was based on the geometry of a field test carried out at Chattenden.

Shotton (2004) reported some preliminary findings on the effect of installing additional mini-piles and a pile cap around existing piles from the trial in Chattenden. The observations were then reported in more detail by Fernie et al (2006).

A total of eight tests were undertaken (see Figure 2.38 and 2.39). The individual piles were installed from the ground level and were 7m long (see Figure 2.40). The groups were on the same overall depth but tied together with a 1m deep cap thus the active pile shaft was only 6m long (see Figure 2.41). The summary of tests undertaken is shown in Figure 2.42.

Single piles (T40 and T44) were tested for the first time in 2002 and then retested in 2004. There are no records from the 2004 test for the single pile T44 due to problems with data capture during testing. During the first loading the single piles both under-performed in comparison with theory and it is believed that this is due to problems with the pile installation, see Figures 2.43 and 2.44. When re-tested in 2004 pile T40 showed an increase in capacity of around 30% at a displacement of 10% of the pile diameter.

Different pile group configurations were also investigated. The mini-piles were of the same diameter (143mm) and length (7m) with a 6m active shaft, but the number and the spacing of the mini-piles in the group was varied. After the test in 2004 (T40 and T44), six mini-piles were installed around existing pile T40 and four mini-piles were installed around existing pile T44. The centre to centre distance between the existing pile and the mini-piles in the group for both cases was 325mm. At the same time the grouped ring of six and eight mini-piles were also installed and load tested.

The measured group resistance for all of the tests was less than the theoretical resistance. The groups were loaded in three stages. With the 4mm displacement case being chosen as the point at which the individual mini-piles reached their capacity, the 10mm displacement as a reasonable settlement limit for a group generally and at ultimate capacity. Assessments of group efficiency at 4mm settlement, 10mm settlement and at ultimate are shown in the Figures 2.45, 2.46 and 2.47. The theoretical ultimate capacity of test 5e was limited by the calculation for an equivalent large diameter pile whilst the theoretical capacity of the remainder were limited by the sum of the individual components. The groups (T7/T8 and T9/T10) reached their ultimate capacity at about 10mm displacement while the mini-pile groups supplementing an existing pile had one to two further load stages applied before the ultimate condition was reached.

The investigation illustrated above showed that in a controlled and a known environment with good history one should be able to maximise reserve capacity by accepting strain as the arbiter. In a less controlled and less understood environment a simple approach of global factor of safety on unaltered summations of individual components ultimate capacity would give a safe assessment of load carrying capacity that at least employs the cap reserve.

2.5 Summary

There are a number of published cases relating to the behaviour of jacked and driven piles when subjected to unload / reload cycles. However, for bored piles, there is a lack of data on their behaviour during loading cycles. The data that is available is from physical model testing, numerical analysis and field studies some of which were introduced. In an ideal situation a series of similar piles would be installed and loaded and then tested over a number of years; in this way the variations of capacity with time and loading conditions could be studied without any complicated additional variables. It is necessary to understand the behaviour of the ground during both, the first time loading and during unload-reload cycles. However, from the data available, it is clear that the bearing capacity of piles changes with time and that there is a relationship between the first and second loading conditions applied to the foundations piles.

It is well established that soil behaviour is directly related to stress history, together with the recent and anticipated stress paths. The best approach to understand the performance of the foundations during loading cycles is to improve the understanding of the mechanisms developed during pile loading through a combination of field studies, numerical analyses and physical model testing.

A number of cases were presented where novel pile groups were used. The information available suggests that the performance of the existing piles is influenced by the new pile foundations and the geometry of the novel pile groups is an issue that needs to be considered.

There are many issues that need to be considered when the reuse of existing foundations for a redevelopment site is considered. Important technical issues must be addressed to ensure that foundation reuse is undertaken appropriately (Butcher et al. 2006). Apart from technical issues, there are issues related to provision of insurance and warranties that need to be addressed. However, by reusing the existing foundations the time and cost of the new developments is reduced, archaeological remains are preserved and the impact on the environment is reduced. Reuse of foundations can generate economic advantages. Any reduction in the project time reduces the time the owner/developer is either paying interest on the capital outlay for the redevelopment or not receiving rental income (Butcher et al. 2006). The reuse of foundations will cut the use of natural resources simply because new foundations will not be made and the total energy used on a reused foundation site will be reduced since energy to manufacture, transport and place the constituent material for foundations will not be required. Furthermore, if existing foundations were to be removed the potential energy use in their removal and disposal of the materials will be avoided (Butcher et al. 2006). For the diagrammatic representation of some of the major activities in foundation construction and an estimate of what their relative cost might be see Figure 2.48.

CHAPTER 3

CENTRIFUGE MODELLING AND MODEL TEST PROCEDURE

3.1 Introduction

The potential for using the centrifuge as a modelling tool in civil engineering was recognized during the mid-nineteenth century by the French engineer Eduardo Philips. He proposed that the technique should be used for modelling the superstructure of a bridge. The use of centrifuge model testing for investigation of geotechnical problems started in the USSR between the First and Second World Wars (Schofield, 1980). Elsewhere its use was unknown until Mikasa (Japan) and Schofield (Cambridge) became aware of its potential in the 1960's (Craig, 1995). Due to this the first papers relating to geotechnical centrifuge work since 1936 were published at the International Society for Soil Mechanics and Foundation Engineering Conference in Mexico, 1969. Since then geotechnical centrifuge testing has become used extensively as demonstrated by contributions to specialist and general soil mechanics related conferences. Today the use of geotechnical centrifuge modelling is widespread but only in academic research establishments with the exception of Japan which has the greatest proportion of the world's geotechnical centrifuges used by both industry and academia. Davies (1998) stated that centrifuge modelling is considered by some to be a highly specialist experimental technique used by research that has very little direct relevance to engineering practice. However, centrifuge modelling has been recognised increasingly as a powerful technique in both geotechnical and geo-environmental engineering.

Before describing the equipment used in the centrifuge tests carried out in this research project and analysis of these results, the basic scaling laws and errors inherent in centrifuge modelling are described.

3.2 Principles of centrifuge modelling

Soil behaviour is governed by stress level and the stress history to which the soil has been subjected. As a consequence there is a need to model in situ stresses that change with depth to reproduce the strength and stiffness aspects of soil behaviour. The fundamental principle behind physical modelling is the reproduction of the stress distribution in the model as in the prototype. Physical modelling using a centrifuge involves accelerating a model contained in a strong tub or box, at the end of a centrifuge arm (see Figure 3.1), to create an inertial radial acceleration field many times greater than the Earth's gravity. In the model, stress increases rapidly with depth from zero at the surface to values that are determined by the soil density and radial acceleration.

The aim is to subject the model to a similar stress history to that assumed for the prototype. This is achieved by accelerating the model (of scale 1:N) at N times Earth's gravity using a centrifuge which then conveniently gives stress similarity at homologous points throughout the model.

Thus, when tested at an inertial acceleration field of N times Earth's gravity; the vertical stress σ_{vm} at depth h_m in the model should be the same as at depth h_p in the prototype, σ_{vp}.

$$\sigma_{vm} = \sigma_{vp} \qquad 3.1$$

Inertial stresses in the model correspond to the gravitational stresses in the prototype (see Figure 3.2).

For the range of soil depths encountered in civil engineering the Earth's gravity is uniform. When using the centrifuge in order to generate the same stresses in the model as in prototype, there is a slight variation in acceleration throughout the model. This is because the radial acceleration is a function of the angular velocity and radius from the centre of rotation. Newton's Second Laws of motion state that in pulling a mass out of a straight path into a radial path of radius r the centrifuge will impose an inward acceleration on the mass towards the axis of rotation of:-

$$a = \omega^2 r \tag{3.2}$$

where: ω – angular velocity (rad/sec)
r – radius from centre of rotation (m)

The effect of the radial acceleration is to increase the self weight of the model in the direction of its base.

Thus:-

$$a = N g \tag{3.3}$$

where: a – imposed radial acceleration
N – gravity scaling factor
g – acceleration due to gravity (9.81m/s^2)

With careful choice of model dimensions and radial acceleration, prototype stress profiles which vary with depth can be simulated closely.

3.3 Scaling laws

The model is a reduced scale version of the prototype, and this needs to be related by appropriate scaling laws. Having established the principles behind the creation of an equivalent gravity field N times greater than the Earth's the other scaling laws will now be explained. As stated earlier the fundamental principle behind centrifuge testing is the reproduction of the stress distribution of the prototype (Equation 3.1).

For the prototype with material of density ρ, at depth h_p, the vertical stress σ_{vp} is:-

$$\sigma_{vp} = \rho\, g\, h_p \tag{3.4}$$

Therefore if the density of the material in the prototype is the same as that in the model and central to the theory of centrifuge modelling is the fact that the acceleration of N times the Earth's gravity is applied, then the vertical stress σ_{vm} acting at depth h_m in the model is given by:-

$$\sigma_{vm} = \rho\ N\ g\ h_m \qquad 3.5$$

Thus:-

$$\rho\ N\ g\ h_m = \rho\ g\ h_p \qquad 3.6$$

$$h_p / h_m = N \qquad 3.7$$

Hence the scaling law for length is 1/N and it affects the geometrical properties of all components used in the model.

The affect that a mass has on a model is a combination of the increase in force it exerts due to the increase in acceleration level and the reduction in soil or foundation area on which it acts. The combination gives an effective scale factor for mass of $1/N^3$.

Powrie (1986) provides a comprehensive list of scaling factors (see Table 3.1), but the list of scaling factors, relevant to pile foundation models, derived from the scaling relationship for self weight stress (1:1) and for length (1:N), are given in Table 3.2.

3.4 Errors in centrifuge modelling

Whilst a centrifuge is an extremely convenient method of generating an artificial high gravitational acceleration field, problems are created by the rotation about a fixed axis (Taylor 1995). The effect of changing radius through the model (Equations 3.2) will result in model geometry moving away from a prototype.

In trying to model a prototype event it is inevitable that errors will result from the testing procedure as it is seldom possible to replicate precisely all details of the prototype. This is because there is variation in radius over the height of the model causing variation in acceleration in the Equation 3.2 (see Figure 3.3).

This section identifies the errors caused by the radial acceleration field in centrifuge model testing.

3.4.1 Vertical acceleration field

As stated earlier the stress distribution with depth in the model is non-linear (Taylor, 1995). The vertical stress at any point within the centrifuge model is calculated by taking the average acceleration acting upon the soil above. As acceleration varies linearly with radius this corresponds to the acceleration midway between the point under consideration and the model surface. Care must therefore be taken in model design to ensure that the over-stress at the model base and the under-stress near to the top are within acceptable limits. By finding expressions for the ratios of under-stress (r_u) and over-stress (r_o) to the prototype stress at the same depth and equating the two, it can be shown that the least variation is obtained when the required acceleration is set at 1/3 of model depth. This gives a correct stress at 2/3 model depth as shown in Figure 3.4.

Equations 3.1, 3.4 and 3.5 give:-

$$\sigma_{vp} = \rho\ g\ h_p = \rho\ N\ g\ h_m = \sigma_{vm} \qquad 3.8$$

The nominal gravity acceleration scale N is calculated using the effective radius as (Equation 3.2 and 3.3):-

$$N\ g = R_e\ \omega^2 \qquad 3.9$$

where: R_e – effective centrifuge radius for the model

If the radius to the top of the model is R_t, then:-

$$\sigma_{vm} = {}_0\!\int^z \rho\,\omega^2\,(R_t + z)dz = \rho\,\omega^2 z\,(R_t + z/2) \qquad 3.10$$

If vertical stress is identical in the model and prototype at depth $z=h_i$ then:-

$$R_e = R_t + h_i/2 \qquad 3.11$$

The stress variation with depth in a centrifuge model and its corresponding prototype is shown in Figure 3.4. A convenient rule for minimising the error in stress distribution is derived by considering the relative magnitude of under and over-stress, Taylor (1995). The ratio of maximum under-stress, r_u, which occurs at model depth $0.5h_i$, to prototype stress at that depth is given by:-

$$r_u = \{0.5h_i\rho gN - 0.5h_i\rho\omega^2[R_t+(0.5h_i/2)]\} / 0.5h_i\rho gN \qquad 3.12$$

When combined with equations 3.9 and 3.11, this reduces to:-

$$r_u = h_i / 4R_e \qquad 3.13$$

Similarly, the ratio of maximum over-stress, r_o, which occurs at the base of the model, h_m, to the prototype stress at that depth, can be shown to be:-

$$r_o = (h_m - hi) / 2R_e \qquad 3.14$$

By equating the ratios of maximum under-stress (r_u) and maximum over-stress (r_o) the following is obtained:-

$$r_u = r_o \qquad 3.15$$

$$h_i / 4R_e = (h_m - h_i) / 2R_e \qquad 3.16$$

$$h_i = (2/3)h_m \qquad 3.17$$

Thus:-

$$R_e = R_t + h_m / 3 \qquad 3.18$$

3.4.2 Radial acceleration error

The inertial radial acceleration is proportional to the radius which leads to variation with depth in the model (Equation 3.2). Stewart (1989) described the radial acceleration field acting in a direction that passes through the axis of the centrifuge. Hence, in the horizontal plane there is also a change in its direction relative to vertical across the width of the model. This means that there is an increasing component of lateral acceleration within the model as the distance from the centreline increases. This lateral acceleration will be greatest at the largest offset from the centreline i.e. boundaries at the soil surface. To minimise this problem the model needs to be shaped to take account of the radial nature of the acceleration field. It is therefore considered good practice to ensure that major events occur in the central region of the model where error due to the radial nature of the acceleration field is small. For the centrifuge model geometry see Figure 3.5.

3.4.3 Model foundation orientation in gravity field

Due to geometry of the centrifuge swing it was necessary to place the model foundations offset from the model centre line. The axis of each foundation was offset by 0.1m from the centre line, as shown in Figure 3.6. This resulted in foundation inclination to the resultant acceleration direction. However, the effects of this are mitigated by the direction of foundation loading which is kept fully in line with the foundation axis. The non-axial component of foundation load results from the net weight of the pile (for 10mm diameter pile 200mm long this is 2.2N load at 60g) which is small compared to the magnitude of the axially imposed foundation load. The tub sides are orthogonal to its base allowing the soil to swell uniformly. If the model piles were inclined so that they were parallel to the resultant acceleration direction, then they would be inclined to the principal direction of soil swelling and consolidation.

3.4.4 Boundary effects

The range of scales at which the prototype may be modelled is controlled not only by the practicalities of instrumentation but also by the boundary effects imposed by the container. The tub used for this research was manufactured from stainless steel and had a 420mm internal diameter with 400mm internal height, as shown in Figure 3.7.

Phillips (1995) gives guidance on containers and states that side wall friction is always present to some extent and, consequently, the model should be sufficiently wide so that it does not create significant problems. Craig (1995) suggests that a minimum distance of 5 pile diameters from the boundaries of the model container is sufficient to minimise such effects. The model foundations were 11 pile diameters (i.e. 110mm) away from the edges of the tub and a minimum of 5 pile diameters (i.e. 50mm) from the base of the tub.

3.4.5 Soil stress errors

The model soil was prepared in a consolidation press before the model was assembled and placed on the centrifuge (see Figure 3.8). The sample was subjected to vertical pre-consolidation pressure (p'_{max}) at the top. Before removal from the consolidation press the pressure was reduced leaving the soil layer with a constant mean effective stress distribution (p'_c) with depth. In the equivalent prototype the mean effective stress (p'_c) would increase with depth. If it is assumed that the clay surface has the stress history corresponding to the prototype, the difference between model and prototype can be defined in terms of stiffness, stress and permeability thus:-

- Stiffness

Stallebrass (1990) showed that the soil bulk modulus and shear modulus are dependent on mean normal effective stress and overconsolidation ratio (p' and OCR). The mean normal effective stress and overconsolidation ratio specify the current specific volume of soil. The difference in distribution of mean effective stress (p'_c) at the base of the model compared to the distribution in the prototype will result in higher specific volum. Thus, the overconsolidation ration (OCR) and therefore mean normal effective stress (p') will be lower in the model compared to the prototype leading to a reduction in soil stiffness.

- Strength

Using critical state soil mechanics to determine failure on the critical state line the undrained shear strength is:-

$$S_u = (M/2) \exp [(\Gamma - \nu) / \lambda] \qquad 3.19$$

As the specific volume is higher in the model than the prototype, there will be a lower gradient of undrained strength with depth in the model than in the prototype. This reduction of undrained shear strength is confirmed by Stewart (1989) in which laboratory and centrifuge tests in speswhite kaolin clay showed that:-

$$S_u = 0.22\, \sigma'_v\, OCR^{0.57} \qquad 3.20$$

where: OCR – overconsolidation ratio

- Permeability

For speswhite kaolin clay permeability is a function of voids ratio (Al-Tabbaa 1987). Hence, the reduction in permeability with depth will not be as rapid in the kaolin model as in the prototype owing to more uniform specific volume with depth.

3.5 The geotechnical centrifuge

The Geotechnical Engineering Research Centre at City University uses the Acutronic 661 centrifuge described by Schofield and Taylor (1988) and shown schematically in Figure 3.9. It combines a swing radius of 1.8m with maximum acceleration of 200g. A package weight of 400kg at 100g can be accommodated and this capacity reduces linearly with acceleration to give a maximum 200kg at 200g; thus the centrifuge is a 40g/tonne machine. The package is balanced by a 1450kg counterweight that moves radially on a screw mechanism. The swing platform at one end of the rotor has overall dimensions of 500mm x 700mm with a usable height of 960mm in the central area between the arms.

Four strain gauged sensors are used to detect out-of-balance operations in the base of the centrifuge. The signals from these sensors are monitored and if the out-of-balance exceeds the pre-set maximum of 15kN than the machine is shut down automatically. Such a safety feature enables unmanned overnight running of the machine.

The machine is situated in an aerodynamic shell which is surrounded by a block wall (see Figure 3.10). This wall is in turn surrounded by a reinforced concrete containment shell.

Electrical and hydraulic connections are available at the swing platform and are supplied through a stack of slip rings. Electrical slip rings are used to transmit transducer signals (which are converted from analogue to digital by the on-board computer and may be amplified prior to transmission in bits), to communicate closed circuit television signals, supply power for lights or operating solenoid valves or motors as necessary. The fluid slip rings may be used for water, oil or compressed air.

3.6 Apparatus design development

For the first model test a simple loading device shown in Fig.3.11 was used. This loading apparatus was developed by Morrison (1994). The loading device applies vertical load axially i.e. in line with the foundation. A linear bearing at the support beam of the loading rig was used to control the orientation of the loading pin (see Figure 3.12). The loading pins were instrumented by means of load cells, as shown in Figure 3.11, so the load acting on the pile head was measured. The vertical load was applied by dumping the water from the bucket suspended on the arm through a remotely controlled solenoid valve and then the foundations were subsequently un-loaded by adding water, through a hydraulic slip ring, to the water reservoir (see Figure 3.13). Details of the loading apparatus by Morrison (1994) are shown in Figures 3.14, 3.15, 3.16, 3.17, 3.18, 3.19, 3.20 and 3.21.

To measure the displacement of the foundations two Linearly Variable Differential Transformers (LVDTs) were used for each foundation. For the first test the LVDTs for both piles tested were supported by a 10mm x 50mm cross-section aluminium beam. The ends of the beams were clamped to the top flange of the tub (Figure 3.22). For details of the LVDT support see Figure 3.23 and 3.24.

The preliminary test indicated that the apparatus used, though the basic principle behind the design was very simple, was not very practical and required a long time to prepare the model for testing.

Following this, a period of design development for apparatus suitable to simulate loading, unloading and reloading associated with construction, demolition and reconstruction using the same pile foundations was undertaken. The new apparatus was designed such that the loading reservoirs and the LVDT support plate were completely independent and could be assembled prior to removing the sample from the consolidation press (Figure 3.25)

With the new apparatus the piles were loaded directly using water filled plastic reservoirs (Figure 3.26). The plastic reservoirs were guided by an aluminium tube (Figure 3.27). The water reservoirs rested on springs and moved vertically, thus axially loading and unloading the pile foundations (Figure 3.27). Pile foundations were loaded using a loading pin that was connected to the base of the loading reservoir (Figure 3.28). As the loading reservoir was resting on the spring during testing, it was necessary that the spring has a sufficient stiffness to support the weight of the reservoir prior to loading. At the same time the spring had to be long enough to allow further vertical movement of the reservoir to be able to load the piles. Spring geometry details are shown on Table 3.3. The spring had a sufficient stiffness to support the weight of the reservoir at 60g and allow further vertical movement when the reservoir was filled up with water during loading of the pile. The foundations were loaded by filling up the reservoir with water (see Figure 3.29) and unloaded by emptying the reservoir through a solenoid valve. For details of the base of the loading reservoir and connection to the solenoid valves see Figure 3.30. Figure 3.31 shows the solenoid valves mounted on the centrifuge tub. Figures 3.32 shows the plate used to connect the solenoid valves to loading apparatus during testing. The applied load was measured using a load cell that was connected to the loading pin. The reservoirs and solenoid valves were supported by a 12mm thick aluminium plate (Figure 3.33) that were mounted, when the apparatus was put together, and connected to the top flange of the tub (Figure 3.34).

When the new loading apparatus was developed a new support beam was also designed for the LVDTs. The LVDTs were supported by 10mm x 60mm cross-section beam (Figure 3.35). The beam was connected to the top flange of the tub using four M8 threaded rods which allowed vertical movement of the support beam dependent on the height of the clay model (see Figures 3.36 and 3.37).

3.7 Model container

The container used for testing was a cylindrical stainless steel tub with 420mm internal diameter and 400mm internal height (see Figure 3.38) It had access ports at 5mm, 50mm 100mm, 150mm, 180mm, 200mm and 250mm above the base through which pore pressure transducers could be installed. During the sample preparation an extension was mounted above the tub, as shown in Figure 3.39, which was removed when the tub was removed from the consolidation press. For drainage a 20mm thick base plate was designed as shown in Figure 3.40. The plate had a herringbone pattern of drainage channels machined into the aluminium surface and connected to drainage taps on opposite sides of the tub. Care was taken to ensure that the ends of the drainage pipes were kept submerged in water to prevent air from entering the sample.

3.8 Foundation type and installation

The position, depth and layout of the model piles was based on the geometry of foundations used for field tests carried out by Cementation Skanska under a European funded project entitled Reuse of Foundations for Urban Sites (RuFUS). Initially it was intended that the model piles would be made from cast resin in order to develop more realistic shaft friction during testing. It was however considered important that pore pressure transducers were installed at the base of the pile, so that there was a better understanding of load distribution between the shaft and the base of the pile. Thus, to place the pore pressure transducer at the base of resin model pile, a tube was used to form a hollow centre of the model pile, through which the wiring from the pore pressure transducer could be pulled. The resin was then cast around the tube (see Figure 3.41). Having established the benefits of casting the piles in situ a polyurethane resin and aluminium trihydrate (ON) filler was obtained from Mason Chemicals. The addition of this filler to the resin at a rate of 100g filler to 100g resin resulted in an easily pourable fluid (McNamara, 2001). After several attempts however, the use of resin piles was ruled out as diameter of the piles was very narrow. Thus, after the central tube was in place, the annulus between the tube and the bored hole was too small. Due to this it was impossible to have a uniform distribution of resin around the central aluminium tube.

As a result the model piles were made of solid aluminium rod of 10mm diameter and 220mm length (Figures 3.42 and 3.43). The model piles were embedded 200mm into the clay corresponding to 12m long piles at prototype scale at 60g (Figure 3.44). The model pile foundations were installed in holes pre bored into the clay at 1g prior to placing the assembled model onto the centrifuge swing. The holes were excavated using 10mm and 5mm outside diameter thin wall stainless steel tubes (see Figures 3.45 and 3.46) which were guided using jigs shown in Figures 3.47 and 3.48. Prior to placing the foundations in the hole a small amount of clay slurry was placed in the base of the hole using a syringe (see Figure 3.49). The clay slurry was used to ensure that the pile was in good contact into clay. In order to release trapped air a 0.5mm deep by 1mm wide channel was machined on one side of each pile (see Figure 3.50). For the mini-piles 5mm diameter and 100mm, 120mm, 200mm and 220mm long solid aluminium rods were used (see Figures 3.51, 3.52 and 3.53). The length of the mini-piles was varied depending whether their function was sacrificial and in providing a general stiffness effect or if they were to be loaded. The scale factors for centrifuge model testing are shown in Table 3.2. For the details of individual centrifuge model tests see Table 3.4.

As the influence of the mini-pile group was also investigated there was a need to design the 10mm diameter central piles in such a way that the length of the pile could be varied; whether the mini-pile group alone was loaded or the existing pile together with the mini-pile group were loaded. Thus resulted in the 10mm diameter piles being formed in sections, which could be added or removed (see Figure 3.54), to suit each individual test requirements.

Early tests concentrated on measurement of the load that was applied in the foundations and their displacement. Although the tub had a sufficient number of ports it was very difficult to place the pore pressure transducers sufficiently close to the piles for any pore pressure changes to be measured during the pile loading. It was therefore decided that pore pressure transducers should be installed in the base of the pile as shown in Figure 3.54, to enable a better understanding on the proportion of load supported by the shaft and by the base of the pile.

For all the tests undertaken the pile cap was not in contact with the clay surface, thus this gave no contribution to the pile performance. The cap was used as a reference point for measuring the displacement of pile foundations due to loading. The displacement was measured using two linearly variable differential transformers (Figure 3.55) and the mean value from these two readings was used in the results presented.

- When only the 10mm diameter piles were loaded a plate 2mm thick by 100mm long (see Figure 3.56) made from aluminium was connected to the pile 20mm above the clay surface as shown in the Figure 3.57 and 3.58.

- When the pile group was loaded a plate of 2mm thickness and 100mm diameter was used (see Figures 3.59 and 3.60).

3.9 Standpipe

The condition of the pore water pressure equilibrium was controlled by the top and bottom boundaries of the clay model. The model clay layer had an impermeable surface, as it was sealed with silicone oil (see Figure 3.61) with pore pressure increasing hydrostatically with depth. The average water table was 10mm below the surface level, with 14.6mm at the centre line of the swing and 0.1mm at right and left hand boundary of the model. The upper layer of the clay was theoretically in suction. The drainage base plate was connected to the standpipe, as shown in Figure 3.62. The standpipe was designed such that, depending on the height of the clay model, the height could be adjusted and the water table kept constant (see Figure 3.63).

3.10 Instrumentation

Three different types of instrumentation were used:

- Pore pressure transducers

Pressure transducers fitted with a porous ceramic front element were used to measure pore water pressure within the clay model. The pore pressure transducers used were PDCR 81 (see Figure 3.64). The same type of transducer without a porous stone was used to monitor the water level in the standpipe.

- Linearly variable differential transformers (LVDT)

LVDTs used to measure foundation and soil surface movements were Solatron DCO5 $\pm$5mm (see Figure 3.65). For each test pile two LVDTs were used. The average reading from the two LVDTs was used as a measure of the total pile settlement when analysing pile settlement behaviour.

- Load cell

Load cells used to measure imposed foundation loads at the pile head were Standard Strain Gauge Load Cells. The load cells used were sensors compression

All instrumentation was calibrated through the centrifuge data logging to check against the system before each test.

3.11 Stress history of soil used in the tests

London Clay is unfavourable for use in model testing owing to its extremely low permeability. The required sample preparation time in the consolidation press for this material would be months. Models for testing in clay soils are therefore usually prepared from kaolin owing to its relatively high permeability which minimises sample preparation time. It also has well researched characteristics (Al Tabbaa, 1987). The speswhite kaolin sample was prepared by consolidating clay slurry with 120% water content. The sample was prepared in a consolidation press before the model was assembled and placed on the centrifuge. The sample was subjected to incremental loading up to a vertical stress of 500kPa and than swelled back to 250kPa before being removed from the consolidation press. A preconsolidation pressure of 500kPa followed by swelling to 250kPa was used principally to ensure that measurable movements were achieved. The stress history was therefore chosen for the ability to make models that represented the essential characteristics of overconsoidated clay.

When removed from the press the total stresses on the soil sample was zero and the sample was subject to high negative pore pressures. In order to keep the effective stresses as close as possible to 250kPa the base drainage valves were closed and the exposed surface of the clay was sealed prior to and during model preparation. All necessary equipment for model making and testing were prepared prior to removing the sample from the press, so the model making would be as quick as possible, in order to prevent drying of the clay as much as possible. Undoubtedly however the model was affected during this time. This is shown by the pore pressure transducers readings immediately after spin up of a typical test, indicating a dissipation of negative pore pressures of up to 150kPa (see Figure 3.66).

As a result of the enhanced self weight of the model whilst spinning, the sample was subjected to further consolidation. Thus the sample continued to swell throughout the depth of the model. The degree of swelling of any element in a model is naturally dependent on its depth within the model.

3.12 Sample preparation

As stated earlier, the clay samples for the tests were prepared from a slurry at a water content of approximately 120%.
Slurry preparation was carried out in a large ribbon blade mixer. Distilled water and dry kaolin in powder form or, when available, recycled material from previous tests were mixed until uniform slurry was achieved. The time required for mixing when using recycled clay was longer compared to when clay in the powder form was used. It often took over four hours for a sample to mix when using recycled clay. Prior to use the tub was cleaned and all ports used for insertion of transducers were plugged. The inside surface of the tub was coated with Ramonol, white grease, in order to minimise any boundary effects imposed by the container. Phillips (1995) gives guidance on containers and states that side wall friction is always present to some extent and consequently, the model should be sufficiently wide so that such effects do not create significant problems. Measurements of movements should, if possible, be taken on the model centreline to minimise the effect (McNamara, 2001).

The base drainage system was covered with a 3mm thick porous plastic sheet over which a filter paper was placed. The filter paper was used to prevent any loss of clay particles.

The required thickness of the sample was 250mm. As the clay slurry had a water content of 120% an extension of the tub was necessary. The extension was 300mm deep with the same diameter as the tub. The extension was bolted on the top of the tub and sealed using a coating of silicone grease. The inside surface of the extension was also coated with Ramonol white grease. The coating of the grease in the tub and extension flange contact area was sufficient when the extension was bolted tightly to the tub to prevent any leakage of the clay slurry. The slurry was then placed into the tub using a scoop to prevent as much as possible the entrapment of air. When the required amount of slurry was in place, the surface of the model was covered with a filter paper and porous plastic sheet to enable top drainage. The sample was then placed in a consolidation press which had a loading plate that fitted tightly within the tub. The loading plate was 38mm thick and was made of stainless steel. The load was applied using a hydraulic ram. The applied load was controlled by a computer and the movement due to loading were measured using LVDTs. The initial loading for all the tests was in a range of 25kPa and this was usually the maximum pressure achieved during the first day, owing to the need to prevent slurry leaks during model preparation. In the next two days the pressure was gradually increased until the full required pressure was reached.

The sample was left to consolidate at 500kPa for approximately one week. During this time the pore pressure transducers, LVDT and the load cells were calibrated. After a week the vertical ram movements indicated by the LVDT were negligible. The pressure on the sample was then reduced to 250kPa to commence swelling. The reduction of pressure was carried out in a controlled manner and the sample was left to swell for at least 24 hours. Pore pressure transducers were installed through ports in the wall of the tub using special equipment as soon as the pressure was reduced. A stainless steel tube cutter was used to remove cores of clay slightly larger in diameter than the 6mm diameter of the pore pressure transducer head (Figure 3.67). To ensure that the transducers were installed at the correct level the cutter was guided using a reamed ferrule that screwed into the ports. The pore pressure transducers were removed from the de-airing and calibrating chamber and the stones of the transducers were coated with a small amount of clay slurry to prevent entrapment of air around the stone when the transducer was pushed in gently into the cored holes. Further clay slurry was applied around the transducer cable using a modified syringe. This ensured that no voids were left in the sample due to transducer installation. During the 24 hours of swelling, full consolidation around the pore pressure transducers was reached.

3.13 Model making

The drainage taps at the base of the tub were closed and all the water that had collected was removed to prevent water content changes in the model when the loading plate was raised. The extension was unbolted and removed and the tub was removed from the consolidation press. The top porous plastic sheet was removed and the surface was gently scraped to remove the filter paper and expose the clay surface. The edge of the model was then greased and the top surface was immediately sealed with silicone oil to prevent drying.

To avoid any boundary effects (i.e. have a 5d distance from the base of the tub) the minimum required depth of the sample was 250mm, leaving a 126mm distance from the top of the tub to the surface of the clay (Figure 3.68). For all tests the distance between the top of the tub to the surface of the clay was in a range of 110mm to 125mm, except for the test LQ3 were the model was too deep. Due to the set up of the loading apparatus, the depth of the clay sample was reduced using a scraper.

A stainless steel thin wall tube was used for boring the model pile foundation using as a guide tool the jig and the boss shown in Figure 3.10. For the pile installation see Section 3.8 foundation type and installation. At this stage only the 10mm diameter piles were installed. The model pile displacement during loading was measured using LVDTs resting on the pile cap during testing. Pile displacement was measured by measuring the displacement of the pile cap. Pile cap was a 2mm thick by 80mm long plate that was connected to the pile 20mm above the clay surface.

The support plate for the LVDTs was bolted to the top flange of the tub and the vertical displacement of the piles measured using two LVDTs for each pile. The surface of the model was then covered with 700ml of silicone oil and the model was weighed. The model was then placed on the swing and the base plate of the tub bolted to the swing platform. At this stage the LVDTs were zeroed and the model was complete and required only the loading apparatus to be placed. The loading apparatus support plate was then bolted to the top flange of the tub. The drainage control valves were already positioned at the edge of the loading apparatus support plate.

As the old apparatus (Morrison 1994 and Qerimi et al 2006) was used only for one test (LQ1) the model making will not be explained for this apparatus in detail. The initial stages (removal of the sample from the press, pile installation) are the same for all tests.

3.14 Testing

After the model was removed from the press and the model piles were installed, the apparatus was put together and the model was placed on the centrifuge swing. Once the model was on the swing the loading reservoirs were connected to the water supply. Connection of the transducers, standpipe, load cells and solenoid valves then followed. A camera and a light, to follow the reservoir movements, were positioned at the front side of the loading plate. All cables were securely fastened and the model was ready for spin up (see Figure 3.1). All the above operations took around 30 minutes to complete.
All tests were at 60g. When the model was spinning at 60g it was left for about 20 hours for the pore pressures to come into equilibrium. The rate of increase of the pore pressure was used as a guide to assess the best time to perform the test. The tests were not particularly complicated and no more than two people were required to execute them successfully. The foundations were loaded by filling the reservoirs with water through slip rings. The water supply valves were adjusted to ensure a constant loading rate. The foundations were loaded to failure (a displacement of 10% of the base diameter) or working load (FOS = 2) and left loaded for 10min. The piles were then unloaded and the centrifuge was stopped.

The loading apparatus plate and the LVDT support plate were removed. Around the existing piles 20mm high plastic rings were placed and the oil from the inner side of the ring removed to prevent oil entering the mini-pile bores. The existing piles were enhanced using mini-piles. The mini piles were installed using the same installation procedure as for the existing piles. The model was put back together and was left spinning for 4 hours for the pore pressures to come into equilibrium. The foundations were then loaded as explained in the previous paragraph.

3.15 Summary

The geotechnical centrifuge has been briefly described. The history, principles and limitations of centrifuge testing have been introduced and also the scaling laws and relevant errors have been explained.

A period of design development for apparatus suitable to simulate life, demolition and construction of the new structure using the same pile foundations has been explained. The apparatus developed by Morrison (1994) used for preliminary centrifuge model testing was described. The design and the basic principles of the new loading apparatus developed were described and explained.

The stress history of the sample to be tested was chosen to enable measurable movements whilst also preserving the essential characteristics of stiff overconsolidated clay. The parameters that were varied have been noted. The testing method, including model making and the procedure adopted for test in the centrifuge were described.

CHAPTER 4

EXPERIMENTAL WORK

4.1 Tests details

A total of twenty one centrifuge tests were carried out with two foundations located on each model. The testing procedure was described in section 3.14. The test configurations are presented in Tables 4.1 and 4.2, where the foundation geometry and behaviour during testing are given. Sample preparation and testing for each test took around two weeks, thus time constraints dictated that the variation of the geometry focused on the number of mini-piles in the group (Figure 4.1), the centre to centre spacing of the mini-piles with respect to the existing pile (Figure 4.2) and the length of mini-piles in the group (see Figure 4.3).

In all tests the foundation displacement was measured using two LVDTs (see Figure 4.4). The load applied was measured using a load cell and miniature pore pressure transducers were used to measure the pore pressure in the sample. From test LQ10 the model pile was designed and manufactured with a pore pressure transducer installed at the base. Thus from the test LQ10 the pore pressures changes at the base of the model centre pile were also measured (see Figure 4.5).

For all tests the soil sample was prepared from kaolin clay slurry with 120% water content. The samples were prepared in a consolidation press, loaded to 500kPa then swell back to 250kPa prior to removing form the press. When the sample was removed from the press, two 10mm diameter model piles were installed at 1g (Figure 4.6). The loading apparatus was put together and fixed to the tub. The model was then placed onto the centrifuge swing and spun up to 60g until the model reached a state of stress equilibrium at which point the piles were subjected to first loading. The model piles were tested to failure load or to working load. When these loads were reached the model piles were left loaded for around 10 minutes for excess pore pressures to dissipate. Using the solenoid valves the loading reservoirs were drained and the model piles were unloaded. The centrifuge was then stopped and one or both piles tested were enhanced with a mini-pile group.

- Test LQ1

Test LQ1 provided information on the behaviour of pile foundations when subjected to unload / reload cycles and the influence of the mini-pile group on the performance of the existing pile. For this test same loading apparatus developed by Morrison (1994) was used (see Figure 4.7). Both pile A and B were loaded for the first time to a failure load (10% of the pile base diameter which was exactly 1mm displacement). The mechanism of loading apparatus developed by Morrison (1994) is shown in Figure 4.8. The load was applied on the model piles by draining the water from the reservoirs using solenoid valves. Owing to a component failure during water drainage, pile A was subjected to higher load compared to pile B thus pile A settled by 12mm during the first loading.

After the first loading:

- The centrifuge was stopped and pile B was enhanced using eight 100mm long mini-piles. Centre to centre spacing between the existing pile and the mini-piles in the group was 30mm (3D, where D=10mm diameter of the existing centre pile), see Figure 4.9.

- Piles A and B were re-loaded after conditions of equilibrium were re-established. The mini-pile group itself was not loaded; only the existing pile foundation B was loaded.

- Pile foundations A and B were loaded for a third time under the same conditions immediately after the piles were unloaded from the second loading.

The loading apparatus developed by Morrison (1994) allowed for pile loading in a controlled manner and the mechanism used to develop this loading apparatus was simple however the apparatus could not be put together prior to removing the soil sample from the press. As a result of this together with lack of experience in centrifuge testing and a number of unforeseen problems, the time required for preparing the model for the first test was around seven hours.

Learning from the previous test a new loading apparatus was designed and developed (see Figure 4.10) to enable model preparation in as little time as possible. The new apparatus was designed such that the loading reservoirs and the LVDT support plate were completely independent and could be assembled prior to removing the sample from the consolidation press.

With the new apparatus, the piles were loaded directly using water filled reservoirs which were supported by springs whilst empty, as shown in Figure 4.10. Pile foundations were loaded using a loading pin that was connected to the base of the loading reservoir. The plastic reservoirs were guided by an aluminium tube. The water reservoirs move vertically, thus axially loading and unloading the pile foundations. The spring had a sufficient stiffness to support the weight of the reservoir at 60g and allow further vertical movement when the reservoir was filled with water, i.e. when loading the pile. The foundations were loaded by filling the reservoirs with water through the slip rings of the centrifuge and unloaded by emptying the reservoirs through solenoid valves. Applied loads were measured using load cells that were connected to the loading pin. The reservoirs and solenoid valves were supported by a 12mm thick aluminium plate that was mounted on and connected to the top flange of the tub when the apparatus was put together.

Figure 4.10 also shows the support beam that was designed for the LVDTs. The beam was connected to the top flange of the tub using four M8 threaded rods which allowed vertical movement of the supporting beam depending on the height of the clay model.

- Test LQ2

The time for putting the new loading apparatus together and its performance on the swing were tested in test LQ2. The settlement of the spring used to support the loading reservoir due to its self weight of the reservoirs at 60g was measured and compared with the calculated value. By determining the settlement of the spring at 60g the length of the loading pin was designed such that the piles were not loaded during spin up, thus the loading was completely controlled by filling and draining of the water reservoirs. As all the items of the apparatus and equipment could be prepared prior to the day of the test, the time required for model making was reduced substantially.

- Test LQ3

During model preparation for test LQ3, the volume of clay slurry used was higher compared to the first test (i.e. LQ1). When removed from the consolidation press the model was too deep, thus it had to be trimmed. The trimming of the model was done with great care, but this process took around two hours leading to changes in the effective stress in the model due to drying of the sample and applied pressure during trimming. After the model was put together it was placed in the centrifuge swing. Immediately after spin up some of the cables on the centrifuge arm came loose and the centrifuge had to be stopped to re-secure the cables.

Due to the change in height of the model, owing to swelling during reconsolidation in flight the length required for the loading pin was miscalculated; hence the piles were loaded during the spin up and buried in clay by approximately 10mm. As the model piles were buried prior to first loading they were 6m longer in prototype scale compared to model piles in test LQ1. Nonetheless it was decided that the test should proceed so that, if nothing else, the apparatus could be fully tested.

As the model reached a condition of pore pressure equilibrium the loading reservoirs were filled with water and the piles were subjected to first loading. During the first loading one of the load cells was trapped (pile B) and damaged. As a result the load applied to pile B was not correctly measured. The load cell could not be used for the second loading thus there were no readings available for pile B during the second loading. As a result of the series of problems mentioned above the data from test LQ3 were abandoned. Although there was not any important geotechnical research outcome from this test, it enabled the apparatus to be modified for the future tests.

- Test LQ4

The following test, LQ4, had the same model geometry as test LQ1 to enable direct comparison of results. The stresses applied during preparation on the consolidation press were 20% lower compared to the previous test. Thus the soil model had a different stress history. After the model piles were installed and the loading apparatus put together the sample was placed on the swing and spun up to 60g. Pile B was accidentally constructed longer by 5mm in model scale compared to pile A. After the pore pressures were in equilibrium the model piles were subjected to first loading. At this stage there was no information on the flow of water through the slip rings during the centrifuge spin up at 60g, thus no adjustments were made to the water rate prior to testing. During first loading it was observed that pile B was loaded at a rate that was considerably slower compared to the loading rate that was applied to pile A. The rate was increased until a similar behaviour during loading was observed on both piles tested.

During the first loading there was a leak at the connection of the drainage valve with the loading reservoir A. Although the pile was loaded to failure the load could not be maintained.

After the piles were subjected to first loading, the centrifuge was stopped and eight mini-piles 100mm long were installed at 3D (30mm) spacing from the pile B. The centrifuge was spun up again to 60g. After the pore pressures reached equilibrium the existing piles A and B were re-loaded to failure.

- Tests LQ5, LQ6 and LQ7

In tests LQ5, LQ6 and LQ7 the influence of the length of the mini-piles in the group on the existing central pile was investigated. All three tests had the same model geometry. The only difference between the tests was that in the test LQ7 piles A and B during the first loading were only tested to working load. In tests LQ5 and LQ6 piles were loaded to failure (10% of the pile base diameter which was 1mm displacement). After the first loading the centrifuge was stopped and only pile B was enhanced with eight 200mm long mini-piles. The centre to centre spacing between the existing pile and the mini-piles in the group was 30mm (3d, where d=10mm diameter of the existing centre pile). Although all of the tests were prepared using the same method, both piles A and B showed higher capacity in test LQ6 compared to test LQ5 (Figure 4.11).

- Test LQ8

The soil model for test LQ8 was prepared in the same manner as the previous tests. Two 10mm diameter piles 200mm embedded into clay were installed at 1g and then the model was placed in the centrifuge swing. During the input of the calibration constants for the load cells, pore pressure transducers and the LVDTs used for testing, the load cell calibration constants were incorrectly input by an order of magnitude of ten. Thus during testing the piles were subjected to higher load and displaced by more than 10mm during the first loading cycle. After the first loading the centrifuge was stopped and both piles A and B were enhanced with mini-pile groups; around pile A eight 100mm long mini-piles with 30mm centre to centre spacing were installed and around pile B sixteen 200mm long mini-piles with 30mm centre to centre spacing were installed. After the mini-piles were installed around the existing pile B, it was noticed that the spacing was not the same between the existing pile and all the mini-piles in the group. This was a result of inaccuracies from installation approaches which was redesigned for future tests.

- Tests LQ9, LQ10, LQ11, LQ12, LQ13, LQ14, LQ15, LQ16 and LQ17

In the tests LQ9, LQ10, LQ11 and LQ12 the effect of number, spacing and length of mini piles were investigated. The model piles were installed at 1g. The model was then placed in centrifuge swing. After conditions of pore pressure equilibrium were reached the model piles were subjected to first loading. After the load was maintained for around 10minutes, the centrifuge was stopped. In all tests both piles A and B were enhanced after the first loading. During foundation loading, in all of the above tests, little or no difference was noticed in the output of the pore pressure transducers. To get a better understanding of model pile behaviour during loading, and the distribution of the load between the shaft and the base of the model pile, it was decided to design and manufacture a model pile with a pore pressure transducer installed at the base. From the test LQ10 the new piles with pore pressure transducers at the base were used (see Figure 4.5).

The behaviour of the overconsolidated clays is dependent on the stress history to which the soil was subjected to. In tests LQ11, LQ12 and LQ13, piles A and B were installed after the sample was spun up in the centrifuge until pore pressure equilibrium was reached. Once the model reached equilibrium state, the centrifuge was stopped and model piles were installed. The 10mm diameter piles were installed and load tested prior to installation of the model mini-pile foundations. The same method as for installing the mini-pile foundations was used to install the 10mm diameter piles. A plastic ring with 150mm diameter and 20mm thick was used to prevent the surface oil from reaching the bored pile hole when installing the model piles. Figure 4.12 shows the 150mm diameter plastic rings. As the sample was already mounted on the centrifuge swing, there were restrictions on space. Due to this the initial preparation of the model took slightly longer compared to the previous tests.

In tests LQ9 to LQ12, the centrifuge was stopped after the first loading and one or both of the existing piles were enhanced with the mini-pile group. The model was then spun up to 60g and conditions of pore pressure equilibrium established before the existing piles were reloaded.

In tests, LQ13 to LQ17, after the initial loading of the single piles A and B, the foundations were enhanced with mini-pile groups. After the model reached pore pressure equilibrium only the new mini-pile groups were subjected to loading. As the capacity of the loading reservoir was not sufficient to load the mini-pile groups to failure, the maximum settlement reached due to applied loads were recorded. The performance of the piles A and B in test LQ15 was not as expected. During the first loading the displacements observed were much higher than in previous load testing of the piles to working load. During the second loading, only the new mini-pile group was loaded. In test LQ15(A) a load of 363N for 10mm displacement was reached. It was decided that the results obtained from test LQ15(B) be abandoned, as the mini-pile group was continuously displacing at very low applied loads.

- Test LQ18

In all of the tests described above, after the first loading the centrifuge was stopped and one or both of the existing piles were enhanced with mini-pile groups. As there was no information on the influence of stopping and restarting the centrifuge on pile capacity, it was considered an important issue to be addressed. In test LQ18 the single model piles were loaded to working load after the model had reached equilibrium. The piles were unloaded after around 10 minutes, and the model continued to spin until the excess pore pressures generated during first loading dissipated. The piles were then loaded for a second time to working load.

After the second loading the centrifuge was stopped and mini-piles were installed around piles A and B. In order to determine the effects of method of installation of the mini-piles on the existing pile performance, eight mini-piles 200mm long were driven at 15mm spacing from the centre pile A (see Figure 4.13). Around pile B eight 100mm long mini-piles were installed at 30mm spacing from the centre of pile B. The apparatus was then put together and after the pore pressures reached equilibrium, the existing piles A and B were re-loaded for the third time.

- Test LQ19, LQ20 and LQ21

In the test LQ19, LQ20 and LQ21 10mm diameter model piles were installed at 1g prior to placing the model on the centrifuge swing. The apparatus was put together and the model was spun up to 60g and left over night for pore pressures to reach equilibrium. After pore pressure equilibrium was established, the model piles were subjected to first loading. The load was maintained for around 10 minutes. In tests LQ20 and LQ21 the centrifuge was then stopped. The model mini-piles were installed around the existing piles the centrifuge restarted and the model was left spinning until the generated pore pressures dissipated. Centrifuge was not stopped between the first and second loading cycles in test LQ19. During the second loading cycle the existing piles were loaded together with the mini-pile group. It was observed on the previous test, when the behaviour of the novel pile groups was investigated, that the available capacity of the water reservoirs was not sufficient to load the pile groups to failure. Due to this after test

LQ19 the apparatus was redesigned and the capacity of the loading reservoirs was almost doubled (see Figure 4.14). When re-designing the loading apparatus to increase the available capacity of water reservoirs, geometric constrains of the centrifuge swing had to be considered. Although the reservoir capacity was enhanced, it was not sufficient to load the group to failure (see for example Figure 4.29). The groups were loaded to a maximum available capacity and the achieved displacements were noted. Any further increase in water reservoir capacity it would require a total redesign of the loading apparatus.

As the available loading capacity of the apparatus was increased, the weight of the reservoirs also increased. Due to this initial displacement of the spring at 60g was higher. The loading pin could not be shortened due to the geometry of the centrifuge model thus the position of the loading cell had to be change (Figure 4.15(a)). To avoid any damage the load cell was placed above the spring for test LQ20, as shown in Figure 4.15 (b). During test LQ20 the load cell measured the load that was applied on the pile together with the load taken by the spring. It was unsuccessfully attempted to extract the load applied on the piles by looking at the readings of the load cell. It was then decided to place the load cell inside the spring, see Figure 4.15 (c), and repeat test LQ20. Test LQ21 had the same geometry as test LQ20 and it was a successful test.

For the details of the model geometry for all of the above tests see Table 4.1.

4.2 Observations and results

4.2.1 Behaviour of single pile foundation when subjected to load/unloading/reload cycle – Tests LQ5(A), LQ6(A), LQ7(A) and LQ13(B)

Tests LQ5(A), LQ6(A), LQ7(A) and LQ13(B) investigated the effects of load/unload/reload cycles on single pile foundations. Two different scenarios were investigated:

- The behaviour of piles that had initially been loaded to failure (LQ5 and LQ6)

- The behaviour of piles that had initially been loaded to working load (LQ7 and LQ13)

In all of the above tests, after the pile foundations were subjected to first loading, the centrifuge was stopped. Piles were then subjected to second loading after the sample had reached the equilibrium state. These tests indicated an increase in capacity when subjected to second loading. It was also noticed that the behaviour of piles during the second loading was dependent on the load to which the pile was subjected during the first loading.

As stated in section 2.2.3, the failure load was considered to have been reached when the foundation had displaced by 10% of the pile base diameter. A number of centrifuge tests on single piles were undertaken and in all the cases the failure load reached was around 100N. To calculate the working load a factor of safety (FOS) of two was used, thus giving a working load of 50N. During the second loading, in all the above tests, piles were loaded to failure.

Figure 4.11 shows a plot of first and second loading on a single pile foundation for tests LQ5 and LQ6. Both tests were performed using the same testing method and as expected the piles performed in a similar manner. During the second loading an increase of around 20% in pile capacity was observed in both tests.

In tests LQ7 and LQ13 the piles were loaded for the first time up to the working load, and displacements reached during loading were measured. When subjected to first loading, piles in tests LQ7 and LQ13 did not perform in a same manner. The pile in the test LQ13 settled more than expected. Even though the performance of the piles during the first loading was different, during the second loading both piles reached an ultimate load capacity of around 85N (see Figure 4.16).

4.2.2 Effect of the mini-pile group on the existing pile

When new foundations are required to enhance the capacity of existing piles, it is important to understand the effects of these new foundations on the existing piles. This research investigated the effect of new foundations on the existing piles by varying the geometry of the mini-pile groups (i.e. number, length and spacing of these new mini-pile groups).

4.2.2.1 Effects of spacing of mini-piles on the existing pile – Tests LQ9(A), LQ11(A), LQ10(B) and LQ12(A)

When investigating the influence of the centre to centre spacing of the mini-pile group from the centre pile, three different scenarios were investigated:-

1. The centre to centre distance of 1.5D between the existing pile and the new mini-pile foundations.

2. The centre to centre distance of 2D between the existing pile and new mini-pile foundations.

3. And the centre to centre distance of 3D between the existing pile and new mini-pile foundations.

Where D is the diameter of the existing pile and D = 10mm (see Figure 4.17).

As the diameter of the existing pile and the new mini-pile foundations was different, it was decided to model the geometry in terms of centre to centre distance between old and new foundations (not between the mini-piles in the group).

In all tests described, piles were loaded to working load during first loading and to failure load during the second loading when enhanced by the mini-pile group. All tests were prepared and tested in the same manner. The single piles were subjected to first loading, the centrifuge was then stopped and the mini-piles were installed. After the model had reached equilibrium stresses, only the existing piles were re-loaded to failure. As a datum test LQ7 was used. Test LQ7 investigated the behaviour of single pile foundation subjected to load/unload/reload cycles when the piles were initially loaded to working load.

There were no successful data from the tests investigating the influence of the mini-pile group on the existing pile when the group was installed with 3D spacing between the existing and new pile foundations.

Tests LQ9 and LQ11 investigated the effect of eight 100mm long mini piles. The centre to centre distance for test LQ9 was 1.5D and 2D for test LQ11. Figure 4.18 shows the load settlement behaviour for the test LQ9 and LQ11 during first and second loading. For the mini-pile group with 2D spacing the load/displacement behaviour suggested a higher capacity by around 10% compared to the mini-pile group with 1.5D spacing. The same behaviour was observed for 200mm long mini-piles as well. The load/displacement behaviour for the mini-pile group with 2D spacing (LQ12) suggested a higher capacity compared to the group with 1.5D spacing (LQ10), see Figure 4.19. In this case an increase in capacity of around 15%.

When compared with test LQ7 (see Figure 4.20), it can be noticed clearly that the mini-pile group has a positive effect, in terms of improving the performance of the existing pile foundation. The length of the mini-pile also influences the performance of the existing pile, but this will be discussed in more detail later.

4.2.2.2 Effects of number of mini-piles on the existing pile – Tests LQ11(A), LQ10(A), LQ12(A) and LQ12(B)

In tests LQ10 and LQ11 the existing piles were installed at 1g and the model was assembled. The sample was then placed in the centrifuge. After the equilibrium stress profile was reached the piles were loaded up to working load (50N) and the load was maintained for 10 minutes. The centrifuge was then stopped and mini-pile groups were installed around the existing centre pile. The effect of the number of mini-piles in the group on the performance of the existing pile previously loaded to working load was investigated. For tests LQ11 and LQ10 100mm long mini-piles were used with 2D (20mm) spacing. The initial novel pile groups investigated were based on trials carried out at Chattenden (Fernie et al, 2005). Due to the geometry of the model, the maximum number of the mini-piles in the group that it was possible to investigate was sixteen.

For test LQ11 eight mini-piles were constructed around the existing pile after the first loading. For test LQ10 sixteen mini-piles were constructed around the existing pile after the existing pile was subjected to first loading.

Comparing tests LQ10 and LQ11 with the behaviour of the single pile subjected to load/un-load/re-load cycles when loaded for the first time to working load (test LQ7), it can be seen clearly that both mini-pile groups have a positive effect on the performance of the existing pile (see Figure 4.21). When comparing groups of eight mini-piles (test LQ11) with the groups of sixteen mini-piles (test LQ10) at the same pile displacement, the group of eight mini-piles reached a higher load capacity compared to the group of sixteen mini-piles by around 10%.

For test LQ12 the mini-piles were 200mm long. Test LQ12(A) had eight mini-piles constructed around the existing pile and test LQ12(B) had sixteen mini-piles constructed around the existing pile. When loading for the second time, the existing piles were not loaded to failure (the existing piles were displaced by only 4%), as the piles showed no more increase in load with continued displacement. The behaviour observed was similar to the 100mm long mini-piles. Eight mini-piles in a group showed an increase in capacity of the existing pile of just above 10% compared with a group of sixteen mini-piles (see Figure 4.22).

4.2.2.3 Effects of length of mini-piles on the existing pile – Tests LQ11(A), LQ12(A), LQ10(A) and LQ12(B)

The effects of the length of the mini-piles in the group on the performance of the existing piles were investigated. Groups with 100mm and 200mm long mini-piles were considered. In test LQ10(A), LQ11(A), LQ12(A) and LQ12(B) the existing piles during the first loading were loaded up to working load.

In tests LQ11(A) and LQ12(A), see Figure 4.23, the groups investigated were of eight mini-piles with 2D spacing. In tests LQ10(A) and LQ12(B), see Figure 4.24, the groups investigated were of sixteen mini-piles with 2D spacing.

During the first loading of piles in tests LQ11 and LQ10 model piles displaced more compared to the performance observed in tests LQ12 (A) and (B). When subjected to a second loading cycle the existing piles in test LQ11 and LQ10 were loaded until the piles reached displacements of 10% of the pile diameter. In tests LQ12 (A and B) the existing piles were displaced by only up to 4% of existing pile diameter, as there was no more capacity available in the water reservoirs for further loading.

Comparing test LQ10, LQ11 and LQ12, at pile foundation vertical displacements of 4%, the 200mm long mini-pile increase the capacity of the existing pile by around 20% compared to the 100mm long mini-piles (see Figures 4.25 and 4.26). The same performance as described above was observed for mini-pile groups of eight and sixteen mini-piles.

4.2.3 Behaviour of novel pile group foundation when loading the mini-pile group only – LQ17(A), LQ17(B), LQ16(A), LQ15(A) and LQ13(A)

The behaviour of a novel pile group, when loading the pile group only, was investigated by varying the spacing, number and length of the mini-piles in the group. The geometry of the groups investigated in this research was different from the pile groups used normally in the construction industry as it contained a preloaded existing pile at the centre of the group. Thus it was considered important to investigate the performance of these novel pile groups when loading the new mini-pile group only.

For all subsequent tests the same testing procedure was followed. Single piles were installed and tested to working load after the equilibrium stresses were reached. The centrifuge was then stopped and the mini-piles were installed around the existing pile. After the pore pressures reached the equilibrium values, the mini-pile groups were loaded to failure or to the maximum available capacity of the loading reservoir.

The spacing of 2D and 3D between the existing pile and the mini-pile group was investigated for pile groups of four (LQ17) and eight mini-piles (LQ16, LQ15). In all cases the mini-piles were 200mm long. Figure 4.27 and Figure 4.28 show that the pile groups with 2D spacing performed better compared to the groups with 3D spacing.

The performance of four mini-piles with a group of eight mini-piles was compared. For all the tests the mini-piles were 200mm long. The performance was compared for 2D spacing (tests LQ17(A) and LQ16(A)). From Figures 4.29 it can be observed that the mini-pile group of eight performed better compared to the group of four mini-piles.

As the test results for LQ15(B) were abandoned, there were no results available to make a direct comparison between the performance of 100mm and 200mm long mini-piles.

4.2.4 Behaviour of novel pile group foundations when loading the existing and new foundation – LQ7(A), LQ18(B) and LQ14(B)

If the existing foundation capacity is not sufficient for the new development it was suggested that the capacity can be enhanced by placing a mini-pile group around the existing pile. The influence of the geometry of the group was discussed in section 4.2.2.

In tests LQ7, LQ18 and LQ14, the 10mm model piles were installed at 1g and then the model was placed in the centrifuge. The single piles were tested up to working load after the pore pressure transducers reached equilibrium pressures. After the load was maintained for 10 minutes, the piles were unloaded and the centrifuge was stopped. In test LQ18 the piles were loaded for a second time without stopping the centrifuge after the first loading.

After the centrifuge was stopped, in tests LQ18 and LQ14, the mini-piles were installed around the existing pile. The mini-pile group geometry was the same for both tests; eight 100mm long mini-piles with 3D spacing.

After the pore pressures reached equilibrium the foundations were load tested. In tests LQ7 and LQ18 the existing pile only was loaded. In test LQ14 the existing pile foundation was reloaded together with the mini-pile group.

During the first loading foundation behaviour in tests LQ7 and LQ18 was similar whilst the model pile in test LQ14 displaced more for the same applied load. As expected, during the second loading the enhanced pile (LQ18) performed better compared to a single pile foundation. When loading the existing pile together with the pile group the capacity obtained from the novel group was, as expected, much higher (see Figure 4.30).

4.2.5 Effects of method of pile installation – LQ18(B) and LQ10(B)

Following the tests on the performance of the existing single pile foundations with mini-pile group around, a test was undertaken to investigate the influence of the installation of the mini-piles on the capacity of the existing pile. In the initial tests the mini-piles were installed by boring a 5mm diameter hole in the clay using a thin wall 5mm outer diameter tube. A small amount of clay slurry was placed into the hole to ensure that the mini-piles were in contact with the clay model and then, using a guiding boss, the mini-piles were installed.

In test LQ18 after the existing pile was subjected to first and second loading the centrifuge was stopped and eight mini-piles were driven around the existing pile at 1g. This was done in order to investigate the effect of the method of model pile installation on the foundation performance during testing. The centre to centre distance between the existing pile and the piles in the mini-pile group was 1.5D and the mini-piles were 200mm long. The loading apparatus was then put together and the model placed on the swing and spun up to 60g. After pore pressure equilibrium was reached the existing pile was re-loaded to failure.

The results obtained from test LQ18 were compared with test LQ10. Test LQ10 had the same model geometry as test LQ18. In test LQ10 the mini-piles were bored in place after the existing pile was subjected to first loading. The difference in the foundation behaviour between the tests LQ10 and LQ18 was very small. The load taken by the pile in test LQ18 was slightly higher compared to test LQ10 (see Figure 4.31). However, as the existing centre pile was loaded twice before the mini-piles are installed in test LQ18, it would be expected that the single pile in test LQ18 would perform better compared to the pile loaded only once prior to the mini-pile group installation (test LQ10).

4.2.6 Effects of stress history prior to and after pile installation of the centrifuge model on the behaviour of single pile foundations during loading - LQ7(A), LQ7(B), LQ13(A), LQ13(B), LQ5(A), LQ6(A), LQ19(A) and LQ19(B)

4.2.6.1 Effects of stress history after pile installation – LQ5(A), LQ6(A), LQ19(A) and LQ19(B)

During each centrifuge spin up two foundations were tested. After the first loading the centrifuge was stopped and one or both pile foundations were enhanced with different geometry mini-pile groups, depending on what was investigated at the time. In order to confirm if stopping/restarting the centrifuge had any effect on pile capacity it was decided to load foundations for the second time without stopping the centrifuge after the foundations were loaded for the first time. In test LQ19 the single pile foundations were installed at 1g prior to placing the assembled model onto the centrifuge swing. After equilibrium was reached the foundations were loaded to failure for the first time. The performance of the foundations during the first loading was in good agreement with the previous test as shown in Figure 3.32. The model was left spinning and after pore pressure equilibrium was reached the foundations were loaded to failure for a second time.

The pile foundation behaviour from test LQ19 was compared with tests LQ5 and LQ6. In tests LQ5 and LQ6 the centrifuge was stopped between first and second loading. In Figure 4.32 the results from the first and second loading for test LQ5, LQ6 and LQ19 are presented. It can be see from the graph that the time that the centrifuge was stopped between first and second loading appeared to have no effect on the foundation performance.

4.2.6.2 Effects of stress history prior to pile installation – LQ7(A), LQ7(B), LQ13(A) and LQ13(B)

Ideally the model pile foundations would have been installed during flight. Installation would have taken place after equilibrium stresses were reached. This, however, was not attempted due to complexity of the apparatus and the installation procedure. To determine if the behaviour of the piles during loading would have been different if piles were installed after the soil model had reached the equilibrium condition, in test LQ13 the model was placed in the swing as soon as it was removed from the consolidation press. When the model reached equilibrium stresses the centrifuge was stopped. Model piles were installed using a 10mm outer diameter thin wall tube for boring. To stop the surface oil from entering in the bored holes, 20mm high plastic rings were placed around the intended position of the centre piles. The model was then put together and the centrifuge was restarted. After pore pressure equilibrium was reached the piles were subjected to first loading. The model piles in test LQ7 were installed at 1g after the model was removed from the consolidation press. When comparing the performance of test piles LQ7 and LQ13 (see Figure 4.16), initially the pile performance from both tests was quite similar. As the load increased to working load the single pile in test LQ13 settled significantly more compared to the pile in test LQ7. During second loading the performance observed in tests LQ7 and LQ13 was similar as shown in Figure 4.16. The performance of the piles when reloaded appears not to be influenced by the stress history of the model prior to pile installation. As for the difference in behaviour during first loading the pile installation method for test LQ7 was a more practical and controlled operation compared to the method used for test LQ13 where the piles were installed with the model placed on the centrifuge swing.

4.3 Pore pressures

Druck PDCR81 pore pressure transducers were installed at various depths within the model as shown in Figure 4.33. The principal purpose for using the pore pressure transducers was to determine the time at which the model reached pore pressure equilibrium. From test LQ10 onwards the pore pressure transducers were installed at the base of the model pile foundations (see Figure 4.5). During the consolidation period on the swing, pore pressures at the base of the piles were used as those installed in the soil model to determine equilibrium conditions. During the pile loading the pore pressures at the base of the piles were used to give an indication of the load distribution between the shaft and the base of the model piles. In cases where some of the pore pressure transducers failed during testing, the model was left spinning long enough to be certain that the pore pressure equilibrium was reached based on the timings from previous tests. During the test series the thickness of the clay model was consistently 250mm $\pm$ 10mm, thus the time required for the model to reach pore pressure equilibrium was well known and in the range of 12 hours.

A typical response of the pore pressure transducers during pile loading is shown in Figure 4.34. There was no response from the pore pressure transducers installed in the clay during pile loading. The pore pressure transducers in the base of the model pile however, did respond to pile loading and the excess pore pressures were found to dissipate very quickly. The equilibrium pressures after the first loading were reached far quicker compared to the time required to reach equilibrium prior to first loading of the piles. These equilibrium pressures were usually higher compared to initial values due to changes on the stress conditions during model preparation (i.e. stopping the centrifuge, installing the model mini-piles and spinning up the model for the second time).

4.4 Summary

The programme of tests conducted and the reasons for the developments in testing taken has been described. A number of different events that may have had an impact on the results have been investigated.

The single piles showed an increase in capacity when re-loaded. The behaviour of the piles during the second loading is dependant on magnitude of load to which the piles were initially subjected. The results from test LQ19 show that stopping and restarting the centrifuge had no effect on the load capacity of the existing piles during second loading.

Apart from looking at the behaviour of single piles subjected to load/unload/reload cycles the influence of the mini-pile group on the existing pile was investigated. From the results presented it is clear that the mini-pile group does influence the behaviour of the existing pile. The influence of the group was considered in terms of length, spacing and number of the mini-piles in the group. The 200mm long mini-piles, spacing of 2D and the groups of eight mini-piles suggested better performance compared to 100mm long mini-piles, spacing of 3D and groups of sixteen mini-piles respectively.

The method used for installation of the mini-piles in the group appears not to affect the performance of the existing piles when re-loaded. Both driven and bored mini-piles were investigated.

Behaviour of novel pile groups of four, eight and sixteen mini-piles were investigated when loading the pile group only and also when loading the existing pile together with the mini-pile group. In all tests presented the geometry of the models was such that the existing pile was at the centre of the group. For all the cases the existing piles were loaded up to working load during the first loading. During the second loading the foundations were loaded to failure or if failure was not reached, to the maximum capacity available from the loading reservoir. As expected the maximum capacity was achieved when loading the existing pile together with the mini-pile group.

CHAPTER 5

DISCUSSION

5.1 Introduction

This chapter presents a summary of the observations made from a series of centrifuge tests undertaken and provides explanations of the performance observed. In order to enable the results to be of maximum use, the results are presented within a context relating to the specific problem of establishing the trends of behaviour of re-used piles. This investigation will aim to determine points relevant to foundation reuse.

Reuse of existing piles has been very hitherto limited. Ground congestion is one of the prime drivers for the re-use of foundations in the urban environment. Urban centres are developing continuously thus the re-use of piles will become even more critical with time.

In order to re-use the existing piles the following needs to be considered:-

- Verification of capacity of elements considered for re-use.
- The condition of construction materials, i.e. durability of materials used for foundation construction.
- Understanding of the soil structure interaction during load cycles.
- Understanding of strain compatibility when mixing old and new foundations

As stated earlier, this study looks at the behaviour of straight shafted piles in overconsolidated soils when subjected to load/un-load/re-load cycles and the influence of new foundations on the existing piles. No consideration is given to changes in the properties of the construction materials with time.

The soil and its variability are the most important factors in reuse of foundations. The re-used foundations will behave differently during loading depending on the stress history to which the soil, where these foundations were installed, was subjected. All soils are basically frictional materials with strength being provided by the friction resistance between soil particles. During construction processes soil will be loaded and unloaded. As the soils are loaded and unloaded they will compress and swell. This will lead to volume changes in soil, which involve rearrangement of the soil grains and seepage of water, thus changing the effective stress around the foundations and influencing their performance during loading.

When modelling the behaviour of the single pile foundations during loading/un-loading/re-loading cycles and the behaviour of the novel pile group foundations, attempts were made to model the performance of an overall prototype scale scenario when foundations are considered to be re-used. For typical centrifuge model test geometry see Figure 5.1. The model piles were installed at 1g due to simplicity of model preparation. Any changes in the soil surface during testing were measured using an LVDT (see Figure 5.2) located between the two model pile foundations tested (see Figures 5.3).

After equilibrium pore pressures were reached the pile foundations were load tested. In Figure 5.4 pore pressure responses during loading are shown. There were no changes measured during loading of the pile foundations on the pore pressure transducers in the soil mass. However, the pore pressure transducers at the base of the model piles do react during foundation loading, showing that the excess pore pressures generated dissipate very quickly and that pore pressure changes are very local. After the first loading cycle the centrifuge was stopped. One or both existing piles were enhanced with a group of mini-piles (see Figure 5.5), and the influence of the geometry of the mini-pile group on the performance of the existing pile during re-loading was investigated.

The outline, as referred to in Chapter 4, is used to explore trends in results.

5.2 The behaviour of single pile during load-unload-reload cycles

Modelling a field situation will always require some idealisation. The model piles were straight shafted smooth aluminium rods (Figure 5.6), designed to represent 12m long piles with a 0.6m diameter of prototype scale (see Table 5.1). The capacity of the model piles was calculated using the same method as shown in section 2.2.1.

Soil behaviour is a direct function of past stress history, together with the recent and anticipated stress path. Various relationships have been proposed by Skempton (1957), Bjerrum (1973) and Lerouil et al. (1985) to link the undrained shear strength and effective vertical stress in one dimensional normal compression via peak values obtained from field vane shear tests. By using the Bjerrum‘s factor, μ, the following relationship was suggested by Muir Wood (1990):-

$$\mu S_u / \sigma'_v = 0.22 \qquad 5.1$$

When allowance is made for overconsolidated ratio, then it has been found by Nunez (1989), Phillips (1987) and Springman (1989) as part of their research carried out at Cambridge University, that for the current effective vertical stress:-

$$S_u / \sigma'_v = aOCR^b \qquad 5.2$$

Garnier (2002) suggested, for Speswhite Kaolin clay, the following relationship between undrained shear strength, overconsolidation ratio and vertical effective stress:-

$$S_u = 0.19\ \sigma'_v\ (OCR)^{0.59} \qquad 5.3$$

Springman (1989) proposed the following relationship which represents the mean value obtained from a series of vane shear tests conducted in-flight in the centrifuge:-

$$S_u = 0.22\ \sigma'_v\ (OCR)^{0.706} \qquad 5.4$$

Using Equations 5.3 and 5.4 the distributions of undrained shear strength, after equilibrium of the centrifuge model was reached at 60g, were calculated and are shown in Figure 5.7. Also included in Figure 5.7 is the relationship between undrained shear strength with overconsolidation ratio and vertical effective stress as suggested by Stewart (1989), equation 3.20. For the purpose of this research the undrained shear strength was estimated based on the findings by Springman (1989).

The initial assumptions on the value of the adhesion factor, α, were too high, thus giving a higher calculated ultimate load for the piles, compared to that obtained from the centrifuge model tests. The value of the empirical adhesion factor, α, depends on a number of factors (Patel, 1992), such as:-

- strength stiffness and plasticity of clay
- the size and type of pile
- method of pile installation

Side friction is a measure of shear strength of the bond between the material of the pile and the soil mass. The ultimate skin friction of pile shafts is related to the horizontal effective stress acting on the shaft and the effective angle of friction between the pile and the clay. When a pile is extracted from fine grained soil, a thin layer of soil is invariably adhered to the pile shaft. This indicates that the actual skin friction is greater than the shear strength of the soil and that before full skin friction is mobilized, settlement of the pile is the result of shear deformation of the surrounding soil.

In the centrifuge testing, when the model piles were extracted from the soil, there was no layer of soil adhered to the pile shaft. This was also confirmed during shear box testing. When the shear box model was pulled apart, there was no clay adhering to the surface of the test plate. Thus the shear strength of the soil, in the centrifuge model testing, was greater than the skin friction between the pile and the clay, which explains the low values obtained for the adhesion factor α (α = 1.12).

The vertical displacement of the piles during testing was measured using two LVDTs, and the average reading was used for the analysis of the test data (Figure 5.8). The load applied to the model piles was measured using a load cell (Figure 5.9), which was connected at the loading pin.

Single pile foundations were tested for up to three loading cycles. In tests LQ5, LQ6, LQ7 and LQ13 the centrifuge was stopped between the loading cycles (see Table 5.2). In tests LQ18 and LQ19 the centrifuge was not stopped between the first and second loading cycles (see Table 5.3). From the above tests it was observed that the single pile foundation initially loaded up to ultimate load (foundation displacement due to loading up to 10% of the pile base diameter), when reloaded showed an increase in capacity of 20% (see Figure 5.10). The single pile foundations that were initially loaded to working load, when reloaded to failure reached a lower ultimate load by 15% compared to the piles loaded to failure for the first time (Figure 5.11).

Pile foundations in tests LQ5 and LQ6 were subjected to a third loading cycle immediately after the second load cycle (see Figure 5.12). The capacity of the model pile foundations increased compared to the second loading cycle.

For all the above tests, the piles were loaded for a period of 10min to represent a pile subjected to loading while the building is still standing. The same loading procedure was adopted for the second and third loading cycles.

In all the above tests pile foundations showed an increase in stiffness during second loading (i.e. foundations experienced lower displacements for higher loads). When foundations were subjected to the third loading cycle (LQ5 and LQ6) the pile foundation stiffness was lower compared to the performance during the second loading cycle, as the piles were re-tested prior to equilibrium pore pressures being reached.

During the increase and decrease of the pore pressure on the clay sample throughout preparation of the model and during testing, the vertical and horizontal effective stresses (σ'_v and σ'_h) are continually changing. The horizontal effective stress is stress history dependent and is calculated from the coefficient of earth pressure at rest (K_o) and vertical effective stress:-

$$\sigma'_h = K_o \, \sigma'_v \qquad 5.5$$

For normally consolidated deposits the coefficient of earth pressure at rest (K_{onc}) is given by (Mayne and Kulhawy, 1982) as:-

$$K_o = 1 - \sin\phi' \qquad 5.6$$

Where: ϕ' – the angle of friction

When the normally consolidated deposits are unloaded the ratio of horizontal and vertical effective stresses (σ'_h / σ'_v) changes. The way that the earth pressure coefficient changes as a result of variation in vertical effective stresses is relatively complex. The influence of the stress history was described by Burland et al. (1979) and Mayne and Kulhawy (1982) by way of similar diagrammatic representations. Mayne and Kulhaway (1982) put together a data from over 170 different soils and concluded that for overconsolidated clays:-

$$K_o = (1 - \sin\phi')\,(OCR)^{\sin\phi'} \qquad 5.7$$

Where: ϕ' is the friction angle and OCR is the overconsolidation ratio.

Al-Tabbaa (1987) investigated the behaviour of Speswhite Kaolin using an instrumented odometer and found that:-

$$K_o = 0.69\,(OCR)^{0.46} \qquad 5.8$$

Using the equation 5.3 the distribution of the horizontal stresses in the centrifuge sample can then be calculated.

Hence, the initial loading conditions influence foundation performance during reloading and the behaviour of pile foundations on the overconsolidated clay is dependent on the stress history to which the soil is subjected.

5.3 Effect of new mini-piles group on the existing pile foundation

When the capacity of the existing pile is not sufficient for the new development, the capacity may be improved if a ring of sacrificial mini-piles is installed around it. The influence of these new foundations on the performance of the existing pile was investigated by changing the geometry of the group. The number of the mini-piles, the length and the spacing between the existing pile and the mini-piles were all varied (see Table 5.4). It was observed that the capacity of a single pile belonging to a group is different from that of an isolated single pile due to the confinement offered by the surrounding piles.

5.3.1 The influence of the centre to centre distance between the existing pile and mini-piles on the existing pile capacity

When looking at the influence of spacing between the centre pile and the mini-pile group on the performance of the existing pile foundation both 100mm and 200mm long mini-piles were investigated (see Table 5.4). The increase in capacity observed, when using a group of eight mini-piles, for the 1.5D spacing was lower compared to the 2D spacing by 10% and 15% for the 100mm and 200mm long mini-piles respectively (see Figures 5.13 and 5.14). When comparing with the single pile foundation subjected to load-unload cycles the performance of the existing pile was improved by the sacrificial piles in a range from 17% to 60% (see Figure 5.15).

Figures 5.16 and 5.17 show a plan view of the geometry of the novel pile groups, when the centre piles were enhanced using 100mm and 200mm long mini-piles at 1.5D and 2D spacing. The installation of the mini-pile group confines the soil around the existing pile, thus an increase in capacity was observed on re-testing the existing pile. The effective geometry of the enhanced centre pile observed from the centrifuge model tests is shown in Figures 5.18 and 5.19 for 100mm long mini-piles. No contribution from the mini-pile group was considered when back calculating below the toe level of the mini-pile group as the foundations tested were on clay soils (i.e. main contribution to pile capacity is from the shaft friction).

The effect on the performance of the centre pile of the 200mm long mini-piles up to 100mm depth (i.e. the length of previously described model mini-piles) was considered to be the same as for the 100mm long mini-piles. The behaviour of the enhanced centre pile during centrifuge model testing is shown in Figures 5.20 and 5.21 for 200mm long mini-piles.

It would be expected that the increase in capacity would be higher when the mini-pile group is installed closer to the centre pile. Installation of the mini-pile group closer to the existing pile disturbs soil more in close proximity to the centre pile compared to the mini-piles installed further from the centre pile. The increase in pore pressures, due to mini-pile installation, causes the reduction of the effective stresses around these new pile foundations. When the mini-pile groups were installed closer to the existing pile this change in effective stress has a greater effect on the behaviour of the existing pile when re-tested compared to the influence of the mini-pile group spaced further away from the existing centre pile.

The pore pressure transducers were located at different heights at the centre of the soil model, thus the soil model was far less disturbed due to mini-pile installation in the proximity of the pore pressure transducers. Although the existing pile was retested after pore pressure equilibrium was reached in the soil model, there is no information available to determine the effective stresses around the existing pile, as no pore pressure transducer could be installed next to the pile shaft. If the existing piles were retested after a longer period of time of the model spinning in the centrifuge, than it would be expected that the closer spaced mini-pile group would improve the existing pile foundation more as the excess pore pressures would have dissipated to the equilibrium state and the effective stresses would have increased.

The centrifuge tests showed that by increasing the centre to centre spacing between the centre pile and the mini-pile group, the effective diameter of the centre pile increased by approximately the same percentage as the pile spacing.

5.3.2 The influence of the number of mini-piles on the existing pile capacity

The number of the mini-piles in the group also influenced the performance of the existing pile when reloading. For 2D centre to centre spacing between the existing pile and the mini-piles, groups of eight and sixteen mini-piles were investigated as shown in Figure 5.22. It was observed that for both 100mm and 200mm long mini-piles the group of eight mini-piles suggested a higher capacity compared to a group of sixteen mini-piles by 10% (see Figure 5.23). The performance of the existing pile was enhanced by 17% to 60% when comparing with the performance of a single pile during reloading (see Figure 5.24).

The effective diameter of the enhanced pile foundations with eight and sixteen mini-piles is shown in Figure 5.25. Mini-pile installation will change the stress conditions around the existing pile. By increasing the number of the mini-piles in the group the change in the stress conditions around the existing pile will be more significant. Also as the spacing between the existing centre pile and mini-piles in the group remains the same, the spacing between the mini-piles within the group will reduce as the number of the mini-piles increases (see Figure 5.26).

The existing centre pile was re-loaded after the pore pressure transducers in the soil mass and at the base of the model piles reached equilibrium stresses. In all tests there was no reaction observed on the pore pressure transducers installed in the soil mass during foundations loading. Thus, the equilibrium readings of the pore pressure transducers in the soil mass do not represent the stresses in the soil surrounding the centre pile. If the existing model pile was tested after the excess pore pressures have fully dissipated, it would be expected that the existing pile would reach a higher load capacity when the number of the mini-piles in the group has increased.

5.3.3 The influence of the length of mini-piles on the existing pile capacity

Not surprisingly the length of the mini-piles has been shown to play an important part on the performance of the enhanced pile foundation. In all the geometries tested 200mm long mini-piles performed better compared to the 100mm long mini-piles by 20% (see Figures 5.27).

The sacrificial mini-piles increase the stiffness of the soil, thus improving the capacity of the existing pile when re-loading. As the existing pile is loaded, this load is distributed along the length of the pile until the soil strength is fully mobilised. Thus by increasing the length of the mini-piles the performance of the existing centre pile is enhanced as shown in Figure 5.28.

The increase in capacity in the top sections of the model piles (i.e. the top 100mm of the embedded length) is the same for 100mm and 200mm long mini-piles as the number and spacing of mini-piles in the group was the same. The 200mm long mini-piles influence the performance of the centre pile along the whole length of the pile, thus improving the performance of the existing centre pile by 20% more compared to 100mm long mini-piles.

5.4 Caisson effect – Loading mini-pile group only

In section 5.3 it is shown how the geometry of the mini-pile group influences the performance of the previously loaded centre pile. In this section the influence of the preloaded centre pile on the performance of the mini-pile group will be investigated. For all tests undertaken the pile groups incorporated preloaded existing piles at the centre of the group thus these groups will be referred to from now on as ‘novel pile groups’. Although the length, number and spacing of the mini-piles tested was varied (see Table 5.4) the testing procedure was the same for all tests. The geometry of the central preloaded pile was also not varied (see Figure 5.29).

For all tests the centre pile was initially load tested, the centrifuge was then stopped and the mini-piles were installed at 1g. The centrifuge was re-started and after the equilibrium pore pressures were reached, the mini-pile groups were loaded. The above described geometry (i.e. with preloaded centre pile) of the novel pile groups are generally not practiced in the construction industry. Piles are normally constructed in groups of vertical, raking or a combination of vertical and raking piles. The design methodology adopted when the pile group is subjected to vertical load, should provide calculations of the group capacity and displacement such that the forces are in equilibrium between the structure and the supporting piles. The ultimate capacity of pile groups in clay soil is the lesser of the sum of the capacities of the individual piles or the capacity by block failure.

Due to limitations of the loading apparatus the groups of eight and sixteen mini-piles investigated could not be loaded to failure. Thus, all of the analyses of the novel pile group performance were based on considering the mini-pile group performance as a block and at pile displacement of 0.1mm due to loading. It was observed that the pile groups with 2D spacing from the centre pile performed better compared to the groups with 3D spacing, where D is the diameter of the existing centre pile (see Figure 5.30). The novel pile groups tested were of four and eight mini-piles. The capacity obtained from centrifuge model tests at 0.1mm displacement for a novel pile group of four, at 3D spacing from the centre pile, was equivalent to the performance of a single pile with diameter D (diameter of the existing centre pile). For the spacing of 2D the group performance was equivalent to a pile with diameter of 1.2D (see Figure 5.31). The performance of novel groups of eight mini-pile with 3D and 2D spacing from the centre pile at 0.1mm displacement was equivalent to a single pile with a 2D and 2.2D diameter respectively (see Figure 5.32). The difference in the increase in capacity in groups of four and eight mini-piles, when the spacing from the centre pile has changed is due to the fact that the mini-piles within the group have different spacing between each other. Thus the results as expected are dependant on the mini-pile to mini-pile interaction.

Tests are performed on small diameter mini-piles in overconsolidated clay, thus the shaft capacity is the main parameter influencing the performance of the group. For the same diameter and number of mini-piles, the group consisting of longer mini-piles could be expected to perform better compared to the group with short mini-piles. As the test results for 100mm long mini-piles were abandoned, there were no results available to make a direct comparison between the performance of 100mm and 200mm long mini-piles.

By increasing the number of mini-piles in the group, the capacity of the group was, as expected, higher (see Figure 5.33). The performance of the novel pile groups was compared at 0.1mm displacement due to loading of the model piles, as the capacity of the loading reservoirs was not sufficient to load test the novel pile groups to failure. When considering the performance as a block failure, it was observed that the performance of the novel pile group of four mini-piles at 0.1mm displacement was equivalent to the performance of a single pile with 1.2D, where D equals the diameter of the centre pile (see Figure 5.33(a)). The capacity of the novel pile group of eight mini-piles at 0.1mm displacement was equivalent to a pile of 2.2D (D is the diameter of the centre pile), as shown in Figure 5.33(b). When the centre to centre spacing from the centre pile was increased to 3D, the performance of novel pile groups of four and eight mini-piles at 0.1mm displacement was equivalent to the performance of a single pile with a diameter of D and 2D respectively. As explained earlier the performance of the novel pile groups was also dependent on the spacing of the mini-piles within the group.

The basic principle of the performance of the novel pile groups were similar to the groups consisting of the piles constructed at the same time and usually with equal diameters. As the available capacity of the loading apparatus was insufficient to load the pile group to failure and the number of tests undertaken was very limited the performance of the novel pile groups needs further investigation.

5.5 Novel pile group – Loading existing and new foundations

The performance of the existing piles when loaded together with the new mini-pile foundation group was also investigated. Due to limitations of the capacity of the loading apparatus only the group of eight mini-piles was considered for these analyses. There are no results for any other novel group geometry with which they can be compared. The group investigated consisted of a preloaded centre pile and eight 100mm long mini-piles. The centre to centre spacing between the existing pile and the mini-piles in the group was 3D. For all tests considered (LQ7, LQ18 and LQ14) the single piles were installed and tested to working load. The centrifuge was then stopped. Only the foundations in tests LQ18 and LQ14 were enhanced with the same geometry mini-pile group (eight 100mm long mini-piles with 3D spacing). The foundations were then re-tested. In test LQ14 the existing pile foundation was reloaded together with the pile group. For details of the tests LQ7, LQ14 and LQ18 see Table 5.5.

During the first loading, foundation behaviour in tests LQ7 and LQ18 was similar. The model pile in test LQ14 displaced more for the same applied load, compared to the model piles in tests LQ7 and LQ18. As expected, during the second loading, the enhanced pile (LQ18) performed better compared to a single pile foundation and when loading the existing pile together with the pile group the capacity obtained from the novel group was even higher (see Figure 5.34). The capacity of the group when loading the existing pile together with the mini-pile group was higher than the sum of the individual elements of the group at 1mm displacement.

As the geometries investigated in a pile group consisted of elements installed at different times, with a different geometry and subjected to different loading conditions, the question immediately arises as to what comparative order of settlement should be used to define failure. In this situation the foundations should be designed in such a way that the settlements, and particularly the differential settlements of the structure, remain within tolerable limits.

5.6 Consideration of the test results with field monitoring data

The behaviour of the centrifuge model tests was categorised with the observation made on the tests performed by Cementation Skanska as part of the RUFUS project (Trials at Chattenden). As part of the trials at Chattenden the performance of the single piles subjected to repeated loading and the influence of the new mini-pile foundations on the performance of the existing pile were investigated. The site and the centrifuge model possess some elements that are geometrically similar as shown in Figure 5.35; although during centrifuge testing there was no influence on the foundation performance from the pile cap as the cap was modelled above the clay surface (see Figure 5.36).

The field test showed an increase in capacity of the single piles of around 30% at a displacement of 10% of the pile diameter when re-tested after two years. In the centrifuge tests an increase in capacity of 20% was observed when re-testing single piles. This seems reasonable owing to the much greater soil stiffness in the field compared to the centrifuge model therefore re-loading might have a relatively greater effect. It should be noted that during the first loading in the field, both single piles tested, underperformed in comparison with theory as did the centrifuge model piles. It is believed that this was due to problems with the pile installation (Fernie et al, 2006).

The measured group resistance for field test T40 was approximately the same with the theoretical resistance. The pile groups tested had a different configuration compared to centrifuge model. It would have been preferable to have the same pile group geometry to facilitate better comparison between field trials and centrifuge model testing. The pile group tested at Chattenden were tied together with a 1m deep cap and only the influence of the number of the mini-piles in the group was investigated. After the single piles were subjected to a second loading cycle, six mini-piles were installed around one of the piles (pile T40) and four mini-piles were installed around the other (T44) with the same spacing and overall depth. At the same time the grouped ring of six and eight mini-piles were also installed and load tested.

The pile groups were loaded tested in three stages:

- With the 4mm displacement case being chosen as the point at which the individual mini-piles reached their capacity.

- The 10mm displacement as a reasonable settlement limit for a group generally.

- Loaded to ultimate (failure).

The groups (T7/T8 and T9/T10) reached their ultimate capacity at about 10mm displacement while the mini-pile groups supplementing an existing pile had one to two further load stages applied before the ultimate condition was reached.

As stated earlier, the geometry of the groups tested during field trial was different compared to centrifuge model geometry, however, the results from both field data and centrifuge model testing show a good correlation when predicting the performance of the novel pile groups.

5.7 Summary

The model testing has enabled a clear understanding of the model behaviour. A good quality of data has been acquired from a series of tests in which a number of geometrical parameters were varied to enable a better understanding of pile performance when these foundations are re-used.

The results of the centrifuge tests have been compared and discussed and the reasons for the observed behaviour are explained and justified. The consistency in data has been assessed. The limitations of the testing procedure and the testing apparatus have been explained and considered especially where this has resulted in incomplete test data.
Although the testing apparatus capacity was insufficient to load the pile groups to failure it did provide sufficient consistency to confirm that the existing pile foundations can be reused for new development, provided the foundations are in good condition, and it has been established that the capacity of these foundations can be improved using mini-pile groups.

In tests undertaken it was observed that the capacity of the existing foundations increased by up to 20% during reloading, and that this increase in capacity was dependant on the stress history to which the foundation was subjected.

The mini-pile groups have been found to influence the performance of the existing foundations. By varying the geometry of the group the range of this influence has been assessed. Although it is expected that when the pile group is closely spaced it will behave as a single large diameter pile, the results showed that the mini-pile groups appear to improve the capacity of the existing piles less when they are more closely spaced. The piles were re-tested after the equilibrium pore-pressures had been reached, but there were no pore-pressure transducers placed within the group, thus it is believed that the disturbance of the soil in the vicinity of the existing piles led to the observations that were made. Chandler et al. (1982) stated that the difference between the adhesion factors (α) back calculated from the model tests and those normally encountered in practice highlight the effects of disturbance caused by most pile installation techniques. The same phenomenon was observed when the number of the mini-piles in the group was increased. Due to installation of more mini-piles within the group, the ground around the existing pile was disturbed leading to less increase in capacity compared to when less mini-piles were used with the same length and distance from the centre of the existing pile.

The length of the mini-piles was also found to be an important factor affecting the behaviour of the existing piles when reloaded. Longer mini-piles improved the capacity of the existing piles more compared to shorter mini-piles. The longer mini-piles appear to provide a stiffening effect along the length of the existing piles, thus leading to the higher capacities observed after mini-pile group installation.

The testing of the mini-pile groups was not up to failure load as the capacity of the loading apparatus was not sufficient to reach these loads for the groups tested. The behaviour of novel pile groups was investigated in terms of the spacing, number and length of the mini-piles in the group. Prior to mini-pile group installation and testing, a single pile was installed at the centre of the group and load tested to failure.

The performance of the mini-pile group was in agreement with current information available for the performance of mini-pile groups. Closely spaced groups (2D spacing) performed better compared with groups with the same number and length of mini-piles but with a spacing of 3D from the existing centre pile. The shaft capacity is proven to be the main contributor of the performance of the mini-piles in overconsolidated clay soils thus longer mini-piles do show a higher capacity compared to short mini-pile groups with same number and spacing. Finally, by increasing the number of the mini-piles in the group the overall capacity of the group was increased.

As stated in earlier chapters the ground is becoming more congested with the every new development. By reusing the existing foundations the time, cost of construction and the impact on the environment will be reduced.

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS FOR FURTHER WORK

6.1 Introduction

There are obvious advantages for redevelopment if as much as possible of the existing building foundations can be reused to reduce the environmental impact, time and cost of the construction. The research undertaken is an investigation into behaviour of bored piles in overconsolidated clay when subjected to load cycles and foundation improvement using new mini-pile group with the view of the re-use of the existing piles for future redevelopments.

The work presented here investigates aspects of performance of single pile foundations subjected to load/unload/reload cycles. If existing piles are to be re-used, it is necessary to first understand the behaviour of pile foundations when subjected to load cycles, so that a decision can be made on the magnitude of the load to which the existing piles can be re-loaded. An increase in capacity was observed when pile foundations were subjected to load cycles.

In the case that the existing piles had insufficient capacity for the new development, the research also sought to explore if the capacity of an the existing pile could be improved by placing around it a ring of new mini-pile foundations. The new mini-pile group was constructed around the existing pile that had previously been subjected to its working or failure load. The geometry of the group was varied, i.e. the number of the mini-piles, centre to centre distance between the existing and new pile foundation and length of these new foundations.

In this chapter the experimental approach and main findings of the work are summarised. During the course of this research a number of avenues of exploration have been pursued and recommendations for further work are made.

6.2 Experimental procedure

A total of twenty one tests were carried out using the geotechnical centrifuge at City University, London. In each test two piles of 12m deep, in prototype scale, were tested. Initially same apparatus developed by Morrison (1994) was used (see Figure 6.1). As the time required for the preparation of the model for testing was too long a new apparatus was designed and manufactured (Figure 6.2). The design for the new apparatus was such that most could be put together prior to testing. The pile foundations were loaded using plastic reservoirs with the facility of filling and emptying water in flight. The reservoirs rested on a spring with a sufficient stiffness to support the weight of an empty reservoir. Thus the foundation loading was performed in a controlled manner with no load applied to the pile during consolidation. The load was applied to the pile head using a loading pin connected at the base of the plastic reservoir and the load was measured using a load cell connected to the loading pin. The LVDT support was manufactured as an independent part of the loading apparatus and designed such that it could be easily adjusted depending on the height of the model using four threaded rods to connect to the flange of the tub.

Comparison was made between the behaviour of foundations during first loading to the behaviour during the second and third loading cycle. The influence of the mini-pile groups was also investigated in several tests in which the number, spacing and the length of the mini-piles in the group were varied. Mini-piles in groups of four, eight and sixteen piles with 6m and 12m length of prototype scale were investigated. The centre to centre distance between the mini-piles and the existing centre pile of 1.5D, 2D and 3D (where D is the diameter of the centre pile) were investigated.

The centrifuge models were made from overconsolidated Speswhite kaolin clay prepared from slurry with 120% water content. The single 10mm diameter piles were installed at 1g (Figure 6.3). The model was then put together and placed on the swing (Figure 6.4). When an acceleration of 60g was reached, models were left to reach effective stress equilibrium prior to subjecting the piles to the first loading conditions. After the first load-unload cycle the centrifuge was stopped and one or both foundations were enhanced using mini-pile groups (Figure 6.5). The models were left to achieve conditions of effective stress equilibrium prior to retesting.

Miniature pore pressure transducers were used to monitor the changes of the pore pressure in the sample (Figure 6.6). The displacements of each foundation were measured by two LVDTs and the average reading was used for the analysis (Figure 6.7). The load cells used for testing are shown in Figure 6.8.

A broad literature review concerning reuse of pile foundations and foundation design has been carried out as part of this project. The influence of time, loading cycles and new foundations on the existing piles was considered. There are conflicting views on the behaviour of piles during reloading in overconsolidated clays. A great deal of literature focuses on case studies where existing pile foundations have been reused.

6.3 Conclusions

This project focuses on the performance of existing piles during reloading assuming no deterioration of the pile material has taken place. The improvement of the performance of the existing piles was investigated when using different geometry mini-pile groups. Centrifuge model testing and field monitoring data have illustrated the behaviour of the existing piles and the positive effect of the existing pile capacity improvement technique.

The pile foundation performance observed in the centrifuge model tests has been consistent thus allowing the following conclusions to be drawn:-

- When the single pile foundation, which was initially loaded to failure (foundation displacement of 10% of the pile base diameter), was reloaded the foundation showed an increase in capacity of about 20% (Figure 6.9).

- The single pile foundations which were initially loaded to working load, when reloaded to failure, reached a lower ultimate load by about 15% compared to the piles loaded to failure for the first time (Figure 6.10). Hence the initial loading conditions influence foundation performance during reloading.

All tests in which the existing pile foundations were enhanced with the sacrificial mini-pile group, were compared to the performance of a single pile foundation when subjected to the same loading conditions. In all the cases presented the introduction of mini-pile groups around the existing pile had a positive effect on the performance of the existing pile during reloading. This improvement in performance of the existing pile during reloading was dependent on the number of the mini-piles used in the group, the length of the mini-piles and also the centre to centre spacing between the existing pile and mini-pile foundations.

When considering the influence of spacing between the centre pile and the mini-pile group on the performance of the existing pile foundation, both 100mm and 200mm long mini-piles were investigated. The increase in capacity observed, when using a group of eight mini-piles, for the 1.5D spacing was lower compared to the 2D spacing by 10% and 15% for the 100mm and 200mm long mini-piles respectively (Figures 6.11 and 6.12).

The number of the sacrificial mini-piles in the group also influenced the performance of the existing pile when reloading. Mini-piles groups of eight and sixteen, with 2D centre to centre spacing between the existing pile and mini-pile group, were investigated. It was observed that for both 100mm and 200mm long mini-piles the group of eight mini-piles resulted in higher group capacity compared to a group of sixteen mini-piles by 10% (Figures 6.13 and 6.14).

When comparing with the single pile foundation subjected to load-unload cycles the performance of the existing pile was improved by the sacrificial piles in a range from 17% to 60%.

The length of the mini-piles has been shown to play an important part in the performance of the enhanced pile foundation. In all geometries tested 200mm long mini-piles performed better compared to 100mm long mini-piles by about 20% (Figure 6.15).

The sacrificial mini-piles provided an increase in capacity to the existing piles when reloading the existing pile only. The performance of novel pile groups was also investigated in terms of number, spacing and length of the mini-piles within the group. The geometry of these novel pile groups incorporated a preloaded existing pile at the centre of the group. Centrifuge testing showed that the groups with 2D spacing from the centre pile performed better than the groups with 3D spacing (Figure 6.16). The performance of 200mm long mini-piles was compared to the 100mm long mini-piles, see Figure 6.17. Finally by increasing the number in the group a higher capacity was achieved (Figure 6.18). Only groups of four and eight mini-piles were investigated in this series of experiments.

Reloading the existing pile together with the new pile foundation group was also investigated but only one scenario was considered. The group investigated consisted of a preloaded to working load centre pile enhanced with eight 100mm long mini-piles. The centre to centre spacing between the existing pile and the mini-piles in the group was 3D (Figure 6.19). The results were compared to the performance of the single pile, single pile surrounded by sacrificial mini-piles and a mini-pile group. The capacity of the group when loading the existing pile together with the mini-pile group was higher than the sum of the individual elements of the group at 1mm displacement (Figure 6.20).

6.4 Limitations and implications of these results

The difficulty posed by soil is that its behaviour is very much influenced by its stress state and stress history. Therefore soil properties will vary with depth and this variation needs to be reproduced whenever a stratum of soil is to be modelled. For the centrifuge model testing Speswhite Kaolin clay was used as a model ground. As kaolin clay has much higher permeability compared to London Clay, the time required for model preparation is considerably reduced.

The model piles used for the centrifuge testing were made from smooth aluminium rod (see Figure 6.6). From the centrifuge model tests and the shear box test it was concluded that the adhesion factor for the model piles was much lower compared to the values used in design of cast in situ concrete foundations in, for example, London Clay. It was initially intended to use resin model piles to achieve a more realistic representation of the soil / pile interface of concrete bored piles but this was abandoned owing to difficulties in obtaining consistent model piles. This method would also have proved difficult to incorporate pore pressure transducers into base of pile.

Other limitation of the work carried out was installation of bored pile foundations at 1g rather then during flight at the required scale factor. There were no attempts made to install pile foundations in flight thus there are no results to be compared to observe the effect of the method of installation on foundation performance. Mini-pile groups were also installed at 1g. After the existing piles were subjected to first loading the centrifuge was stopped and one or both existing piles were enhanced by different geometry mini-pile groups. It was however demonstrated that stopping and restarting the centrifuge after the first loading had no effect on the existing foundation performance when reloaded (Figure 6.21)

The re-use of pile foundations is a positive step towards improving sustainability in construction. This is because the cost and time of new development and the impact on the environment will be reduced. In general, piles showed increase in capacity when subjected to load-unload-reload cycles and also when the existing piles were surrounded by a sacrificial mini-pile group. When loading the existing piles together with a new mini-pile foundation group, the capacity obtained was higher than the sum of the individual elements of the novel pile group.

6.5 Recommendations for further research

The development of a new centrifuge modelling technique that would enable pile installation during flight would allow the prototype installation procedure to be followed more closely. This would also allow a more realistic investigation of bored pile behaviour during load-unload-reload cycles and the influence of the new foundations on the existing piles. While this is probably unfeasible for rotary bored piles it may be possible for Continuous Flight Auger (CFA) piles.

In the tests undertaken in this project the influence of only a few parameters on pile foundation behaviour has been investigated. When investigating the behaviour of single pile foundation or pile group, no contribution of the pile cap was considered. For completeness, the influence of the pile cap should be explored since it could provide significant increase in the overall capacity of the foundations.

As stated earlier the length of the mini-piles and centre to centre spacing between the existing pile and mini-pile group was varied. The influence of the diameter of the mini-piles on the existing foundation should also be investigated. When investigating the influence of the mini-pile groups on the performance of the existing pile during re-loading it was decided to use the diameter of the existing pile as a normalising factor. Therefore, by varying the diameter of the mini-piles in the group, the spacing of the mini-piles within the group will change if the normalising factor is still the diameter of the existing piles and is kept as a constant (Figure 6.22). By varying the diameter of the mini-piles in the group a better understanding of the "caission" effect and the influence on the performance of the existing pile when re-loading could be attained.

No investigations were carried out to explore the influence of the material integrity of foundation elements when these foundations are considered for re-use. Defects may occur during foundation construction or the life of the structure and these defects need to be investigated in order to determine their influence on foundation performance during reloading. The defects due to demolition need to be investigated as well. How does the pile respond to heave in unloading, and does the cracking due to heave effect the performance of pile foundations on reloading.

Continued monitoring of performance of pile foundations during the first loading and during the demolition of the existing structure would provide much needed additional data to enable the prediction of foundation performance during reloading.

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Quantity	Prototype wall	Model wall
length	1	1/N
self weight stress	1	1 at N gravities
stress x area	1	$1/N^2$
strain	1	1
curvature	1	N
Young's modulus (E)	1	1
The following are expressed per	metre length	1/N metre length
moment of inertia (I)	1	$1/N^4$
intensity of load	1	1/N
shear force	1	$1/N^2$
bending moment	1	$1/N^3$

Table 3.1 Scale factors for centrifuge tests on model diaphragm walls (after Powrie 1986).

Quantity	Example	Scale factor	Prototype	Model
No. of g	-	-	1	60
Length	Pile diameter	1/N	0.6m	0.01m
	Pile length		12m	0.2m
	Mini-pile diameter		0.3m	0.005m
	Mini-pile length		6m	0.1m

Table 3.2 Scale factors for centrifuge tests on model pile foundation.

Spring Design

Spring rate: The force required to produce unit deflection,F/d.

Outer dia	Coil dia	Inner dia	Wire dia	Spring index	Spring rate	Max defl	No coils	Max load	Shear Modulus
Do	D	Di	d	C	k	δmax	Nt	Pmax	G
D+d	(Do+Di)/2	D-d		D/d	(Gdexp4)/8Dexp3Na	Lo-Ls		k*δmax	E/(2(1+v)
35.8	31.74	27.68	4.06	7.81773399	11.03021039	61.812	9.8	681.7993646	81000

		Plain ends		Closed ends		Plain ends ground		Closed ends ground	
Pitch	p	(Lo-d)Na	646.212	(Lo-3d)/Na	7.412820513	Lo/(Na+1)	7.1428571	**(Lo-2d)/Na**	**11.98461538**
Free length	Lo	Na*p+d	70	Na*p+3d	70	(Na+1)p	70	**Na*p+2d**	**101.6**
Solid length	Ls	(Na+1)d	43.848	(Na+1)d	35.728	(Na+1)d	39.788	**(Na+2)d**	**39.788**
Activ coils	Na	Nt	9.8	Nt-2	7.8	Nt-1	8.8	**Nt-2**	**7.8**

Table 3.3. Details of spring design used to support the loading reservoir for centrifuge model testing.

Test	Pile	First Loading	Second Loading
LQ1	**A**	Single Pile	Single Pile
	B	Single Pile	8MP 3D 100mm
LQ2			New Apparatus
LQ3	**A**		
	B		
LQ4	**A**	Single Pile	Single Pile
	B	Single Pile	8MP 3D 100mm
LQ5	**A**	Single Pile	Single Pile
	B	Single Pile	8MP 3D 200mm
LQ6	**A**	Single Pile	Single Pile
	B	Single Pile	8MP 3D 200mm
LQ7	**A**	Single Pile	Single Pile
	B	Single Pile	8MP 3D 200mm
LQ8	**A**	Single Pile	8MP 3D 100mm
	B	Single Pile	16MP 3D 200mm
LQ9	**A**	Single Pile	8MP 1.5D 100mm
	B	Single Pile	16MP 3D 100mm
LQ10	**A**	Single Pile	16MP 2D 100mm
	B	Single Pile	8MP 1.5D 200mm
*** LQ11**	**A**	Single Pile	8MP 2D 100mm
	B	Single Pile	8MP 1.5D 200mm
*** LQ12**	**A**	Single Pile	8MP 2D 200mm
	B	Single Pile	16MP 2D 200mm
*** LQ13**	**A**	Single Pile	8MP 3D 100mm – Loading PG only
	B	Single Pile	Single Pile
LQ14	**A**	Single Pile	16MP 2D 200mm
	B	Single Pile	8MP 3D 100mm – Loading EP & PG

LQ15	**A**	Single Pile	8MP 3D 200mm – Loading PG
	B	Single Pile	8MP 2D 100mm – Loading PG
LQ16	**A**	Single Pile	8MP 2D 200mm – Loading PG
	B	Single Pile	16MP 2D 100mm – Loading PG
LQ17	**A**	Single Pile	4MP 2D 200mm – Loading PG
	B	Single Pile	4MP 3D 200mm – Loading PG
LQ18	**A**	Single Pile/ Single Pile	Driven - 8MP 1.5D 200mm
	B	Single Pile/ Single pile	8MP 3D 100mm
LQ19	**A**	Single Pile/ Single Pile	4MP 3D 200mm – Loading EP+PG
	B	Single Pile/ Single pile	8MP 2D 100mm
LQ20	**A**	Single Pile	4MP 2D 200mm – Loading EP+PG
	B	Single Pile	4MP 3D 200mm – Loading EP+PG
LQ21	**A**	Single Pile	4MP 2D 200mm – Loading EP+PG
	B	Single Pile	4MP 3D 200mm – Loading EP+PG

* Piles installed after the sample was accelerated to 60g for 12h

Table 3.4 Details of tests conducted.

Test	**Pile**	**First Loading**	**Second Loading**
LQ1	**A**	Single Pile	Single Pile
	B	Single Pile	8MP 3D 100mm
LQ2			New Apparatus
LQ3	**A**		
	B		
LQ4	**A**	Single Pile	Single Pile
	B	Single Pile	8MP 3D 100mm
LQ5	**A**	Single Pile	Single Pile
	B	Single Pile	8MP 3D 200mm
LQ6	**A**	Single Pile	Single Pile
	B	Single Pile	8MP 3D 200mm
LQ7	**A**	Single Pile	Single Pile
	B	Single Pile	8MP 3D 200mm
LQ8	**A**	Single Pile	8MP 3D 100mm
	B	Single Pile	16MP 3D 200mm
LQ9	**A**	Single Pile	8MP 1.5D 100mm
	B	Single Pile	16MP 3D 100mm
LQ10	**A**	Single Pile	16MP 2D 100mm
	B	Single Pile	8MP 1.5D 200mm
*** LQ11**	**A**	Single Pile	8MP 2D 100mm
	B	Single Pile	8MP 1.5D 200mm
*** LQ12**	**A**	Single Pile	8MP 2D 200mm
	B	Single Pile	16MP 2D 200mm
*** LQ13**	**A**	Single Pile	8MP 3D 100mm – Loading PG only
	B	Single Pile	Single Pile
LQ14	**A**	Single Pile	16MP 2D 200mm
	B	Single Pile	8MP 3D 100mm – Loading EP & PG

LQ15	**A**	Single Pile	8MP 3D 200mm – Loading PG
	B	Single Pile	8MP 2D 100mm – Loading PG
LQ16	**A**	Single Pile	8MP 2D 200mm – Loading PG
	B	Single Pile	16MP 2D 100mm – Loading PG
LQ17	**A**	Single Pile	4MP 2D 200mm – Loading PG
	B	Single Pile	4MP 3D 200mm – Loading PG
LQ18	**A**	Single Pile/ Single Pile	Driven - 8MP 1.5D 200mm
	B	Single Pile/ Single pile	8MP 3D 100mm
LQ19	**A**	Single Pile/ Single Pile	4MP 3D 200mm – Loading EP+PG
	B	Single Pile/ Single pile	8MP 2D 100mm
LQ20	**A**	Single Pile	4MP 2D 200mm – Loading EP+PG
	B	Single Pile	4MP 3D 200mm – Loading EP+PG
LQ21	**A**	Single Pile	4MP 2D 200mm – Loading EP+PG
	B	Single Pile	4MP 3D 200mm – Loading EP+PG

* Denotes piles installed after the sample was spun up at 60g for 12h

Table 4.1 Details of the centrifuge tests conducted.

Pile A

Test	1st loading		2nd loading		3rd loading	Geometry after the first Loading
Test 1	65		110		60	Single Pile
Test 2	Sand					
Test 3						
Test 4	70		81		90	Single Pile
Test 5	110		130		135	Single Pile
Test 6	115		140		150	Single Pile
Test 7	50		85			Single Pile
Test 8	90		140			8MP L=100mm
Test 9	50		103			8MP L=100mm cc=1.5D
Test 10	50		100			16MP L=100mm cc=2D
Test 11	50		113			8MP L=100mm cc=2D
Test 12	50		138			8MP L=200mm cc=2D
Test 13	50		111			8MP L=100mm cc=3D – PG
Test 14	30		73			16MP L=200mm cc=2D
Test 15	50		363			8MP L=200mm cc=3D – PG
Test 16	50		390			8MP L=200mm cc=2D – PG
Test 17	50		162			4MP L=200mm cc=2D – PG
Test 18	50		50		120	8MP(Driven) L=100mm cc=1.5D
Test 19	94		135		318	4MP L=200mm cc=3D - EP+PG
Test 20	-		-		-	4MP L=200mm cc=2D - EP+PG
Test 21	86		347			4MP L=200mm cc=2D - EP+PG

Table 4.2a Details of the centrifuge tests conducted and maximum loads applied - Pile A.

Pile B

Test	1st loading		2nd loading		3rd loading	Geometry after the first Loading
Test 1	65		85		90	8MP L=100mm
Test 2	Sand					
Test 3						
Test 4	110		120		135	8MP L=100mm
Test 5	100		128		138	8MP L=200mm
Test 6	185		190		205	8MP L=200mm
Test 7	50		130			8MP L=200mm
Test 8	100		95			16MP L=200mm
Test 9	50		93			16MP L=100mm cc=3D
Test 10	50		119			8MP L=200mm cc=1.5D
Test 11	50		82			8MP L=200mm cc=1.5D
Test 12	50		122			16MP L=200mm cc=2D
Test 13	50		88			Single Pile
Test 14	50		345			8MP L=100mm cc=3D – EP & PG
Test 15	50		73			8MP L=100mm cc=2D - PG
Test 16	50		430			16MP L=100mm cc=2D - PG
Test 17	70		130			4MP L=200mm cc=3D - PG
Test 18	50		50		127	8MP L=100mm cc=3D
Test 19	116		140		169	8MP L=100mm cc=2D
Test 20	-		-		-	4MP L=200mm cc=3D - EP+PG
Test 21	55		382			4MP L=200mm cc=3D - EP+PG

Table 4.2b Details of the centrifuge tests conducted and maximum loads applied - Pile A.

Quantity	Example	Scale factor	Prototype	Model
No. of g	-	-	1	60
Length	Pile diameter Pile length Mini-pile diameter Mini-pile length	1/N	0.6m 12m 0.3m 6m	0.01m 0.2m 0.005m 0.1m

Table 5.1 Scale factors for centrifuge tests on model pile foundation.

Test	1st loading		2nd loading		3rd loading
LQ5(A)	110		130		135
LQ6(A)	115		140		150
LQ7(A)	50		85		
LQ13(B)	50		88		

Table 5.2 Details of the centrifuge tests conducted on single piles.
Centrifuge was stopped between loading cycles.

Test	1st loading		2nd loading
LQ18(A)	50		50
LQ18(B)	50		50
LQ19(A)	94		135
LQ19(B)	116		140

Table 5.3 Details of the centrifuge tests conducted on single piles. Centrifuge was not stopped between loading cycles.

Pile A

Test	1st loading	2nd loading	3rd loading	Geometry after the first Loading
Test 8	90	140		8MP L=100mm
Test 9	50	103		8MP L=100mm cc=1.5D
Test 10	50	100		16MP L=100mm cc=2D
Test 11	50	113		8MP L=100mm cc=2D
Test 12	50	138		8MP L=200mm cc=2D
Test 14	30	73		16MP L=200mm cc=2D
Test 18	50	50	120	8MP(Driven) L=100mm cc=1.5D

Pile B

Test	1st loading	2nd loading	3rd loading	Geometry after the first Loading
Test 4	110	120	135	8MP L=100mm
Test 5	100	128	138	8MP L=200mm
Test 6	185	190	205	8MP L=200mm
Test 7	50	130		8MP L=200mm
Test 8	100	95		16MP L=200mm
Test 9	50	93		16MP L=100mm cc=3D
Test 10	50	119		8MP L=200mm cc=1.5D
Test 11	50	82		8MP L=200mm cc=1.5D
Test 12	50	122		16MP L=200mm cc=2D
Test 18	50	50	127	8MP L=100mm cc=3D
Test 19	116	140	169	8MP L=100mm cc=2D

Table 5.4 Details of the centrifuge tests conducted and maximum loads applied. Novel pile groups.

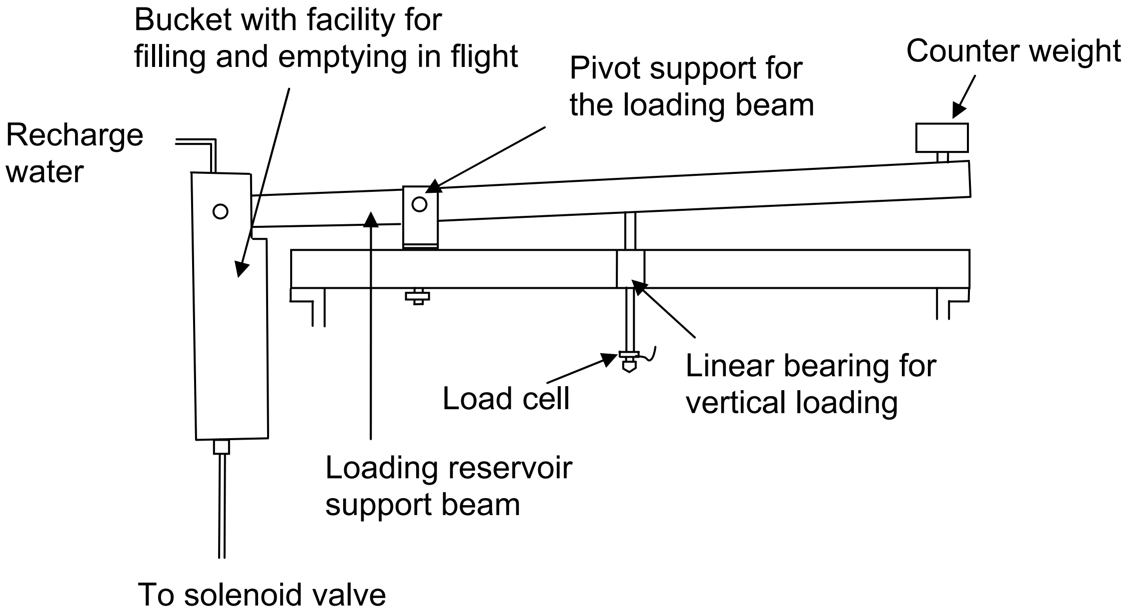


Figure 1.1 Axial loading pile foundation device used in centrifuge model testing (Morrison, 1994).

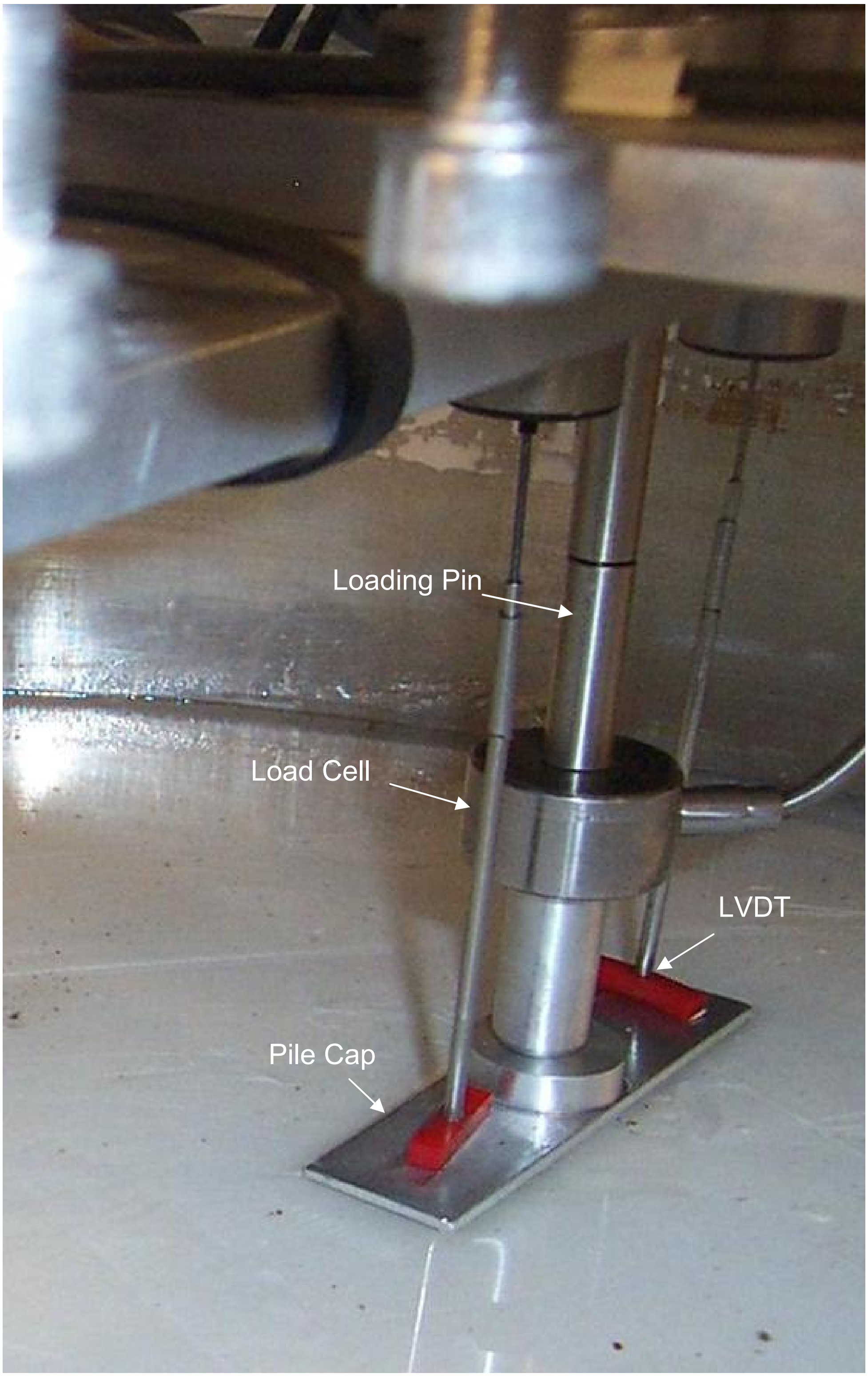


Figure 3.21 Loading pin used to axially load piles during centrifuge model testing.

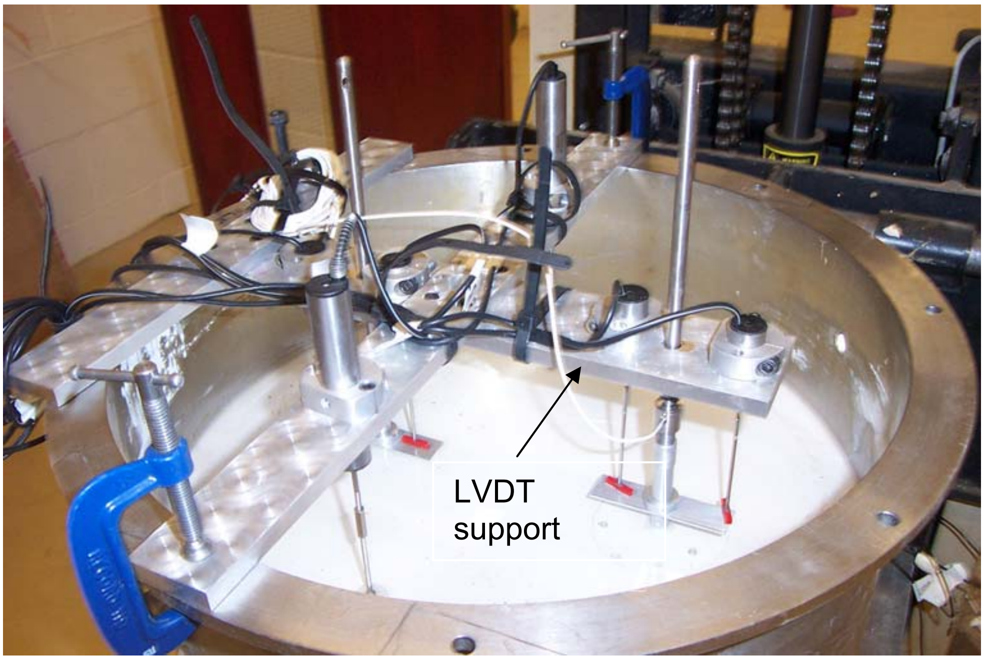


Figure 3.22 Linearly Variable Differential Transformers (LVDT) support clamped at the flange of the tub used for the first centrifuge model test.

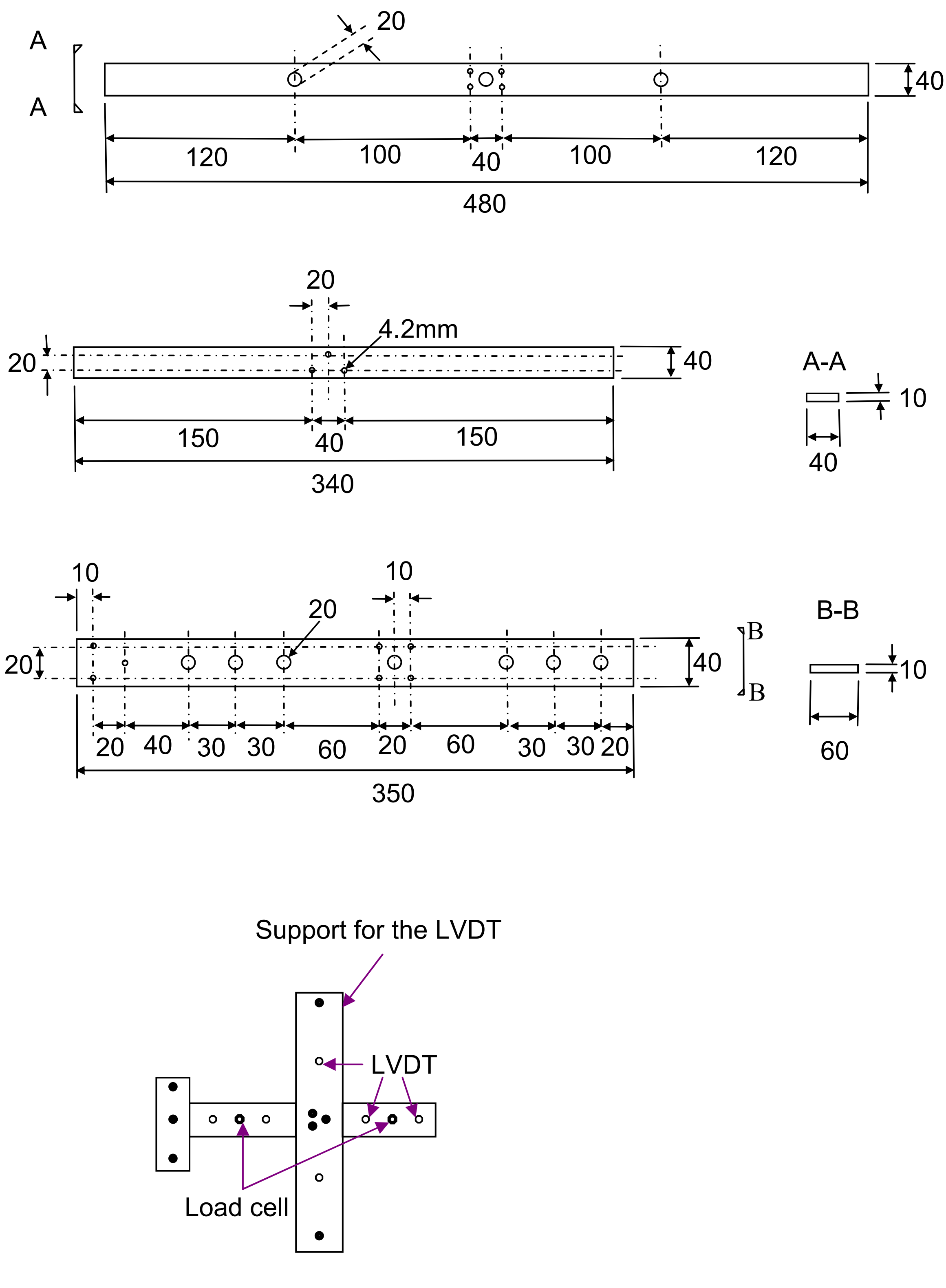


Figure 3.23 Details of aluminium beams used for supporting Linearly Variable Differential Transformers used for centrifuge model testing.

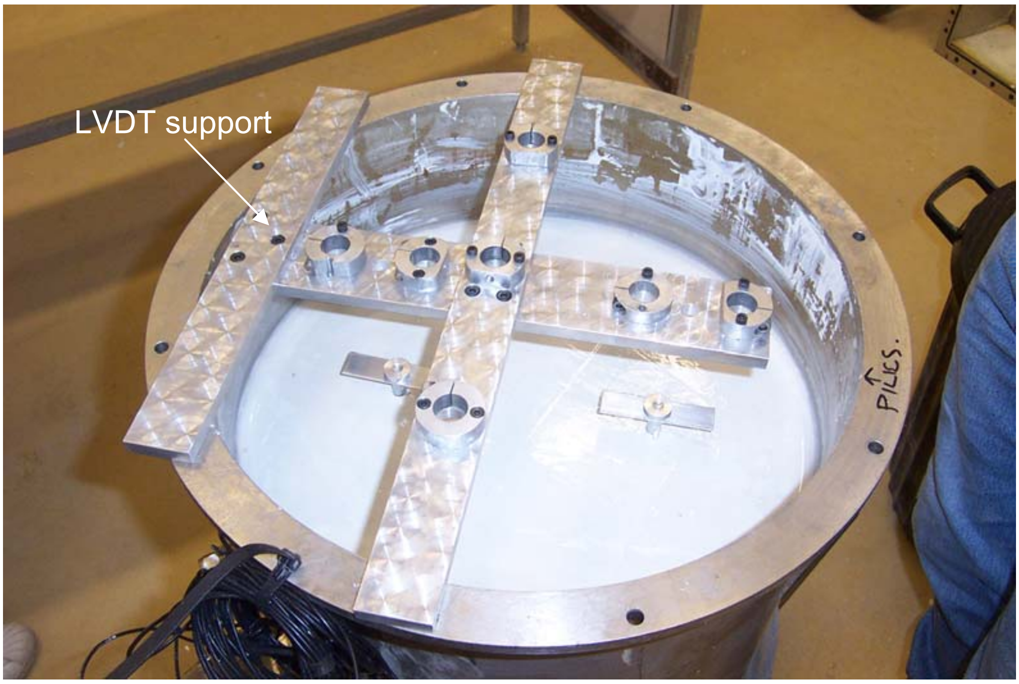


Figure 3.24 Linearly Variable Differential Transformers (LVDT) support used for the first centrifuge model testing.

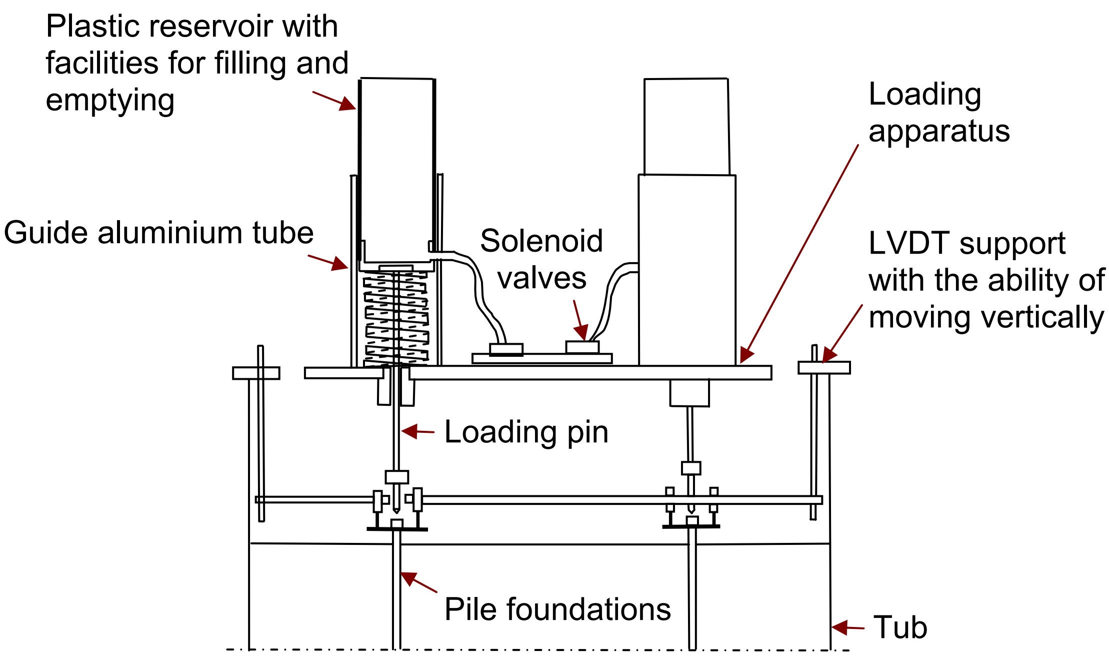


Figure 3.25 New loading apparatus developed for centrifuge model testing.

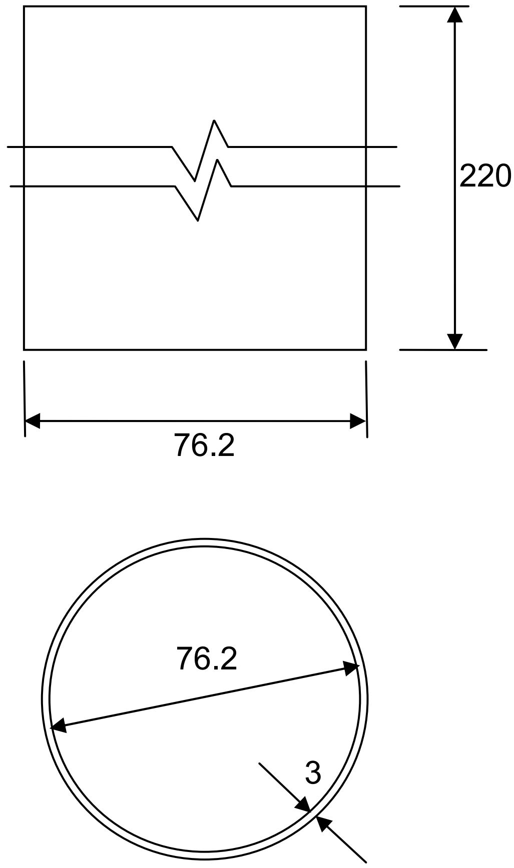


Figure 3.26 76.2 mm diameter plastic loading reservoir.

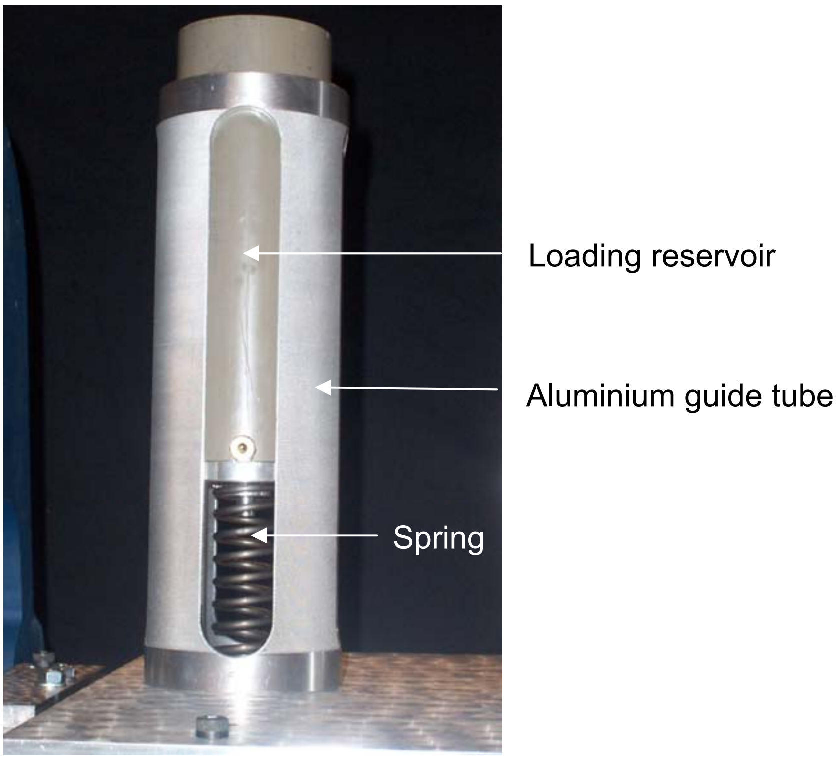


Figure 3.27 Plastic loading reservoir supported by a spring whiles empty.

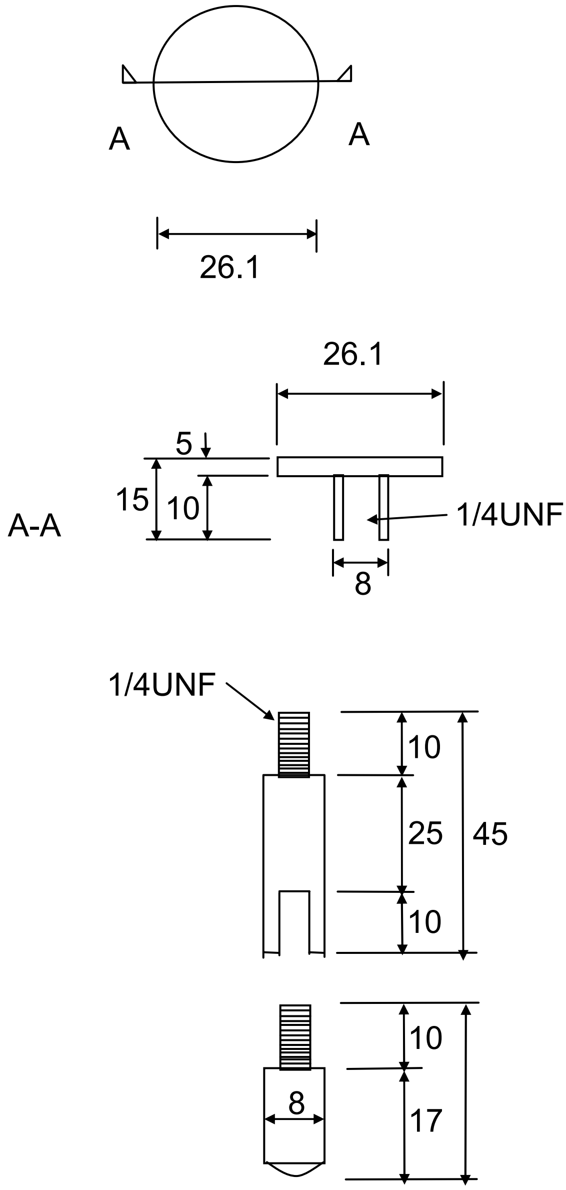


Figure 3.28 Loading pin and the loading pin base used for connecting to the base of the water reservoir.

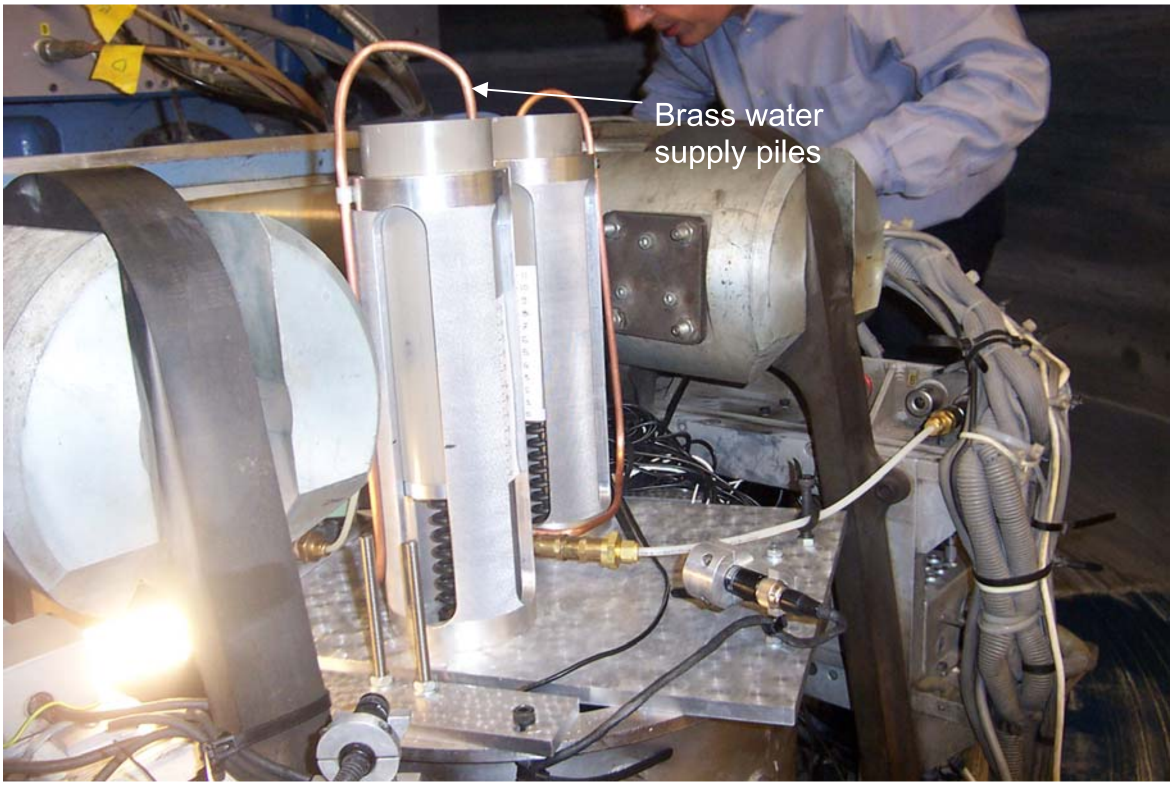


Figure 3.29 Brass pipes, connected to centrifuge slip rings, used to fill the loading reservoirs with water during centrifuge model testing.

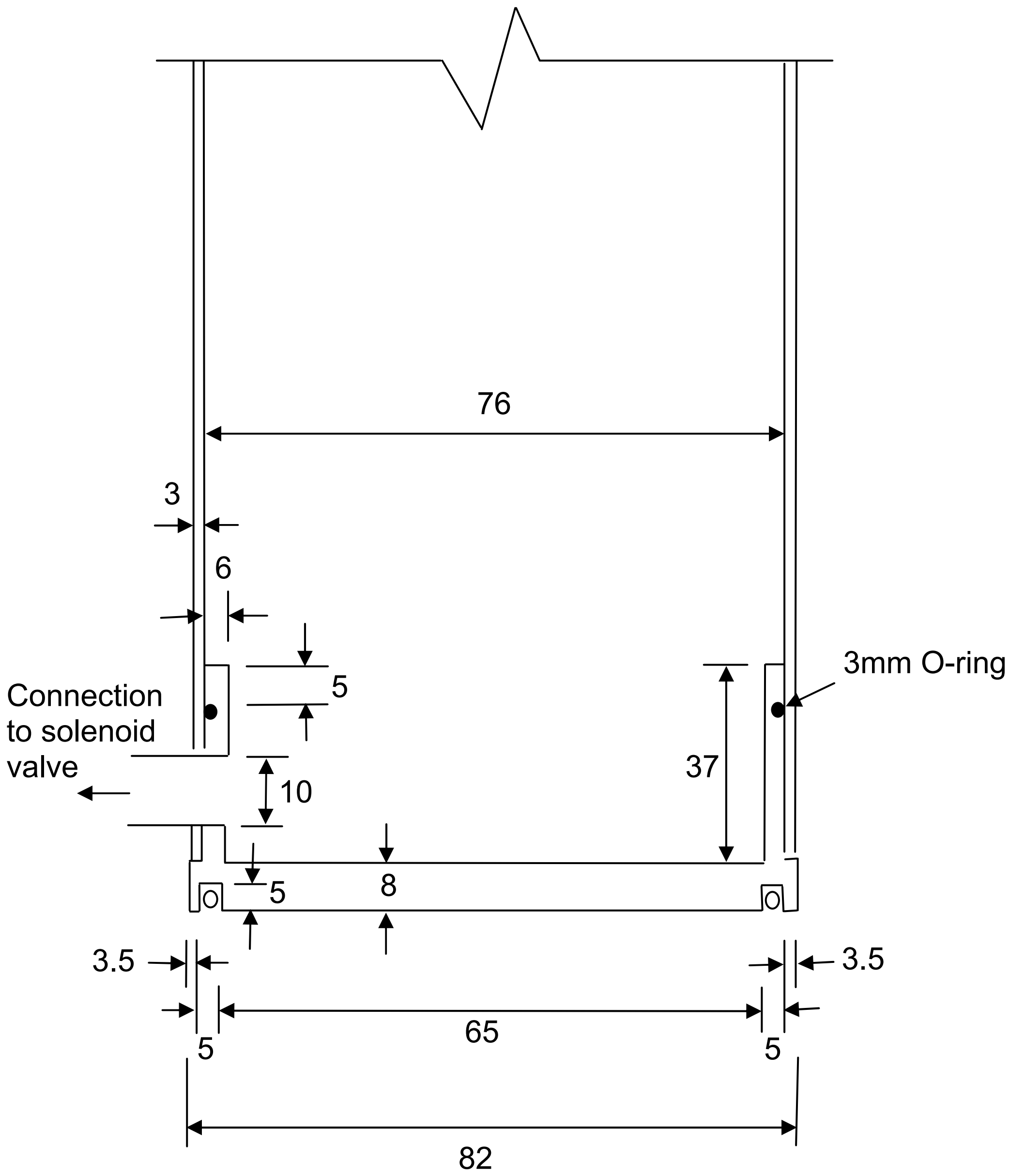


Figure 3.30 Base of the loading reservoir and details of the connection to solenoid valves.

Figure 3.31 Solenoid valves used to drain the water from the loading reservoirs during foundation unloading.

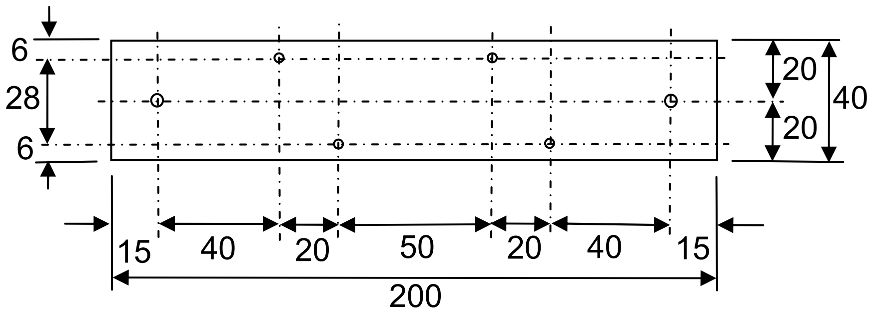


Figure 3.32 Aluminium plate used to connect the solenoid valves to the loading apparatus.

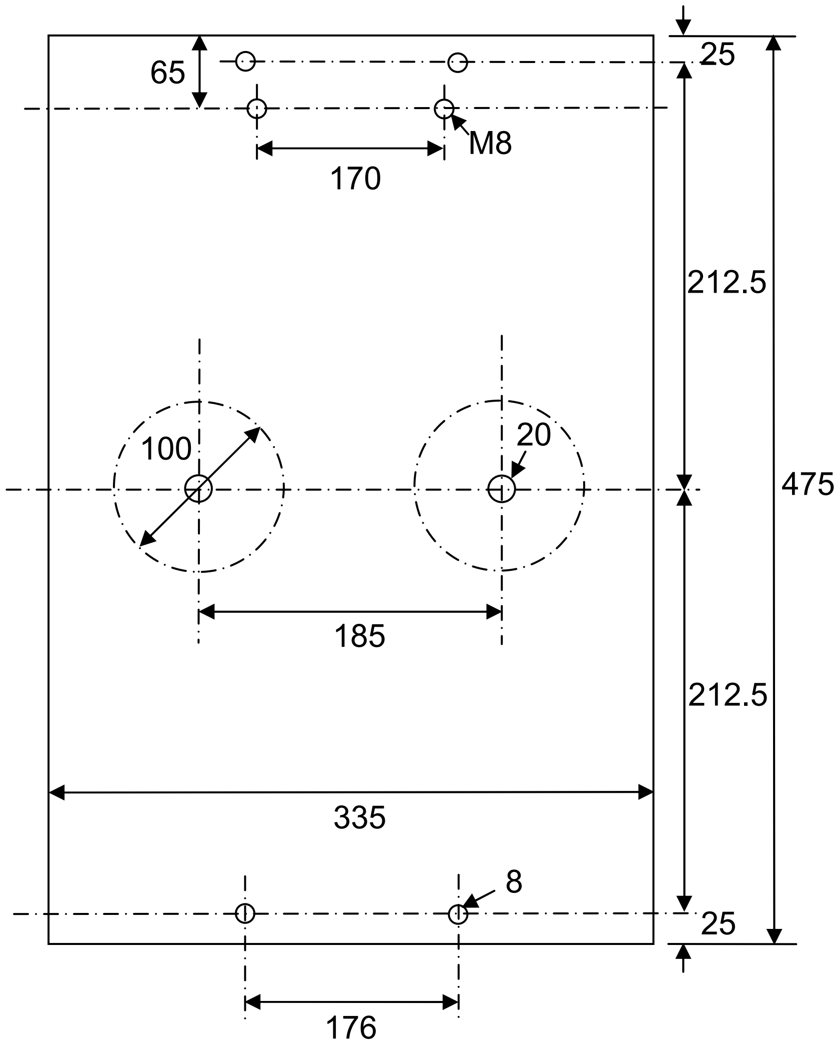


Figure 3.33 12mm thick aluminium plate used to support the loading apparatus and solenoid valves.

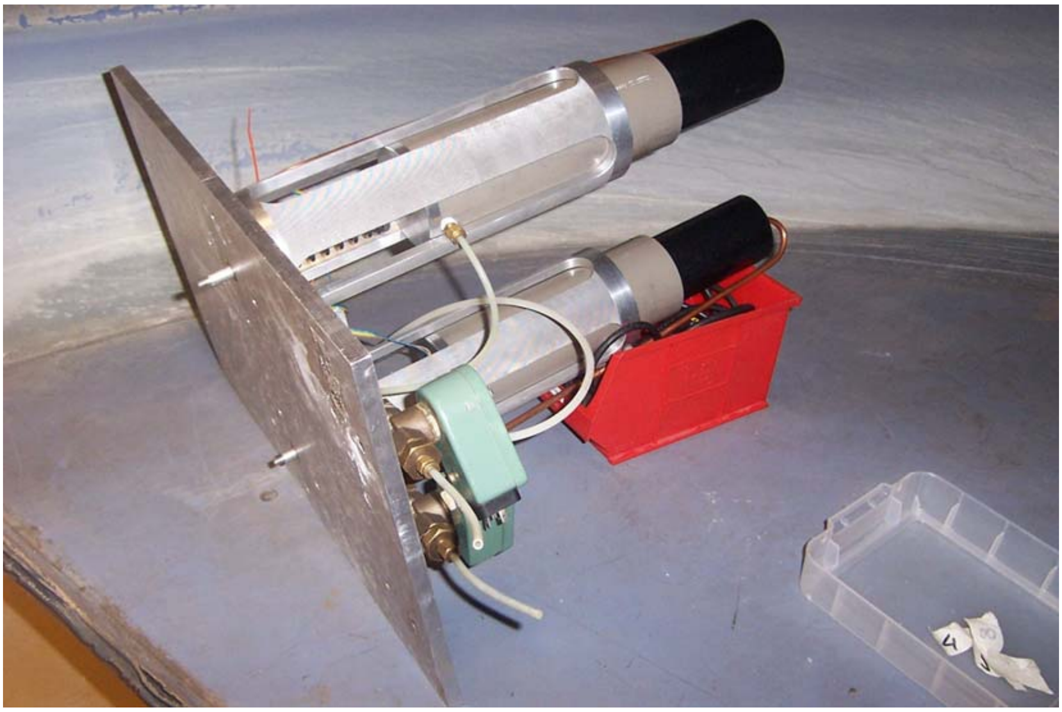


Figure 3.34 Loading reservoirs, guide tubes, springs and solenoid valves put together prior to mounting on the centrifuge tub.

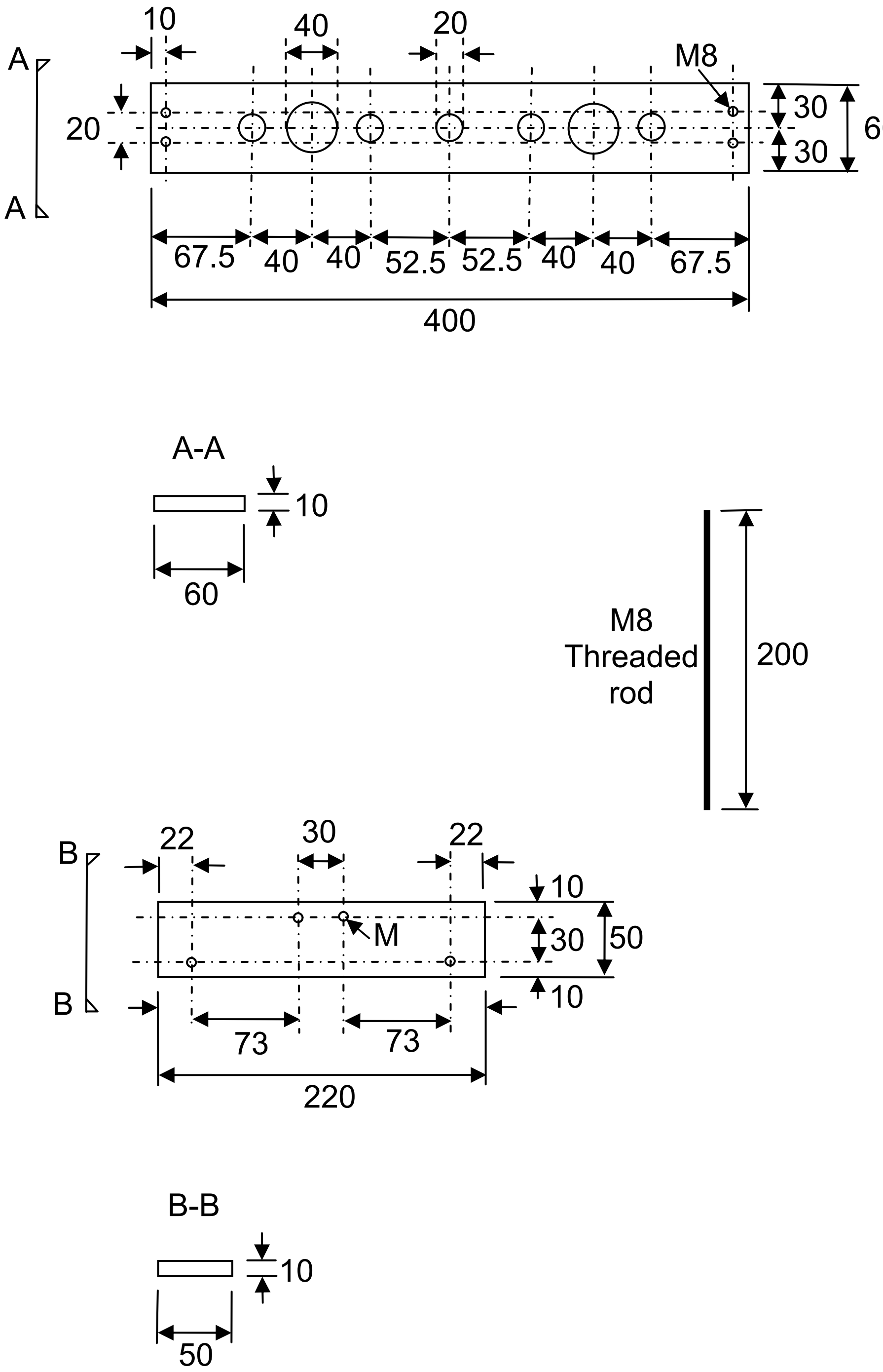


Figure 3.35 Details of the new developed and manufactured LVDT support used for centrifuge model testing.

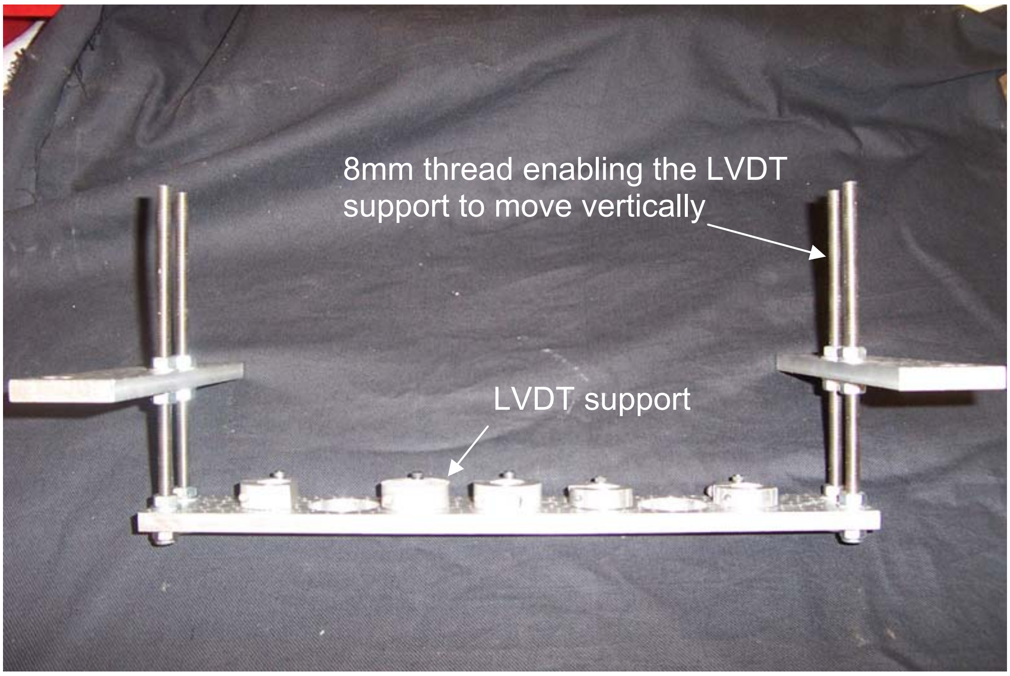


Figure 3.36 The LVDT support beam connected to 8mm threaded rod with the ability to move vertically depending on the thickness of the clay model.

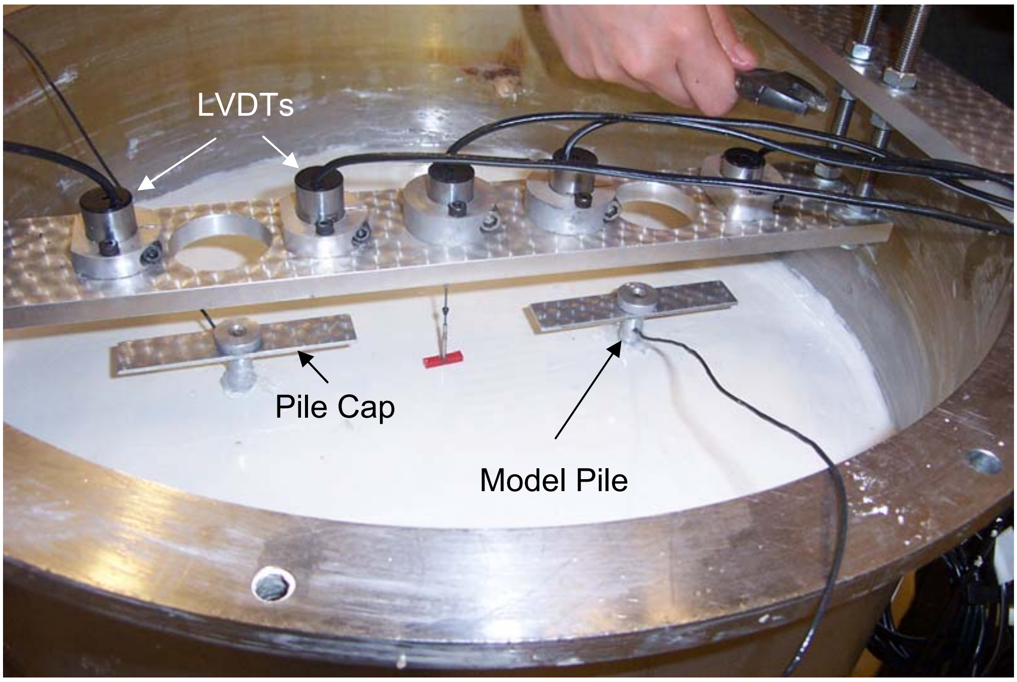


Figure 3.37 The LVDT support mounted and connected to the flange of the tub.

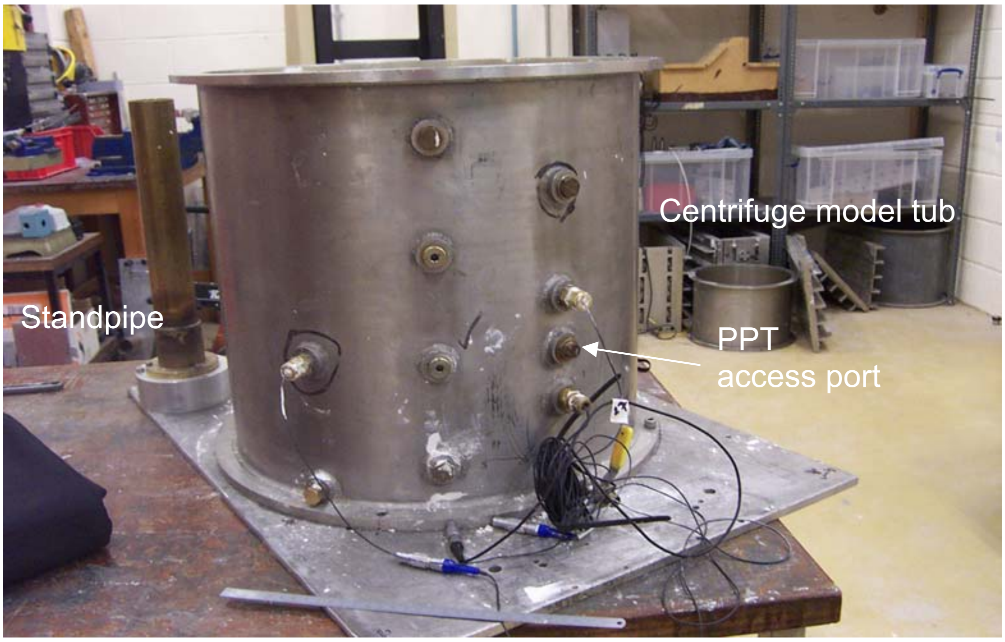


Figure 3.38 Centrifuge stainless steel tub with access ports through which the pore pressure transducers were installed.

Figure 3.39 Stainless steel model tub and extension used for centrifuge model preparation.

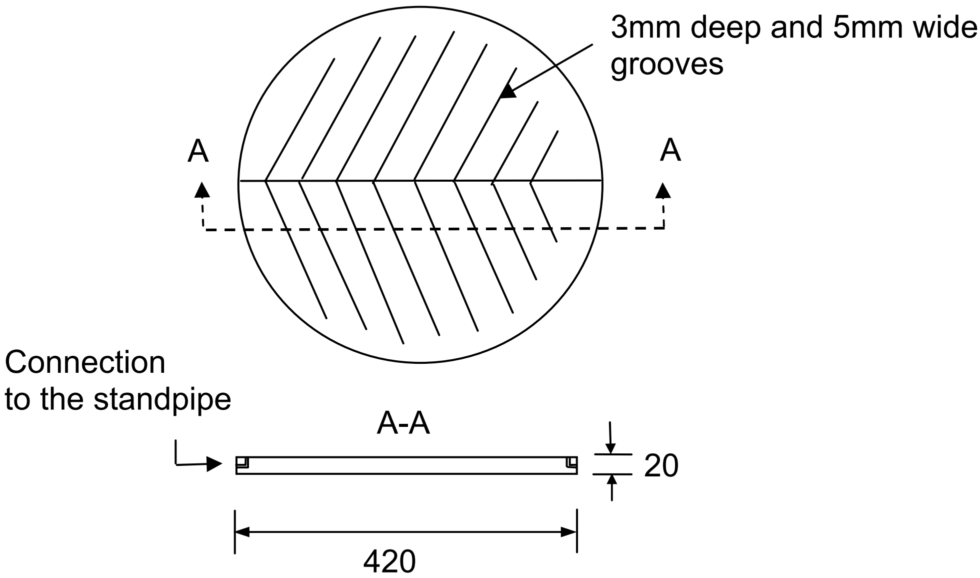


Figure 3.40 Drainage base plate.

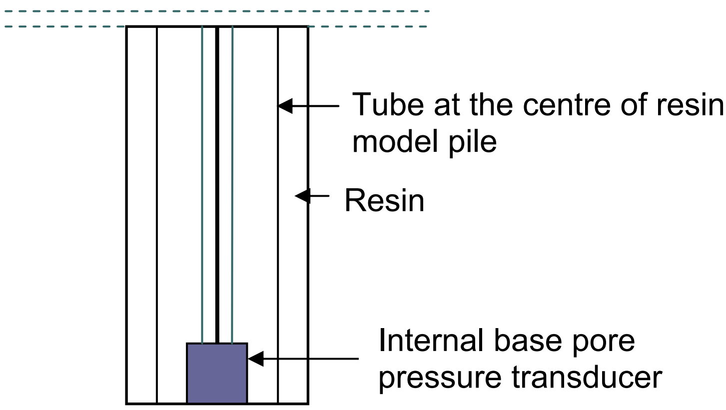


Figure 3.41 Proposed design for resin model piles to be used for centrifuge model testing.

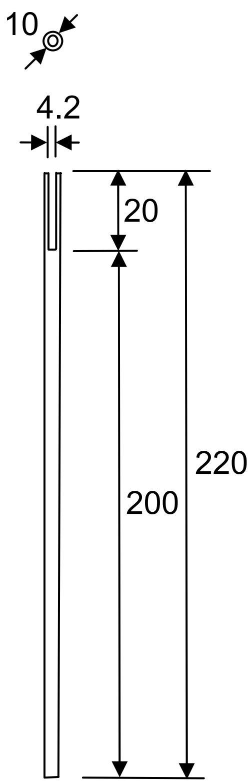


Figure 3.42 Aluminium model pile used for centrifuge model testing.

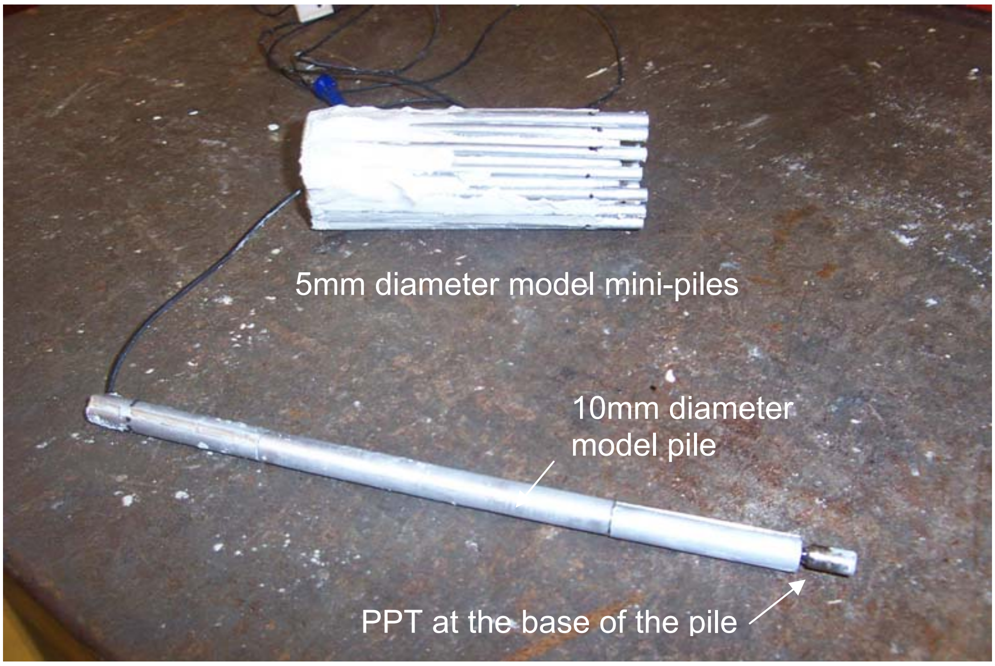


Figure 3.43 Model pile made of aluminium sections with the pore pressure transducer at the base (Test LQ10).

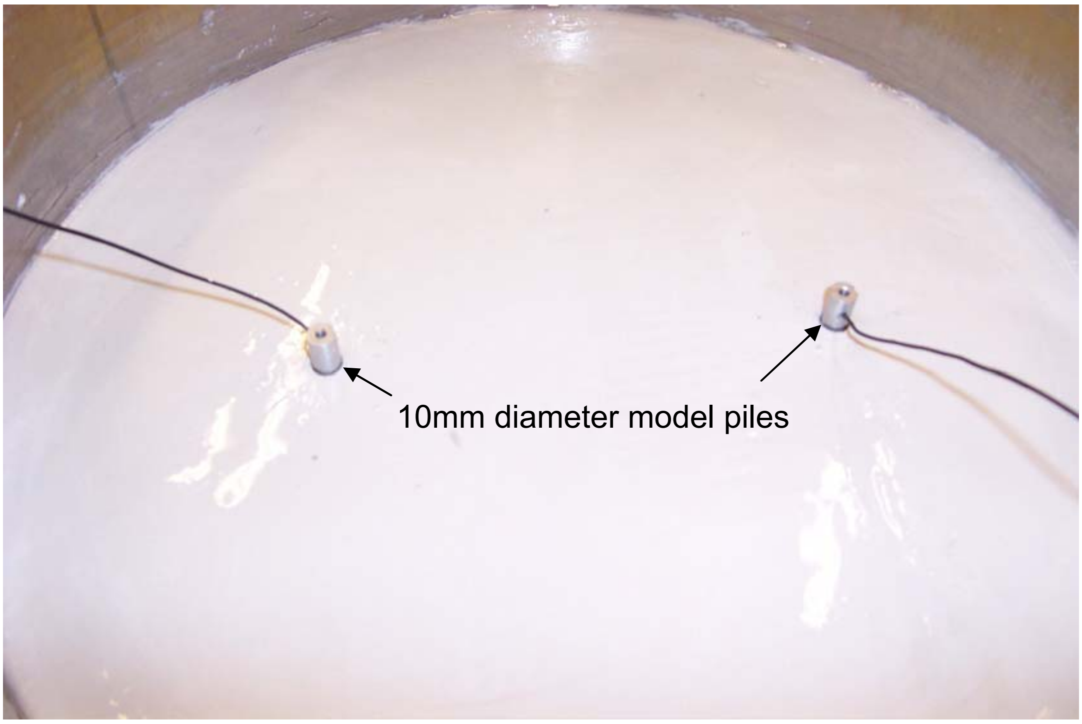


Figure 3.44 10mm diameter piles embedded 200mm into the soil sample

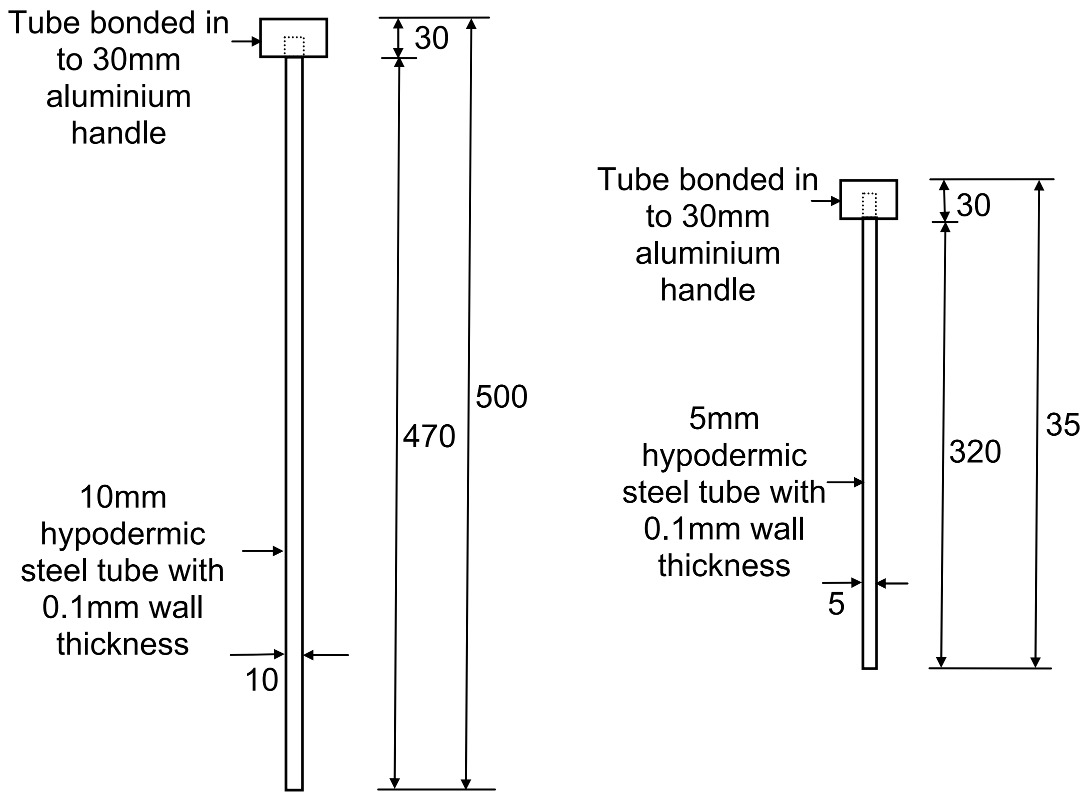


Figure 3.45 Details of tool used for installing bored model piles.

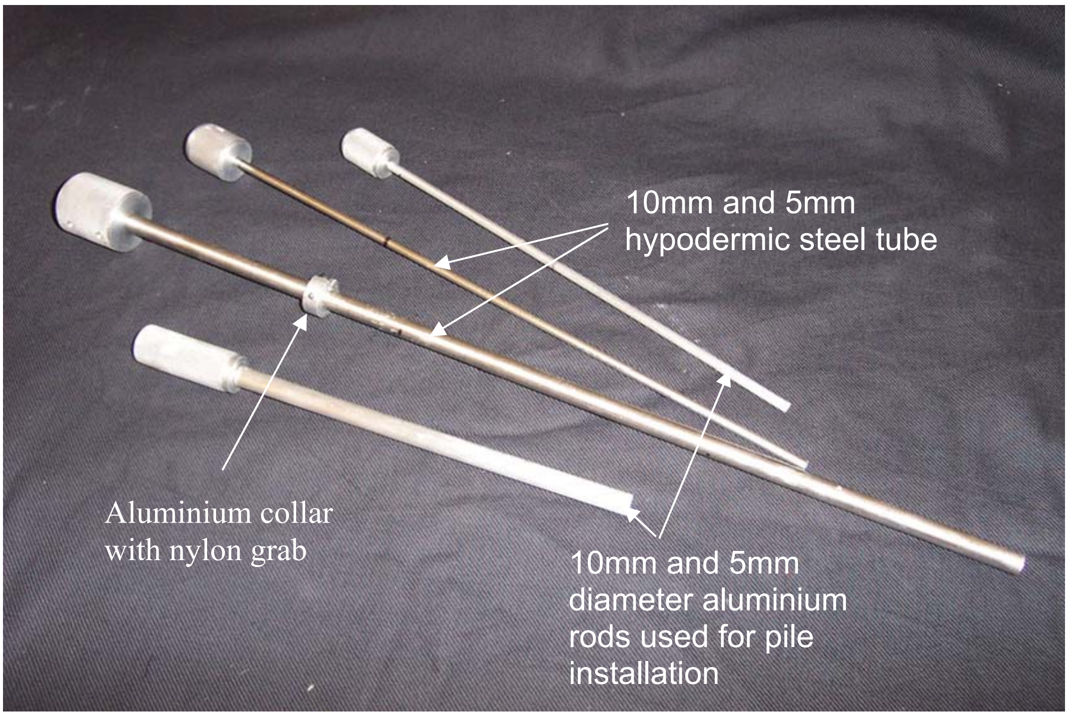


Figure 3.46 Stainless steel tubes used for model pile installation.

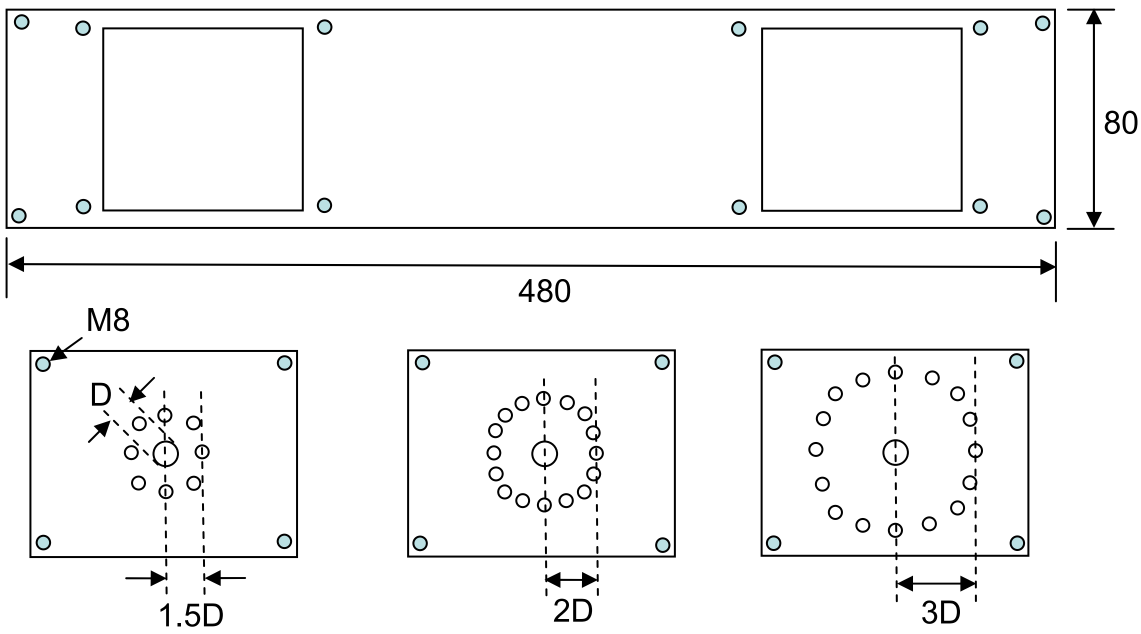


Figure 3.47 Detail of jig used for model pile installation.

Figure 3.48 Model pile installation at 1g using a template to position and install piles.

Figure 3.49 Prior to model pile installation a small amount of clay slurry is placed into the bored hole to ensure that the model pile is in good contact with clay model.

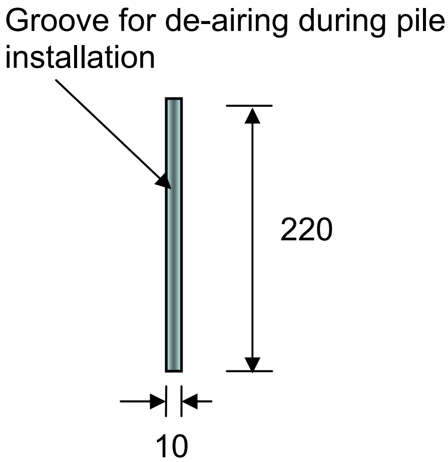


Figure 3.50 10mm diameter model pile with a groove for de-airing during installation.

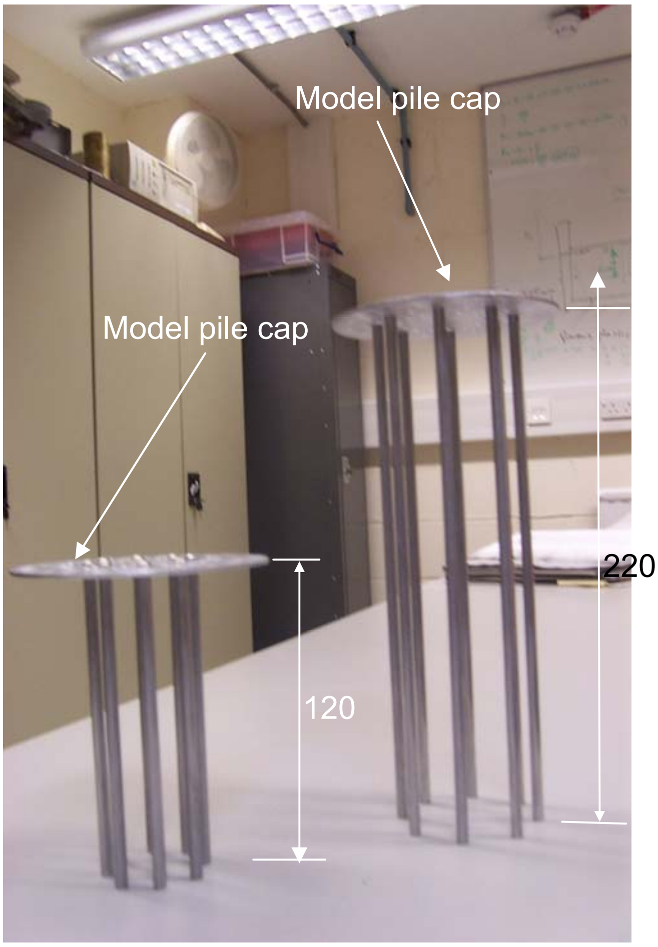


Figure 3.51 120mm and 220mm model mini-pile foundations.

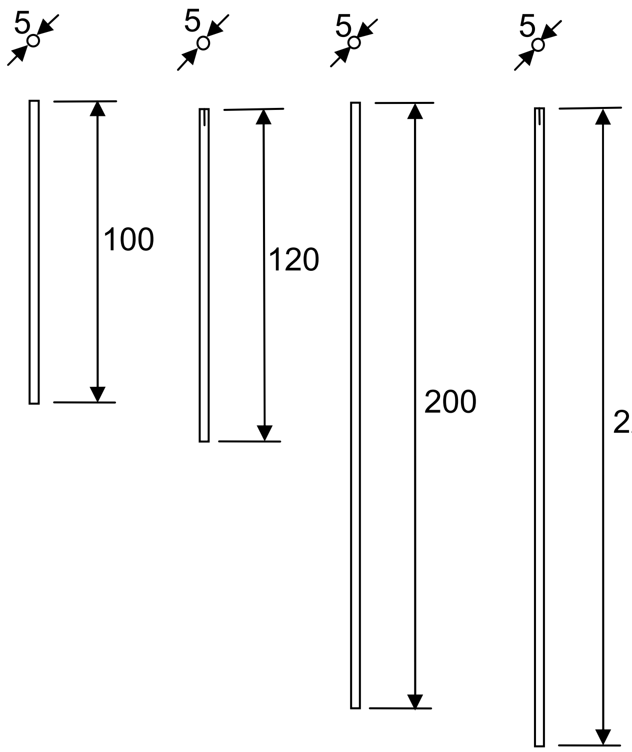


Figure 3.52 Details of model mini-pile foundations used for centrifuge model testing.

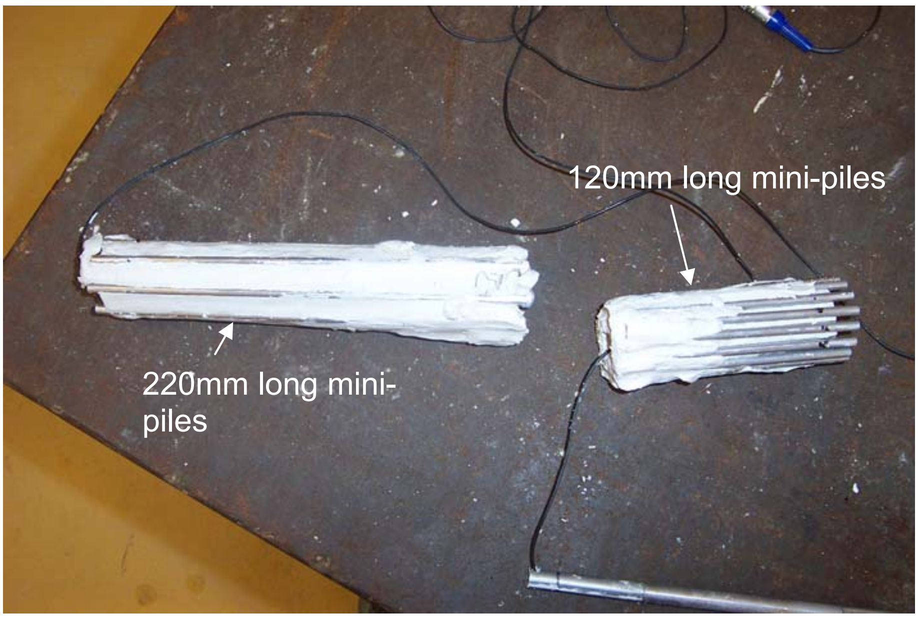


Figure 3.53 Mini-pile groups removed from the model soil after testing (Test LQ16).

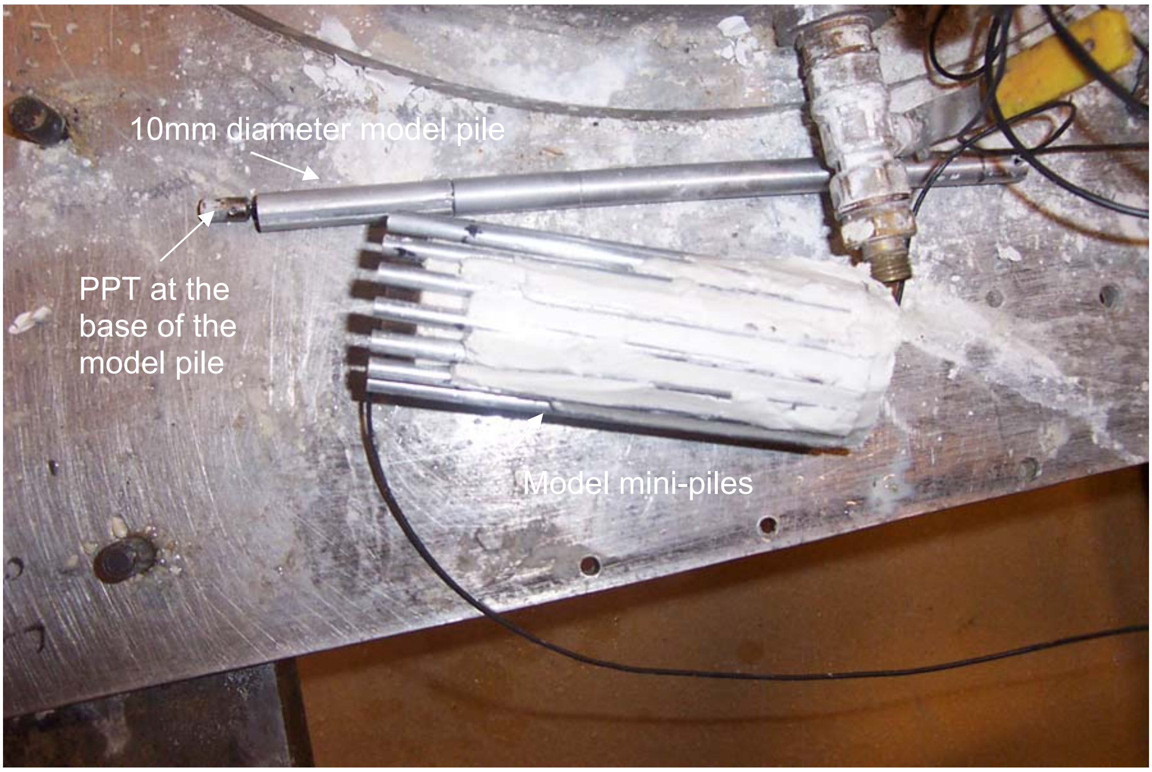


Figure 3.54 10mm diameter model with a pore pressure transducer at the base.

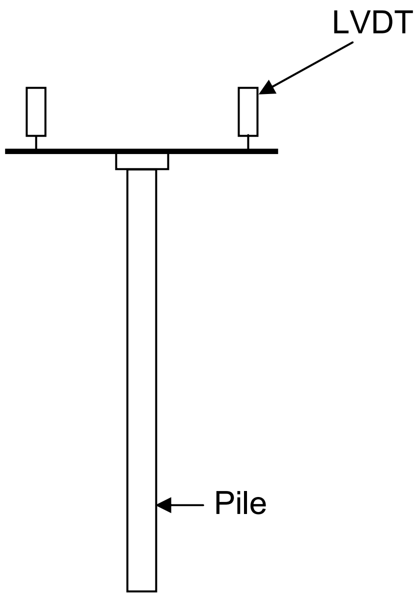


Figure 3.55 Geometry of the single pile centrifuge model test.

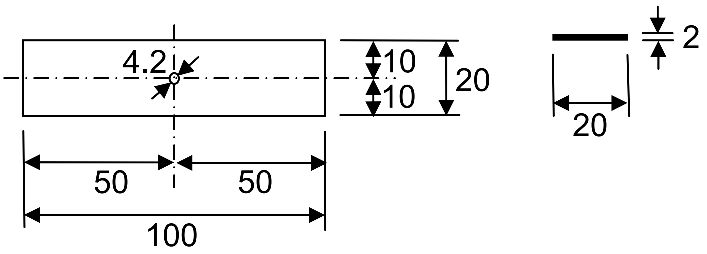


Figure 3.56 100mm by 20mm aluminium plate used when testing the performance of single model pile foundation.

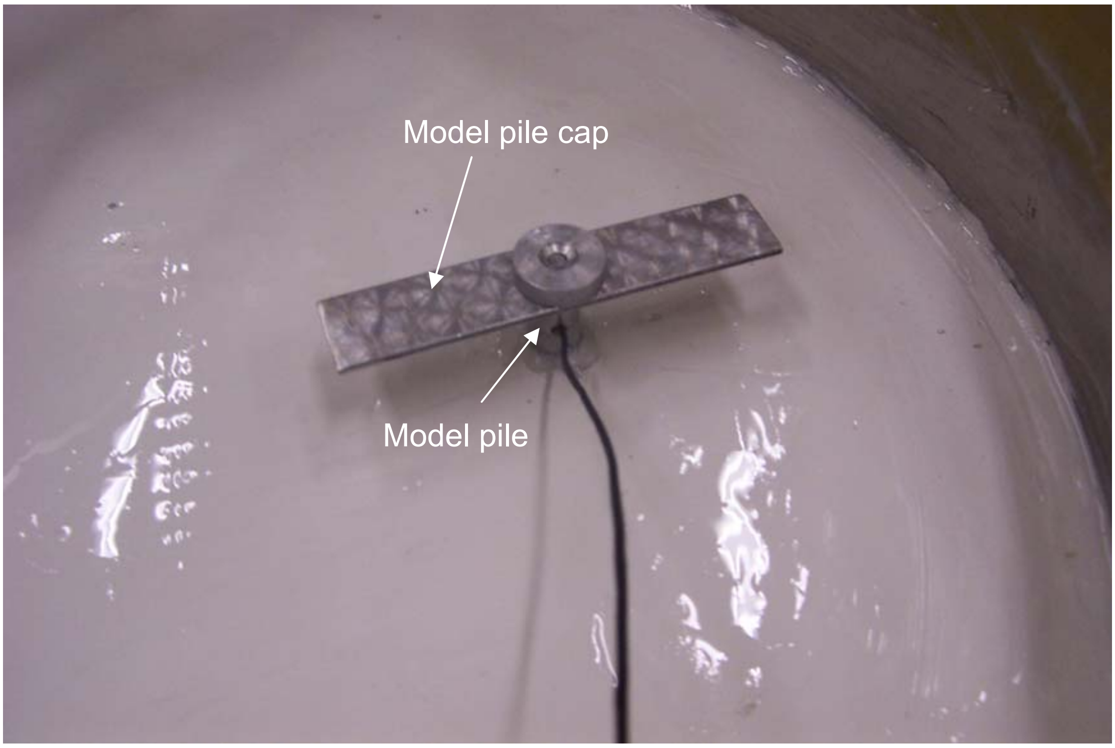


Figure 3.57 Model plate connected to the pile head 20mm above the clay surface.

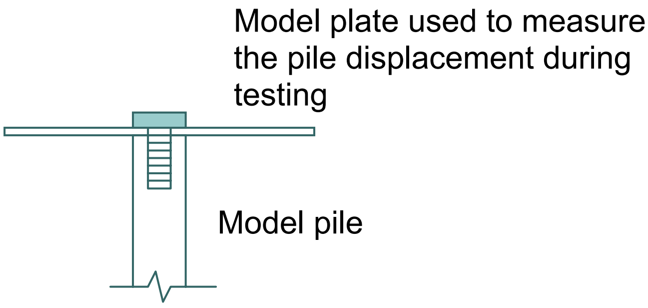


Figure 3.58 Details of connection of the model plate and aluminium model pile.

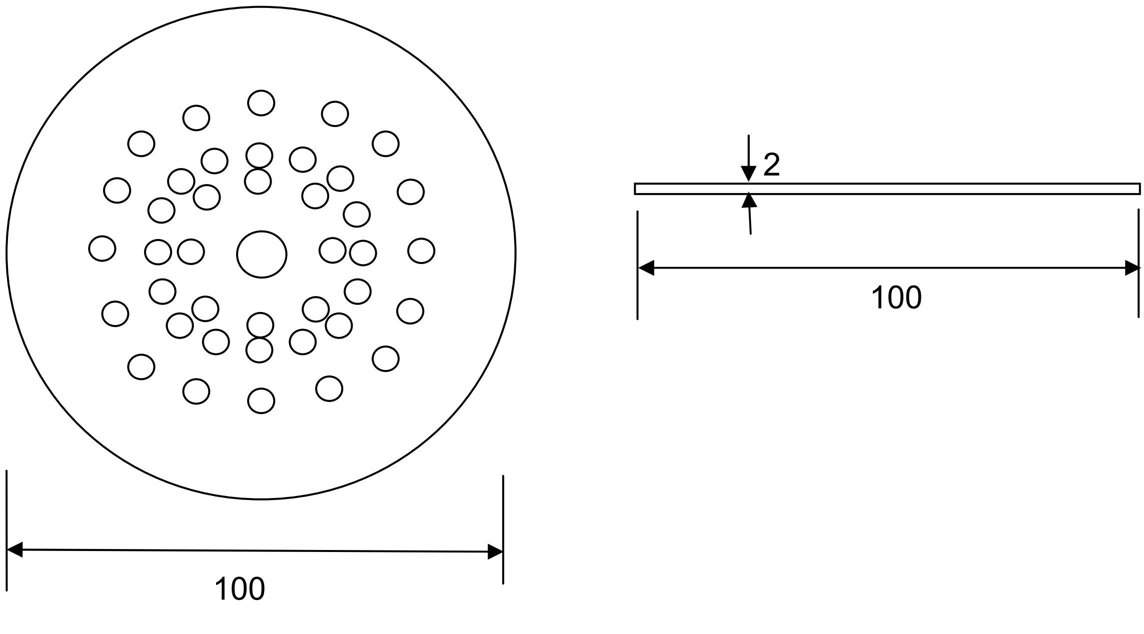


Figure 3.59 Details of the pile cap used for centrifuge model testing of the novel pile groups.

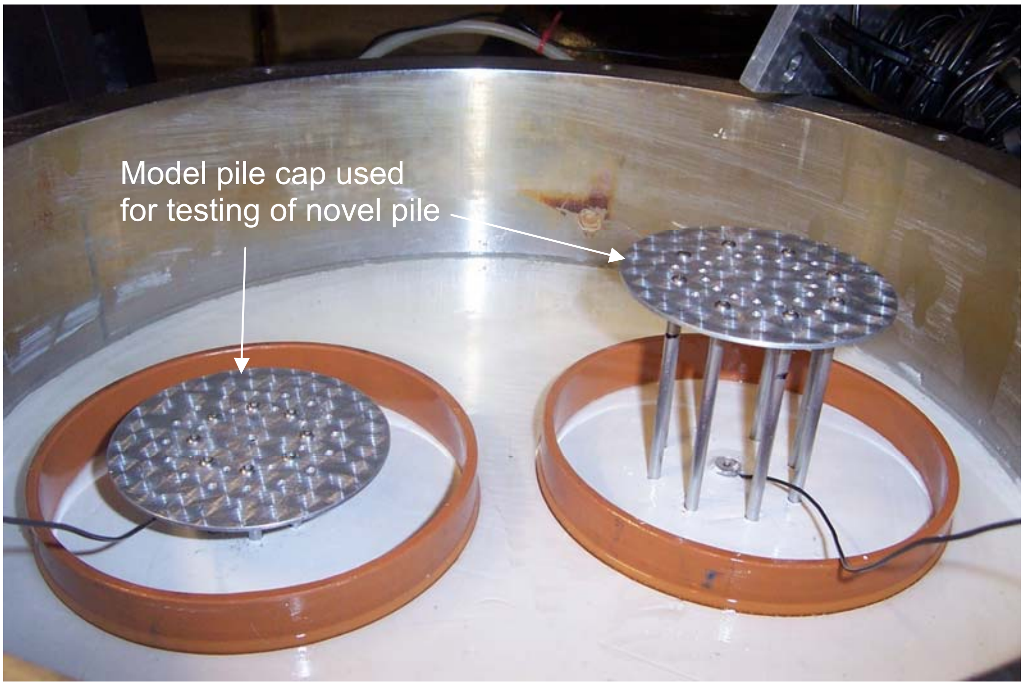


Figure 3.60 100mm diameter aluminum plate used when testing the performance of novel pile groups.

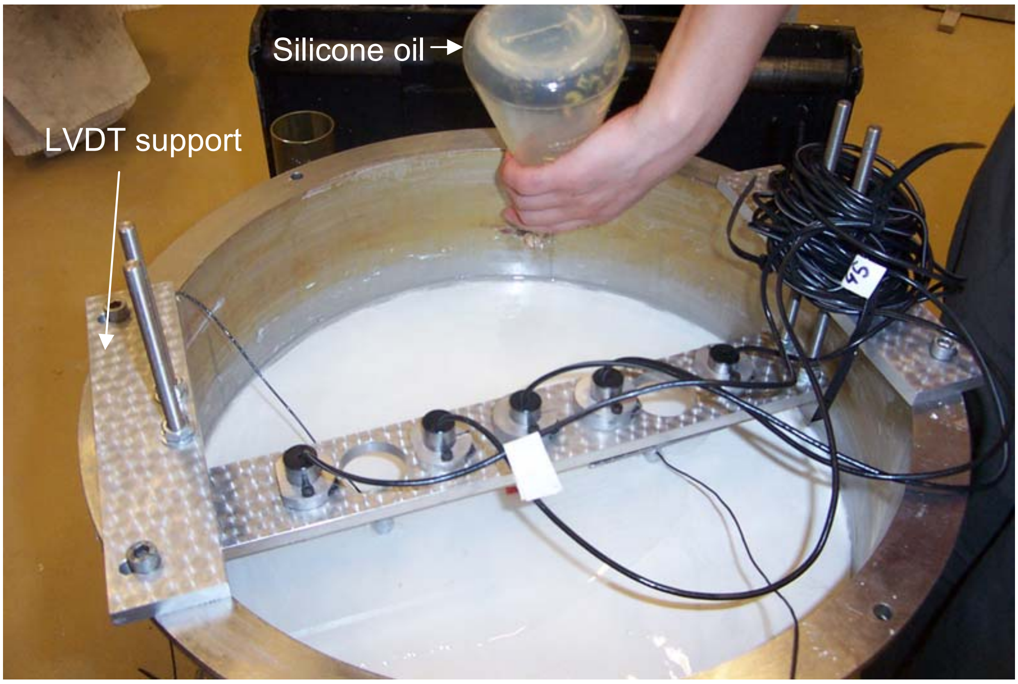


Figure 3.61 The model clay with an impermeable surface sealed with silicon oil.

Figure 3.62 Connection between the drainage base plate to the standpipe.

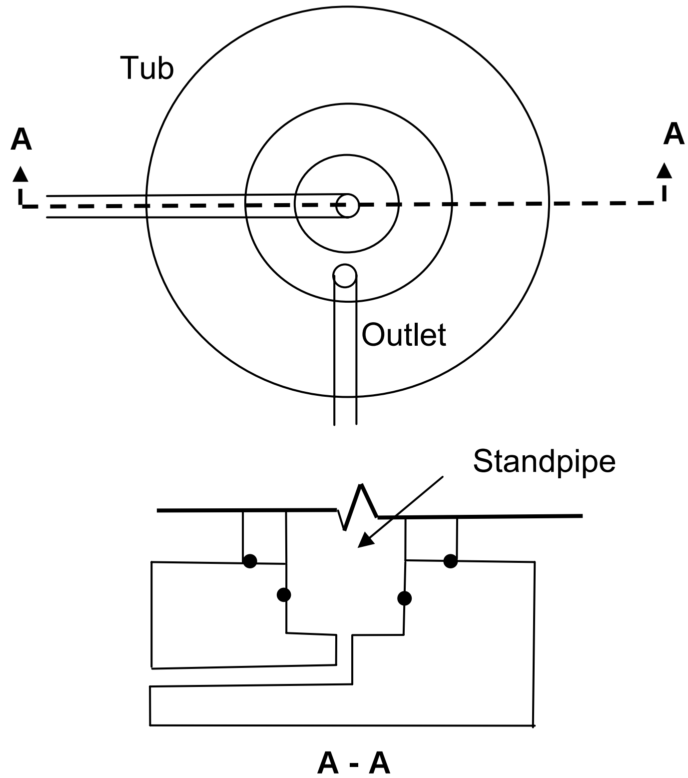


Figure 3.63 Detail of standpipe developed for centrifuge model testing.

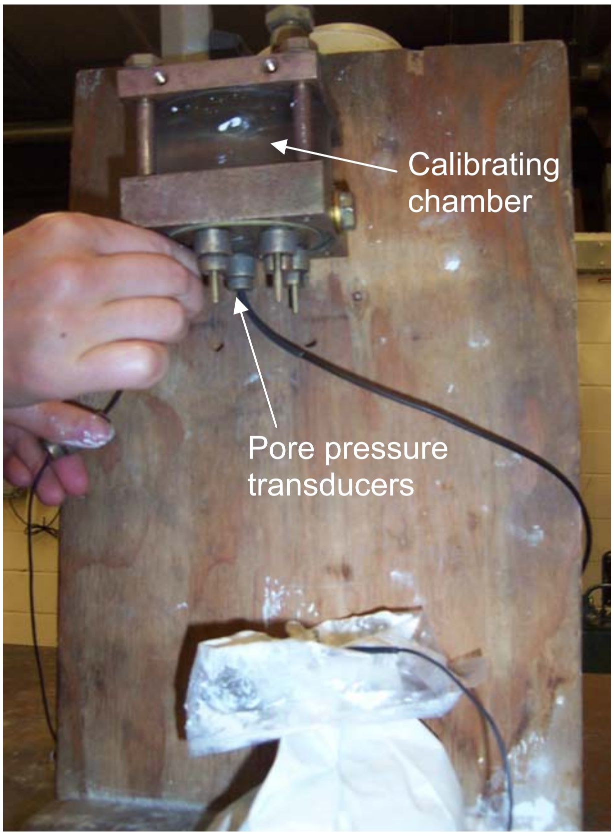


Figure 3.64 Pore pressure transducers in calibration chamber used for centrifuge model testing.

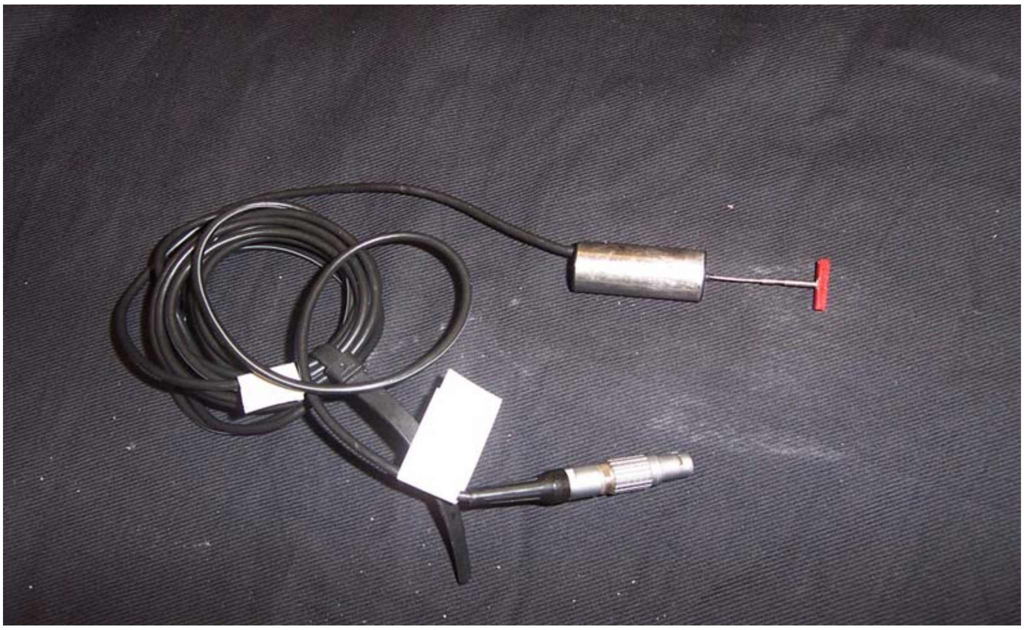


Figure 3.65 LVDT used for centrifuge model testing.

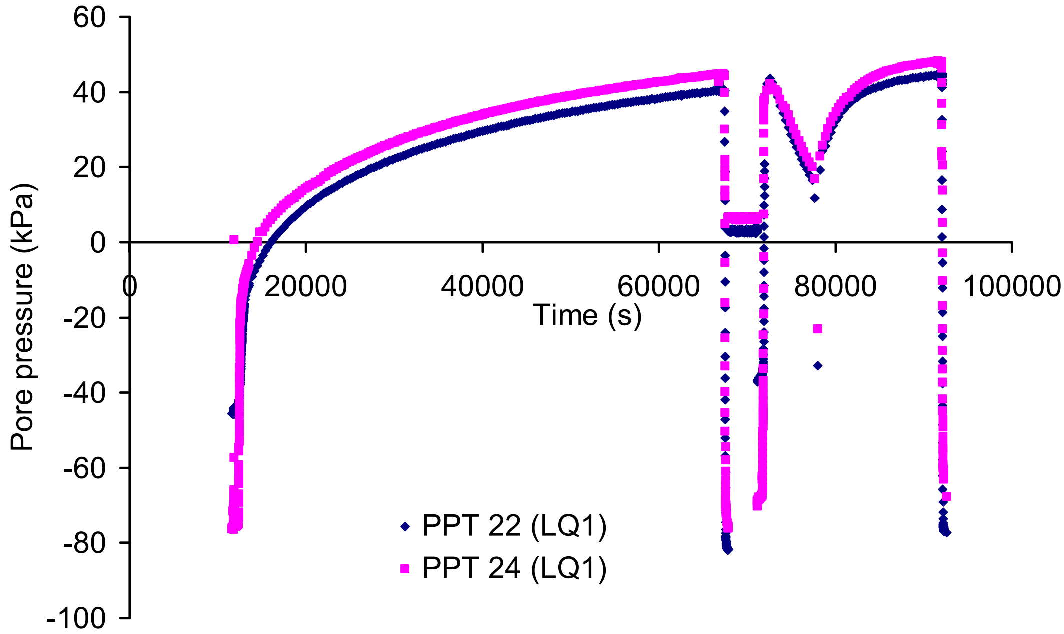


Figure 3.66 Pore pressure distribution during centrifuge model testing.

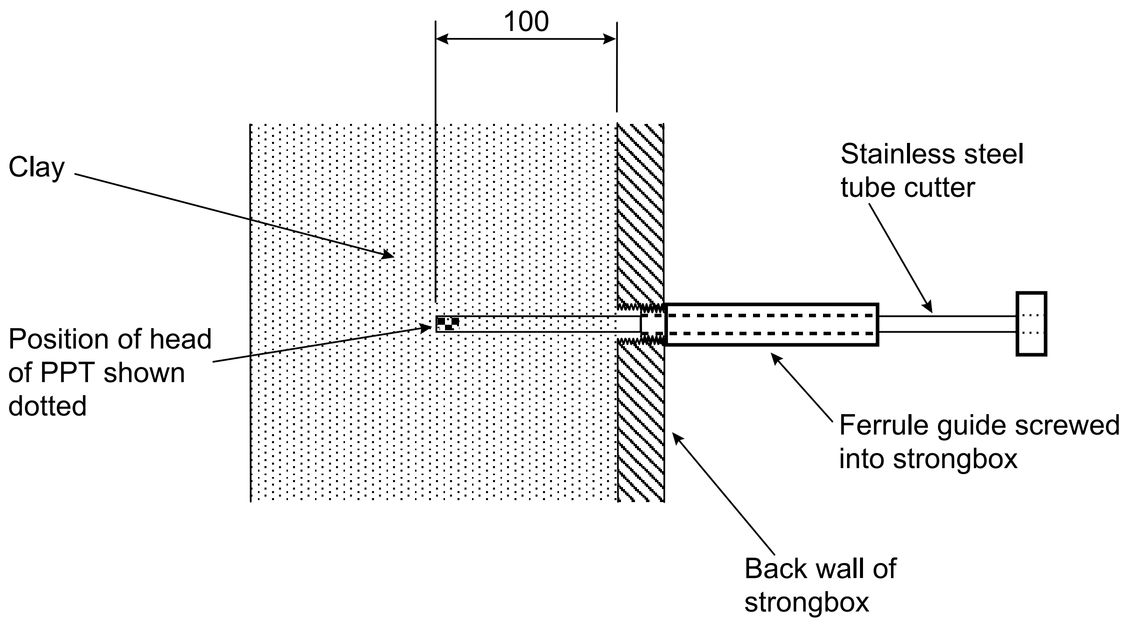


Figure 3.67 Method of ensuring correct positioning of pore pressure transducers within model (McNamara, 2001).

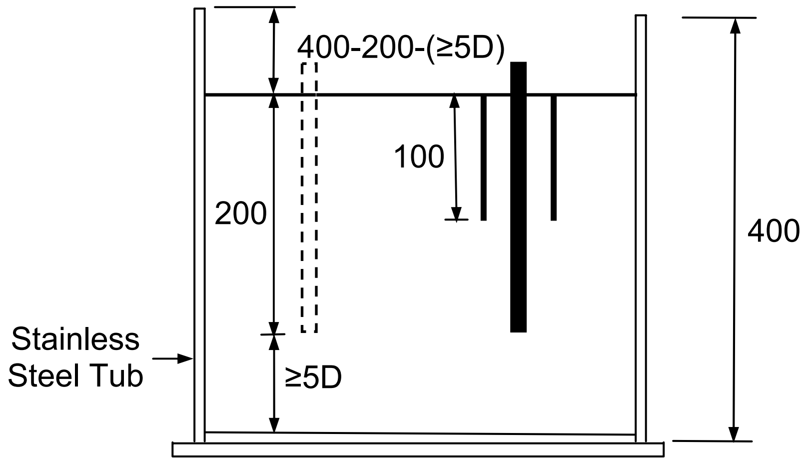


Figure 3.68 Geometry details of the centrifuge model.

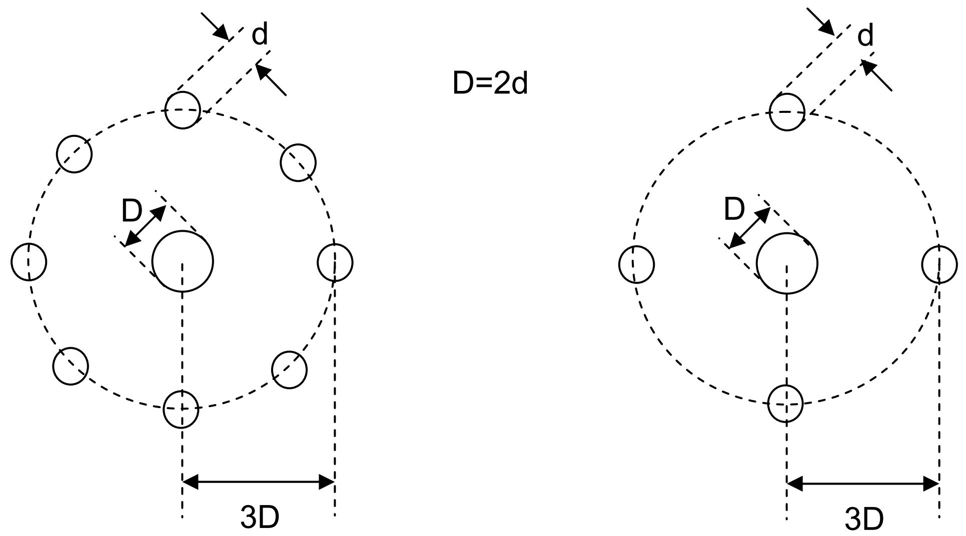


Figure 4.1 Geometry of the centrifuge model; groups of eight and four mini-piles.

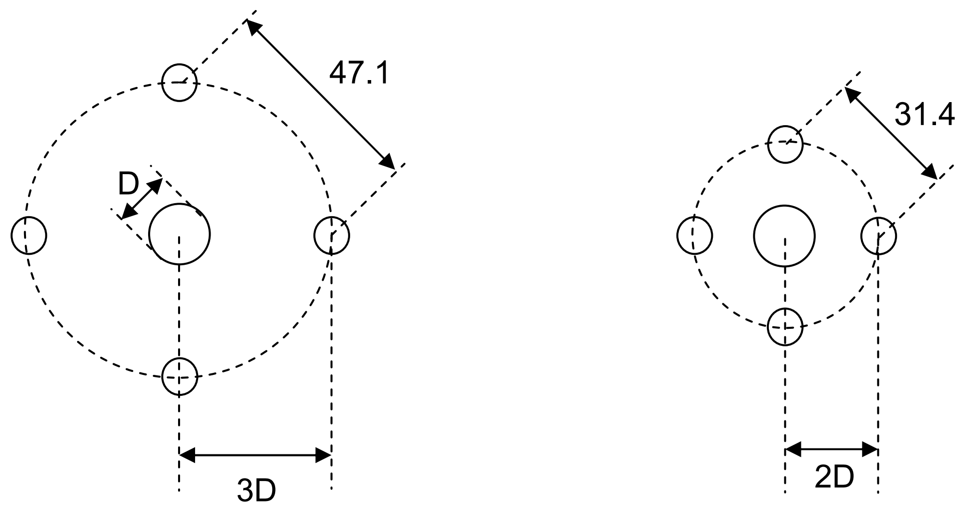


Figure 4.2 Examples of the geometry of the novel pile groups tested.

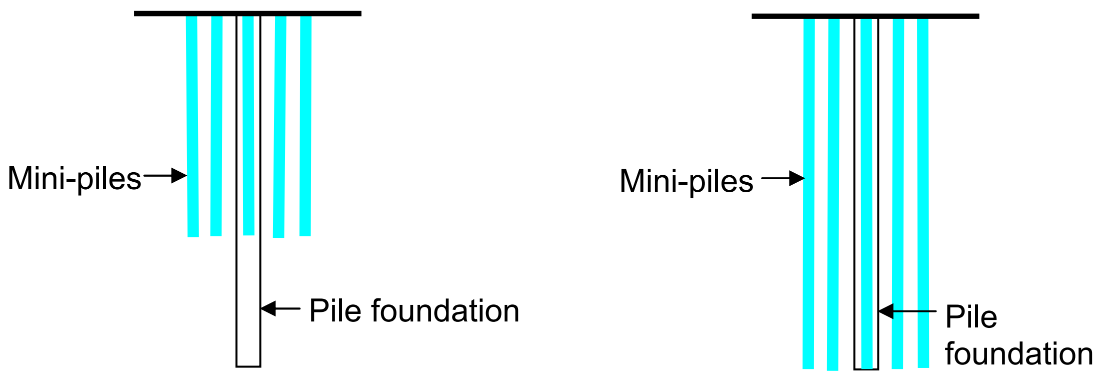


Figure 4.3 Examples of the geometry of novel pile groups with 100mm and 200mm long mini-piles.

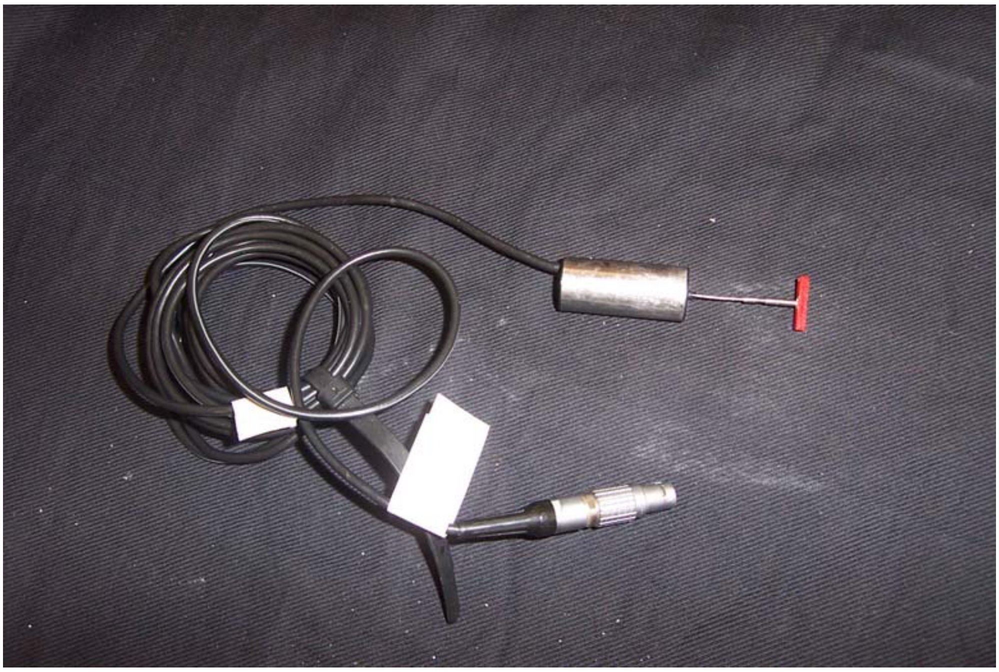


Figure 4.4 LVDT used to measure model pile displacement during centrifuge testing.

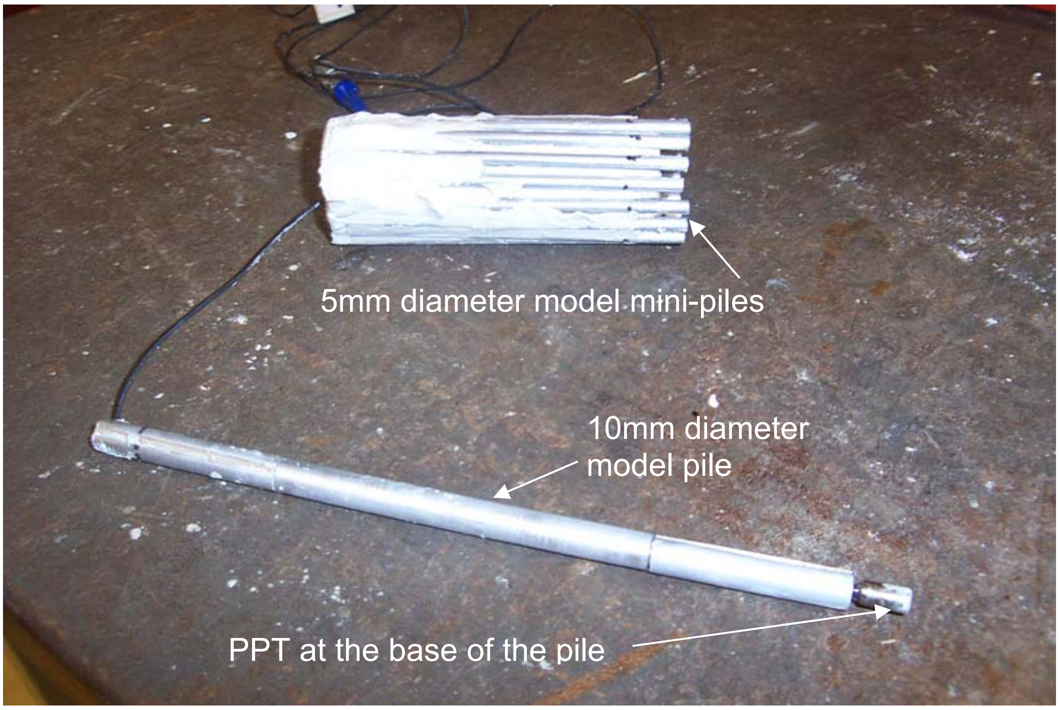


Figure 4.5 Model pile made of aluminium sections with the pore pressure transducer at the base.

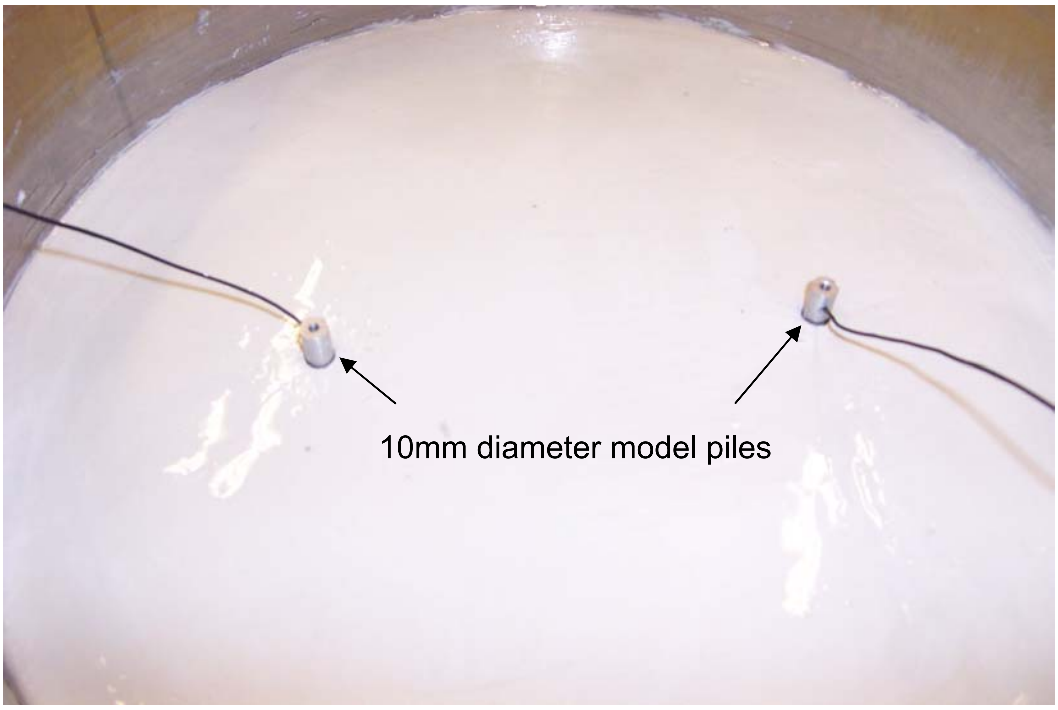


Figure 4.6 10mm diameter piles installed at 1g and embedded 200mm into the soil sample.

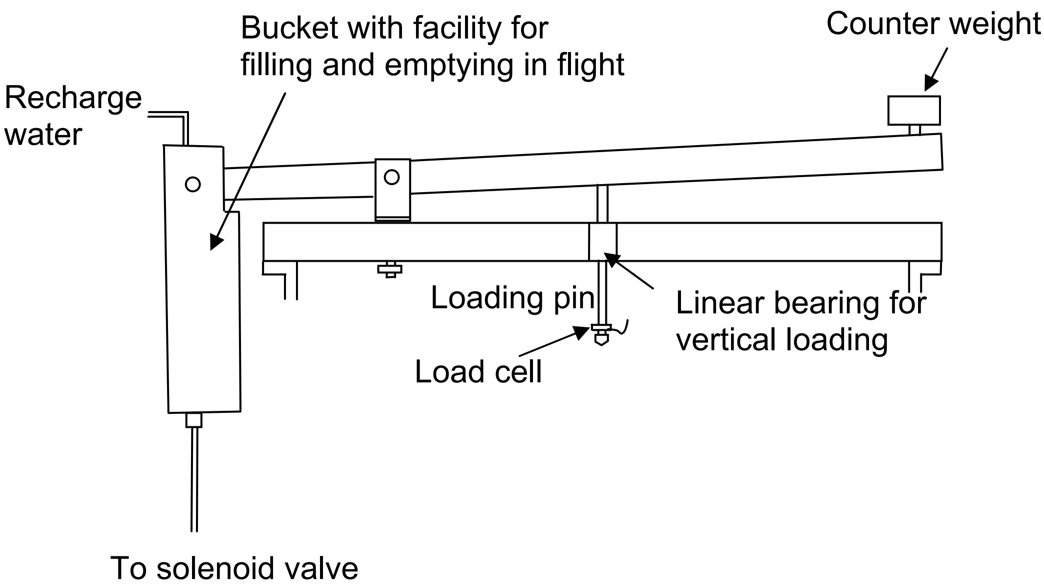


Figure 4.7 Axial loading pile foundation device used in centrifuge model testing (Morrison, 1994).

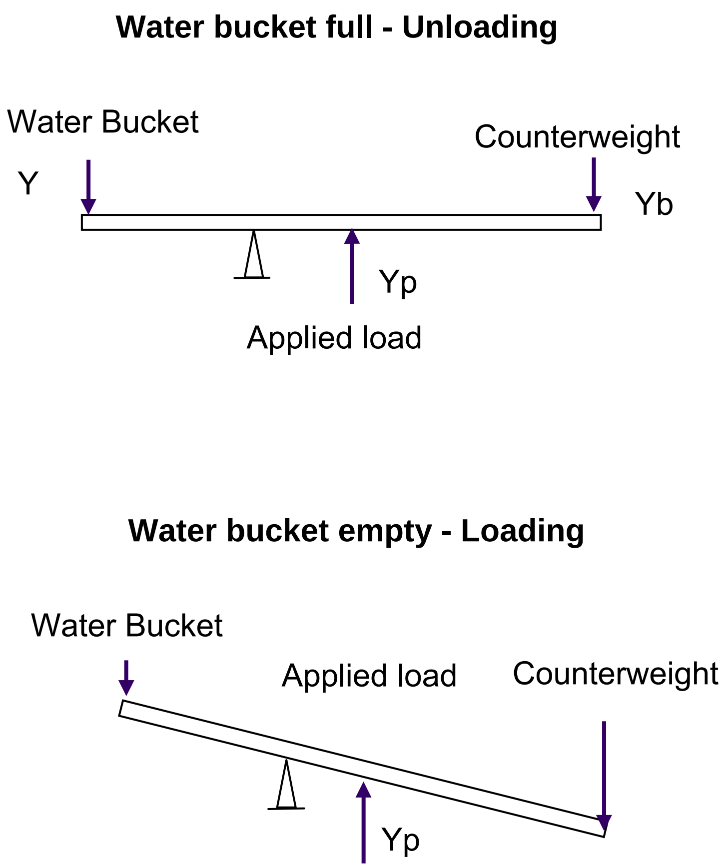


Figure 4.8 Mechanism of foundation loading device by Morrison (1994).

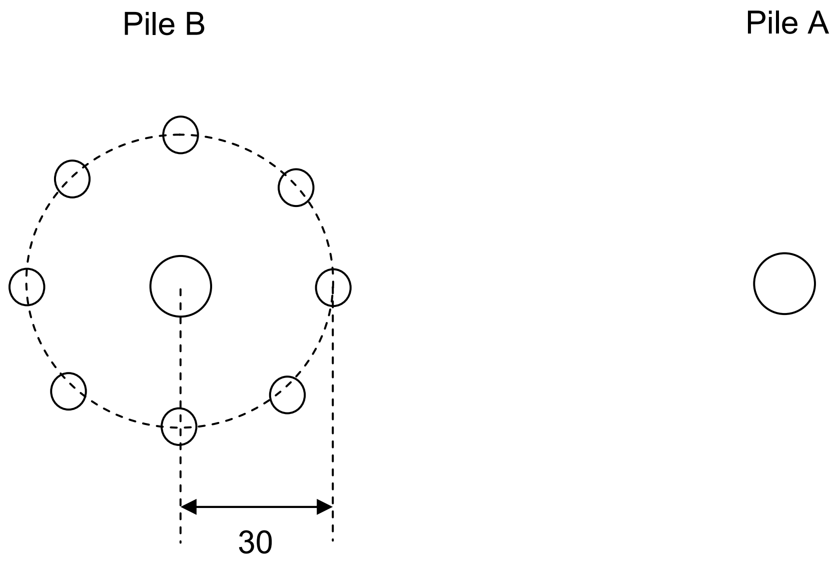


Figure 4.9 Test LQ1 – Geometry of the centrifuge model after piles A and B were subjected to first loading.

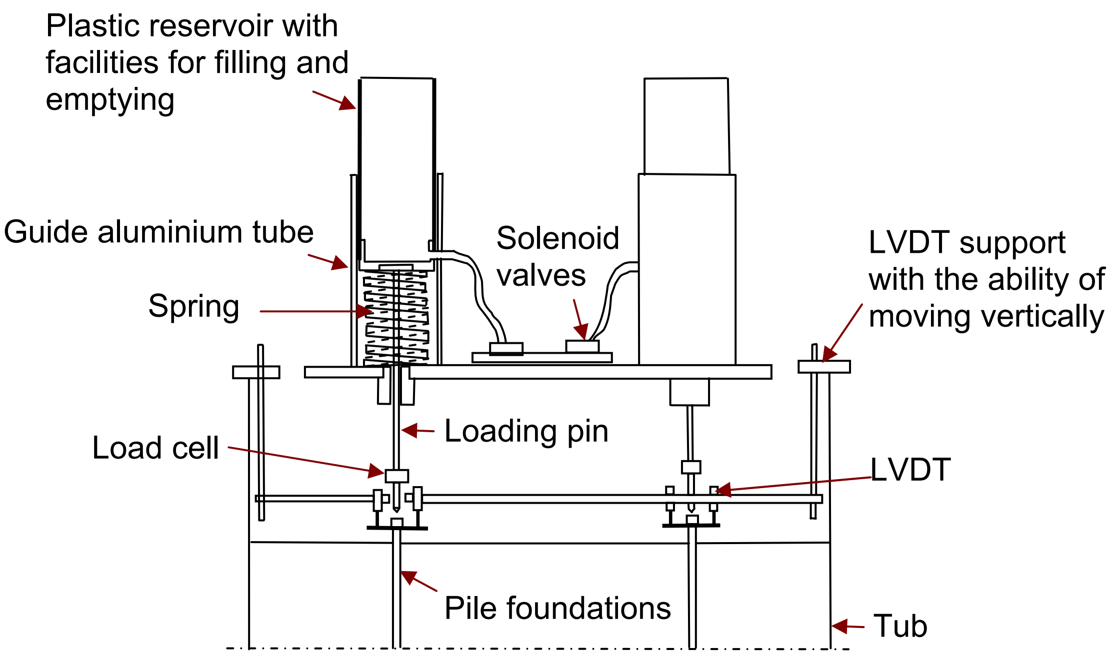


Figure 4.10 New loading apparatus developed for centrifuge model testing.

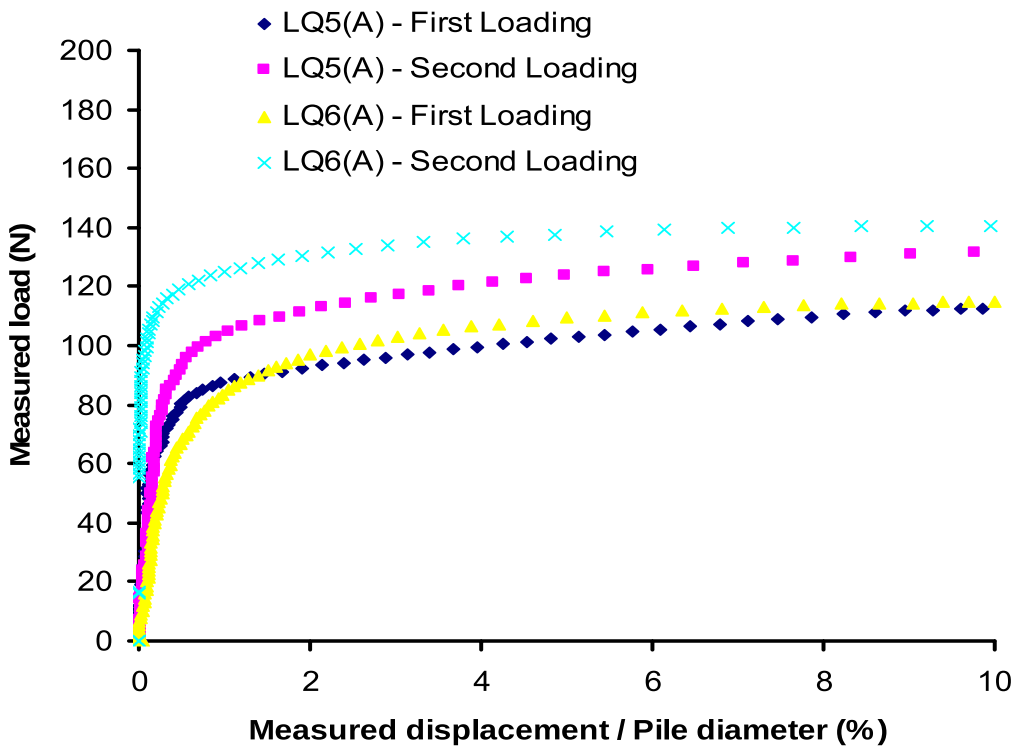


Figure 4.11 Test LQ5(A) and LQ(6) single pile foundations subjected to 1st and 2nd loading to failure. An increase in pile capacity of 20% was observed during 2nd loading.

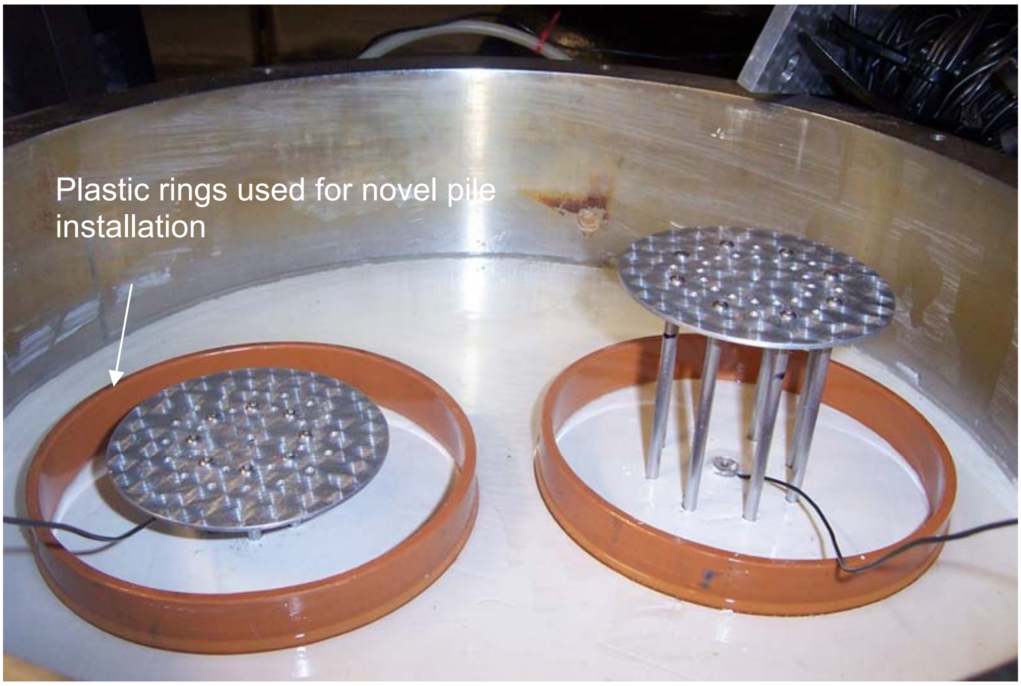


Figure 4.12 Plastic rings used to prevent the surface oil getting into bored holes during the installation of the model piles, after the model was spun up in centrifuge.

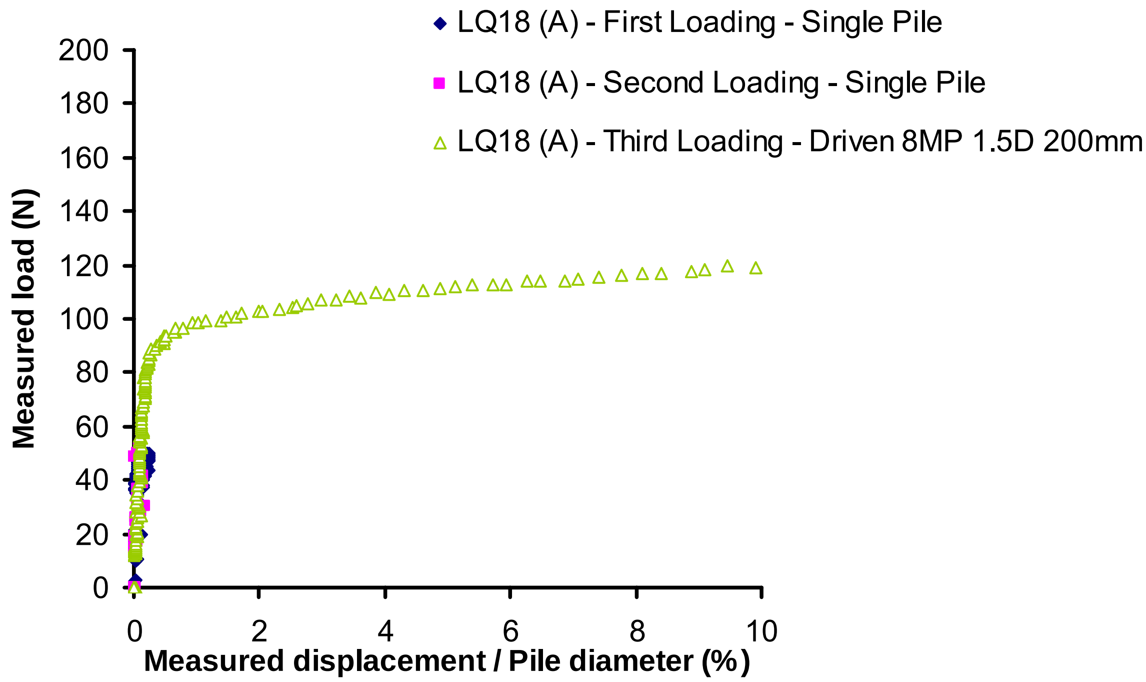


Figure 4.13 Test LQ18(A). Pile foundations subjected to three loading cycles. The centrifuge was not stopped between the first and second loading cycle.

Figure 4.14 The new loading reservoirs developed to increase the load applied during centrifuge testing of novel pile groups.

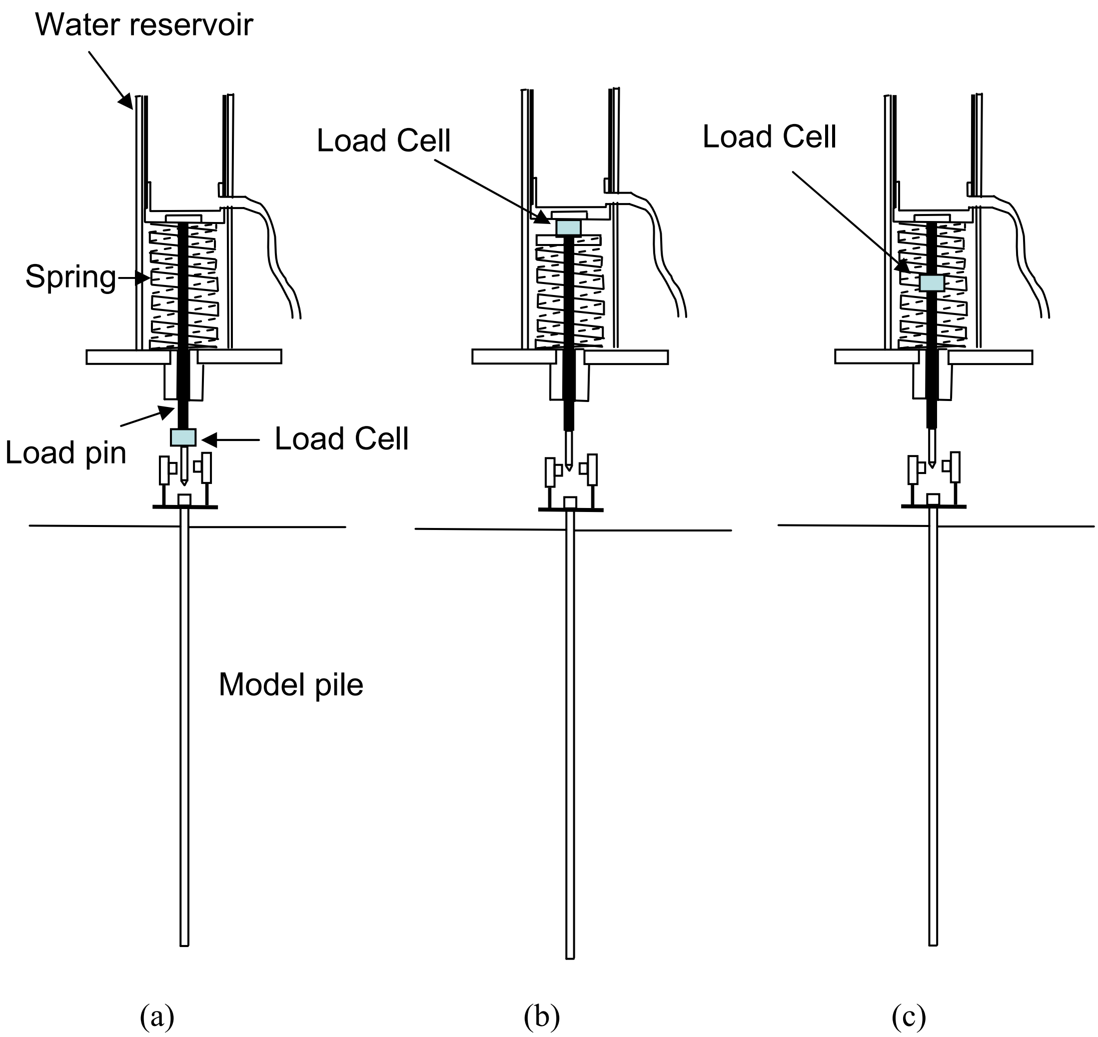


Figure 4.15 Details of the geometry of the loading apparatus for tests: (a) LQ1 to LQ19, (b) LQ20 and (c) LQ21.

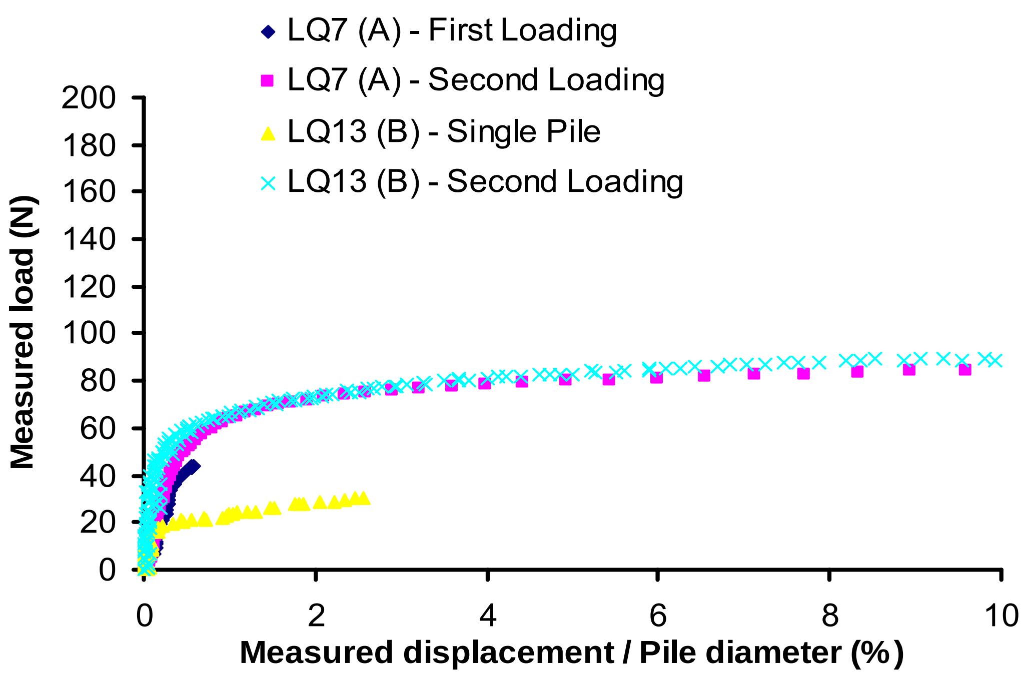


Figure 4.16 Tests LQ7(A) and LQ13(B) single pile foundations subjected to 1st loading and 2nd loading. The piles were subjected to working load during the 1st cycle. The piles were loaded to failure during the 2nd cycle.

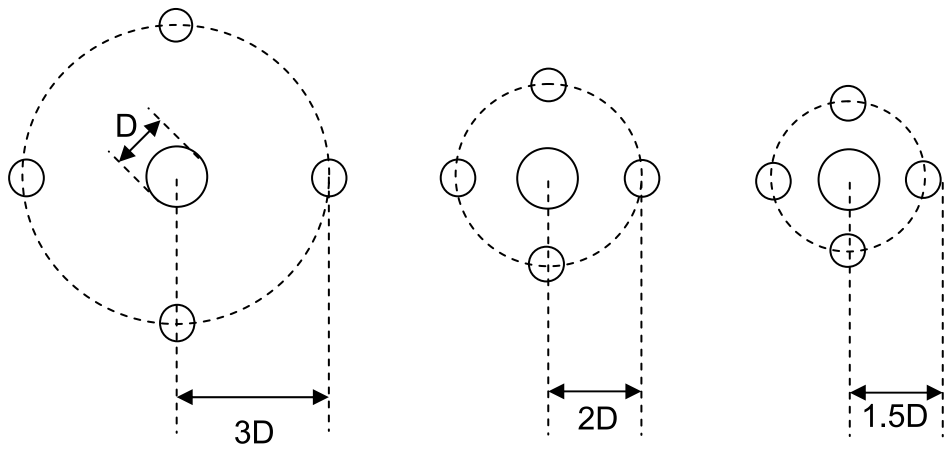


Figure 4.17 Details of the geometry of the novel pile groups used to investigate the influence of centre to centre spacing between the centre pile and the mini-pile group.

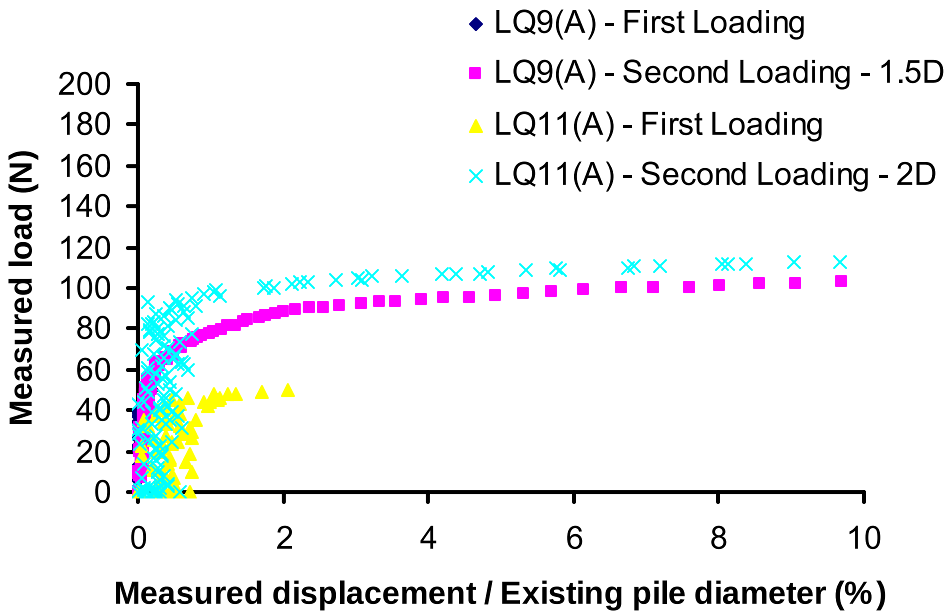


Figure 4.18 1st loading cycle - single pile foundation loaded up to working load (FOS=2.0), tests LQ9(A) and LQ11(A). 2nd loading cycle – enhanced pile foundations with 8 mini-piles 100mm long, test LQ9(A) with 1.5D spacing and test LQ11(A) with 2D spacing.

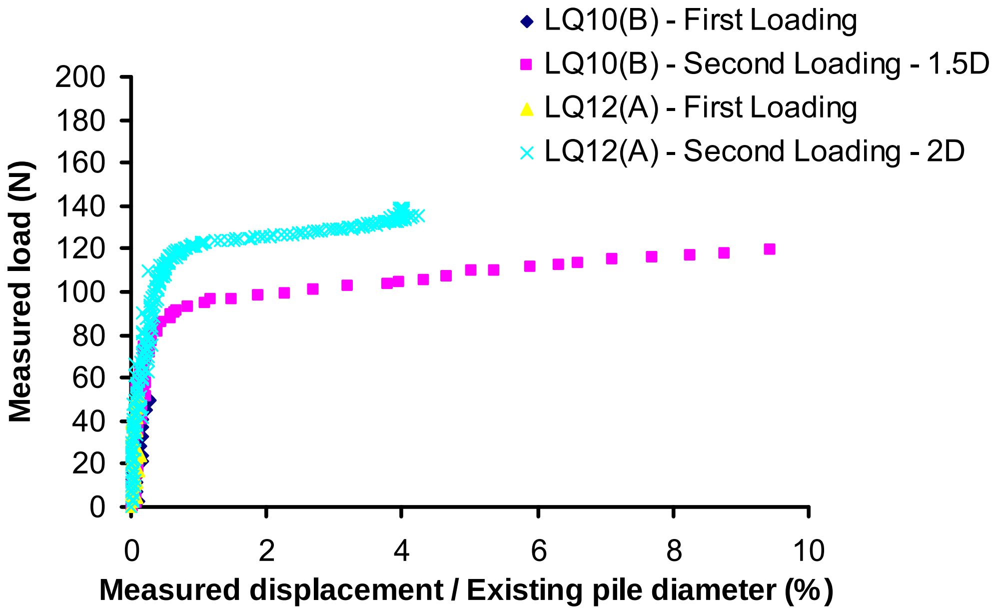


Figure 4.19 1st loading cycle - single pile foundation loaded up to working load (FOS=2.0), tests LQ10(B) and LQ12(A). 2nd loading cycle – enhanced pile foundations with 8 mini-piles 200mm long, test LQ10(B) with 1.5D spacing and test LQ12(A) with 2D spacing.

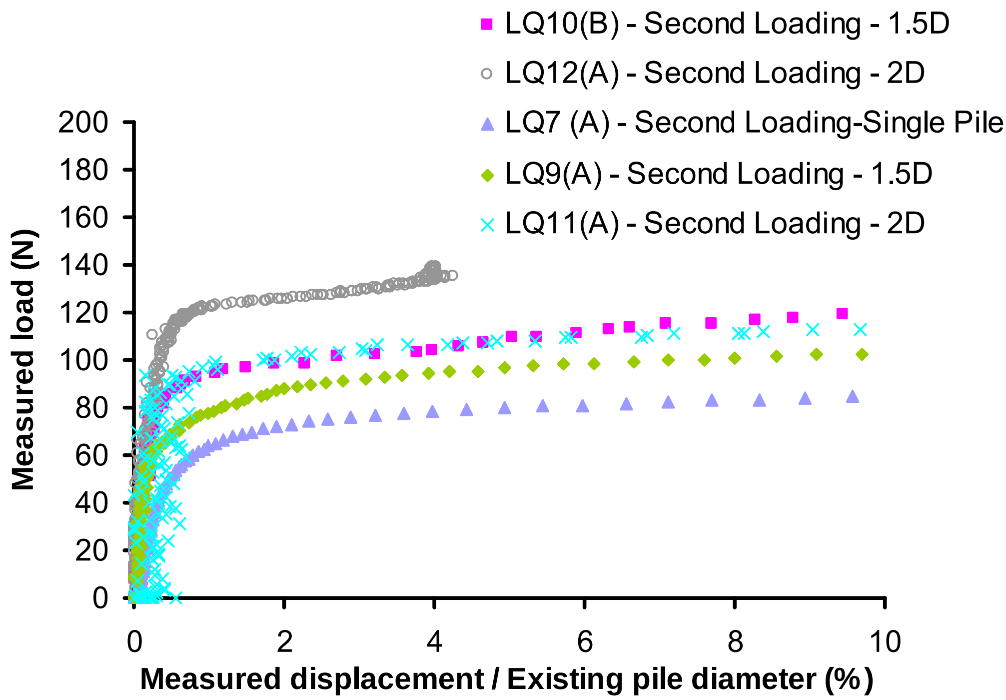


Figure 4.20 The performance of tests LQ7(A), LQ9(A), LQ10(B) and LQ12(A) during second loading cycle. During testing only the existing centre pile was loaded.

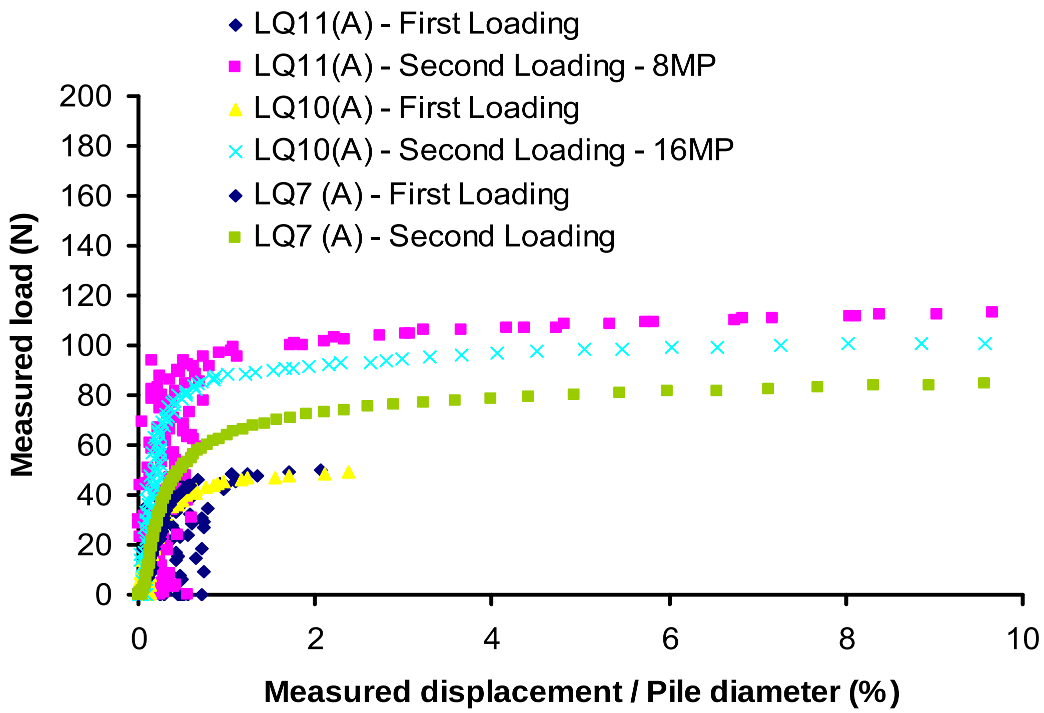


Figure 4.21 Tests LQ7(A), LQ10(A) and LQ11(A). 1st loading cycle – single pile foundations loaded up to working load. 2nd loading cycle – enhanced piled foundations loaded to failure: test LQ7(A) – single pile, test LQ10(A) enhanced pile foundation with sixteen 100mm long mini-piles at 3D spacing and test LQ11 enhanced pile foundation with eight 100mm long mini-piles at 3D spacing.

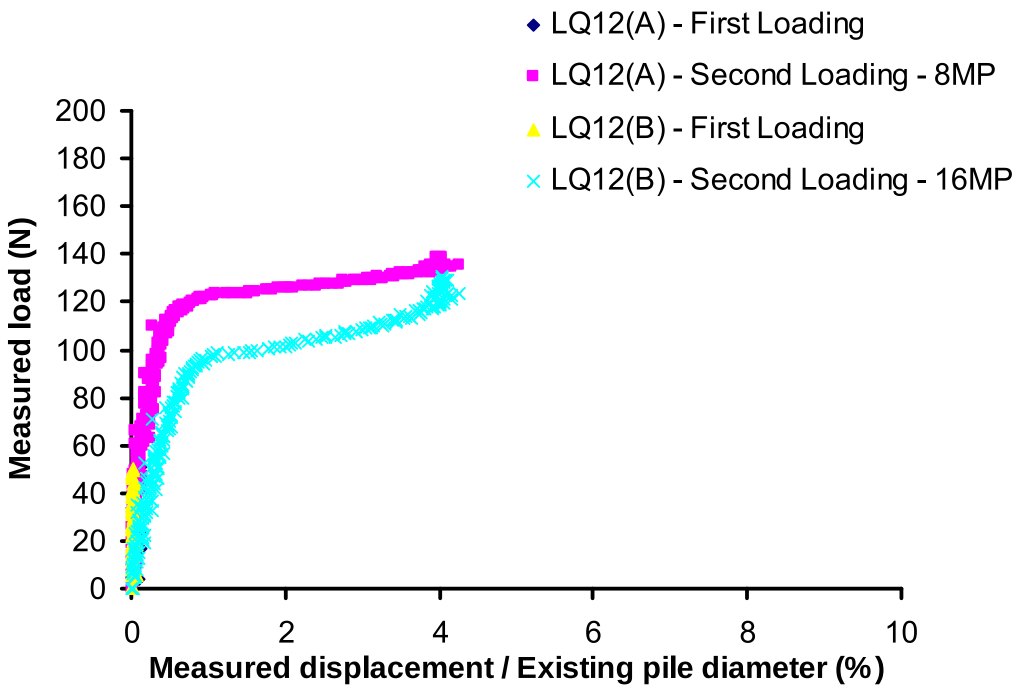


Figure 4.22 Tests LQ12(A) and LQ12(B). 1st loading cycle – single pile foundation loaded up to working load. 2nd loading cycle – enhanced pile foundation: Test LQ12(A) eight 200mm long mini-piles with 2D spacing; Test LQ12(B) sixteen 200mm long mini-piles with 2D spacing.

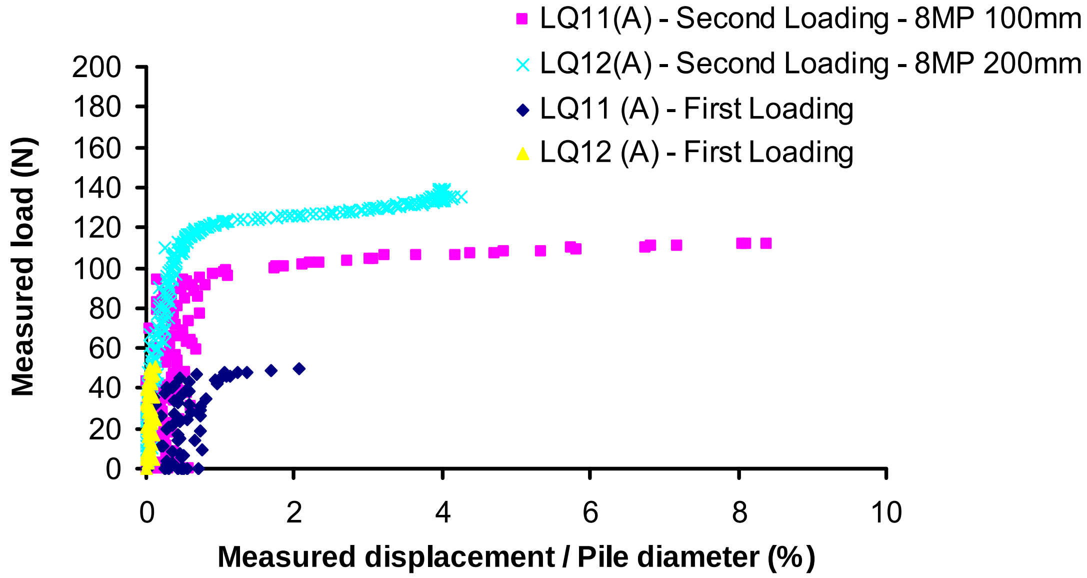


Figure 4.23 Tests LQ11(A) and LQ12(A). 1st loading cycle – single pile foundation loaded up to working load. 2nd loading cycle – enhanced pile foundation: Test LQ11(A) eight 100mm long mini-piles with 2D spacing; Test LQ12(A) eight 200mm long mini-piles with 2D spacing.

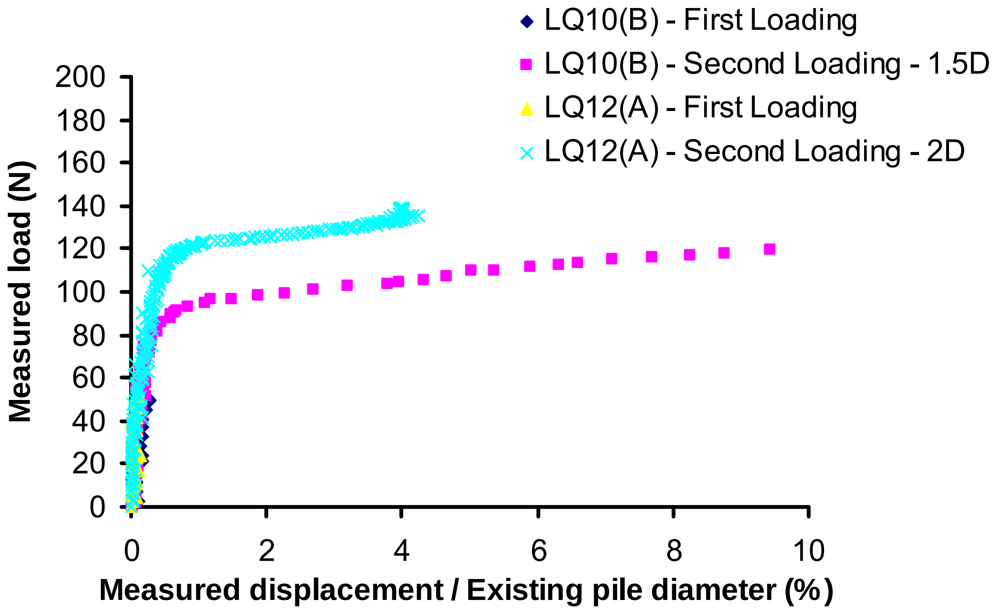


Figure 4.24 Tests LQ10(B) and LQ12(A). 1st loading cycle – single pile foundations loaded up to working load. 2nd loading cycle – enhanced pile foundations with eight 200mm long mini-piles. Test LQ10(B) with 1.5D pile spacing. Test LQ12(A) with 2D pile spacing.

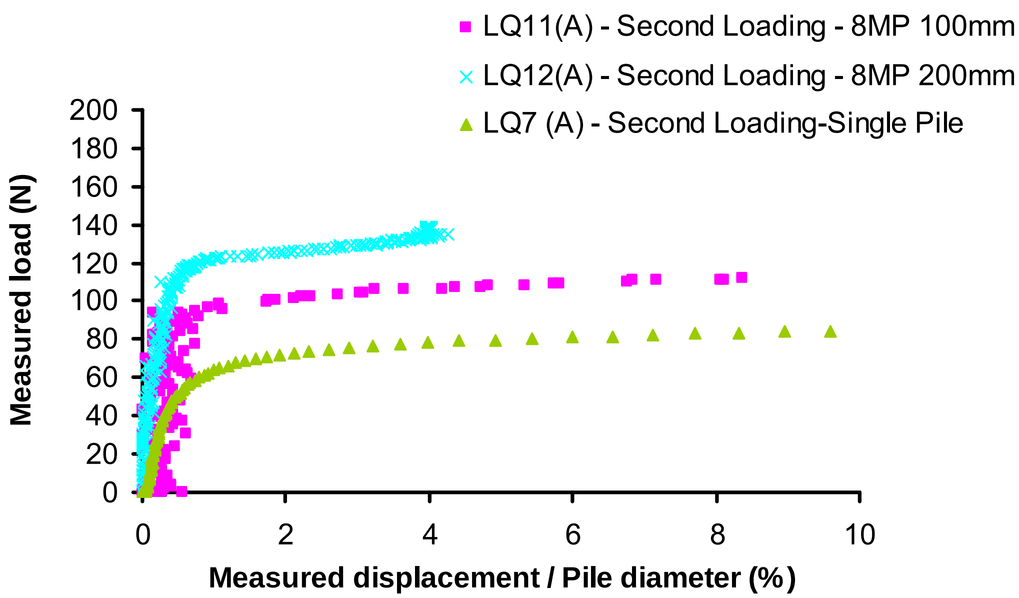


Figure 4.25 The performance of tests LQ7(A), LQ11(A) and LQ12(A) during 2nd loading cycle. In all tests only the centre pile was loaded.

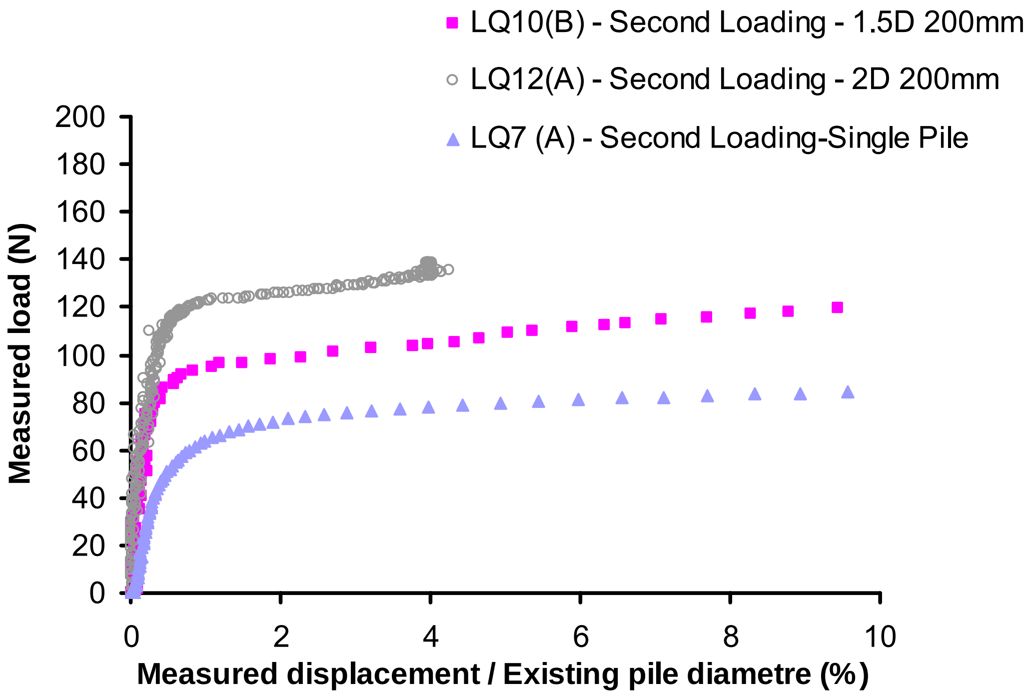


Figure 4.26 The performance of tests LQ7(A), LQ10(B) and LQ12(B) during 2nd loading cycle. In all tests only the centre pile was loaded.

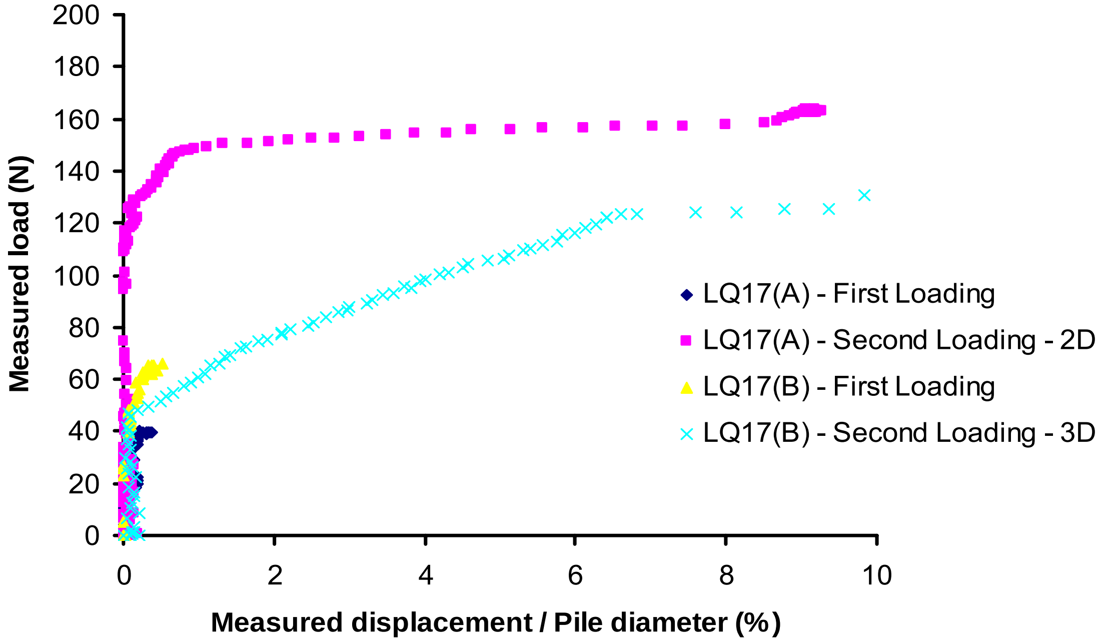


Figure 4.27 Tests LQ17(A) and LQ17(B). 1st loading cycle – single pile foundations loaded up to working load. 2nd loading cycle – novel pile group: Test LQ17(A) four 200mm long mini-piles with 3D spacing; Test LQ17(B) four 200mm long mini-piles with 3D spacing.

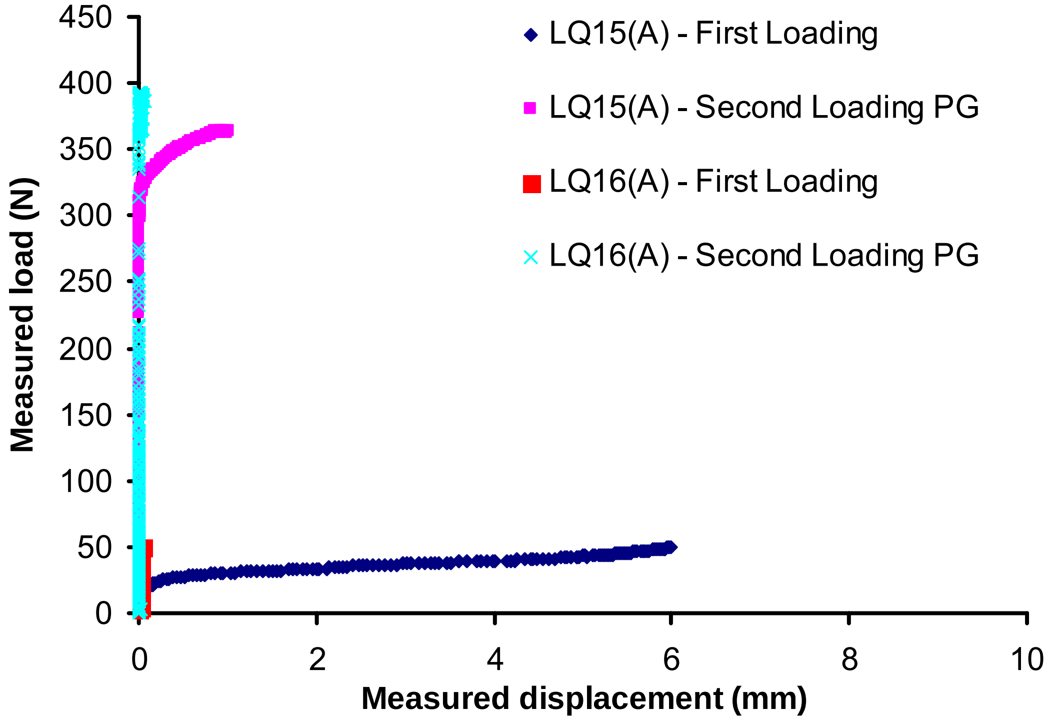


Figure 4.28 Tests LQ15(A) and LQ16(A). 1st loading cycle – single pile foundations loaded up to working load. 2nd loading cycle – novel pile group: Test LQ15(A) eight 200mm long mini-piles with 3D spacing; Test LQ16(A) eight 200mm long mini-piles with 2D spacing.

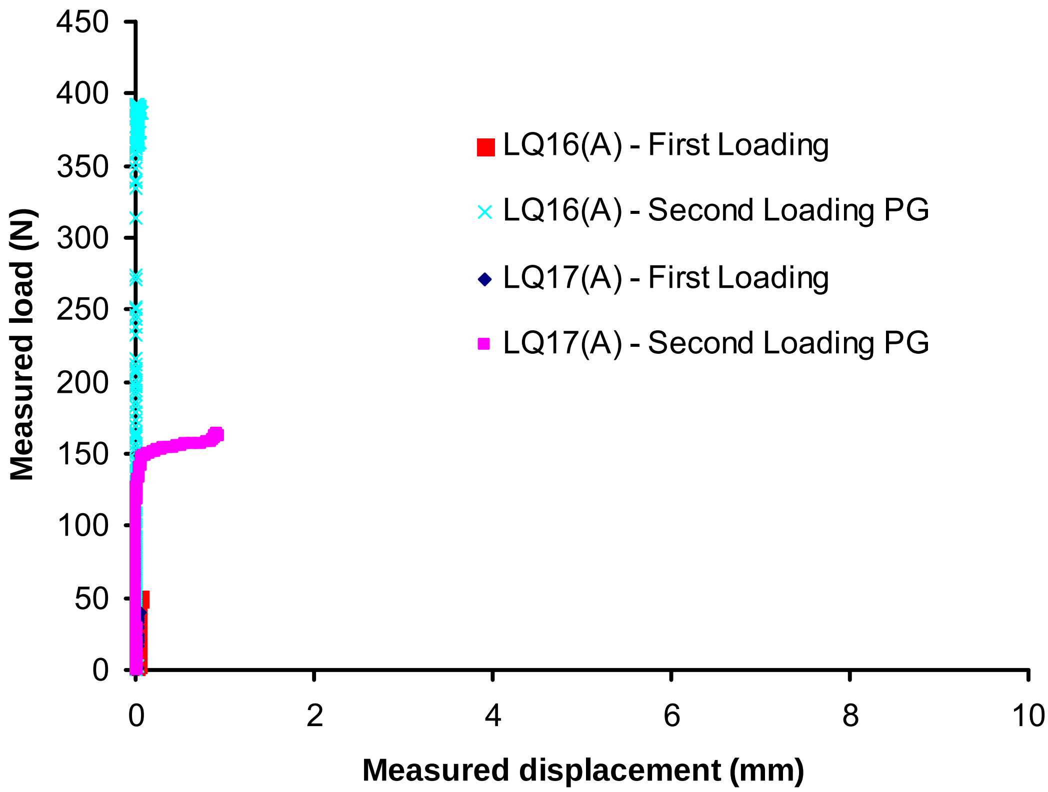


Figure 4.29 Tests LQ16(A) and LQ17(A). 1st loading cycle – single pile foundations. 2nd loading cycle – novel pile group: Test LQ16(A) eight 200mm long mini-piles with 2D spacing; Test LQ17(A) four 200mm long mini-piles with 2D spacing.

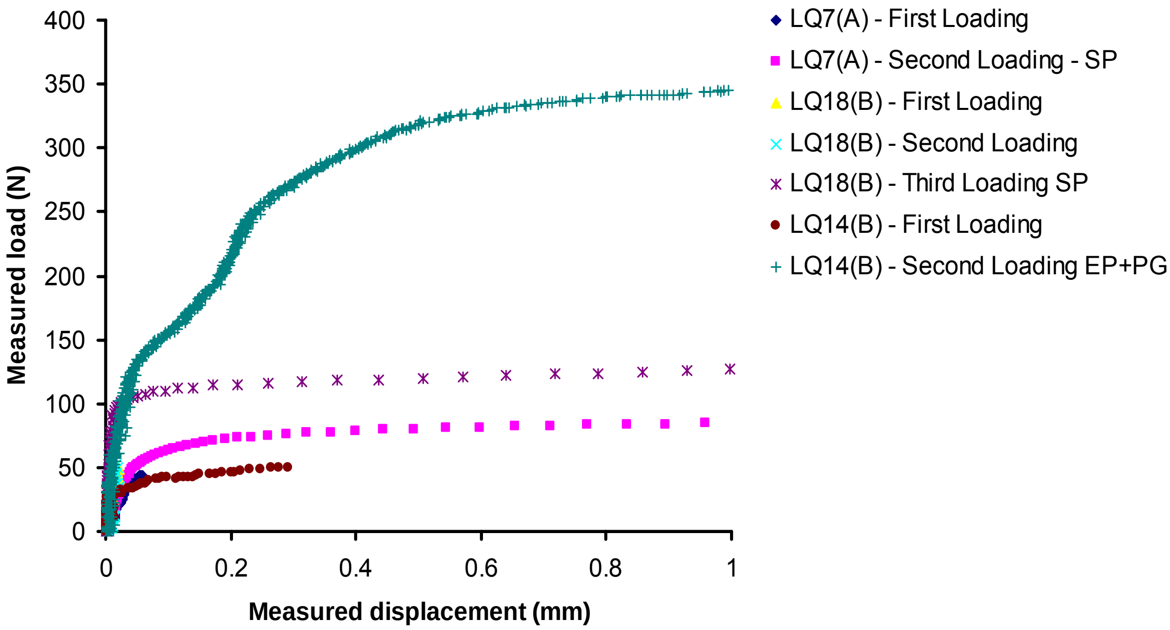


Figure 4.30 Tests LQ7(A), LQ14(B) and LQ18(B). 1st loading cycle – single pile foundation. 2nd loading cycle – Test LQ7(A) single pile foundation; Test LQ14(B) novel pile group (centre pile enhanced with eight 100mm long mini-piles with 3D spacing; Test LQ18(B) single pile foundation (centrifuge was not stopped between the first and second loading cycle). 3rd loading cycle – test LQ18(B) centre pile enhanced with eight 100mm long mini-piles with 3D spacing.

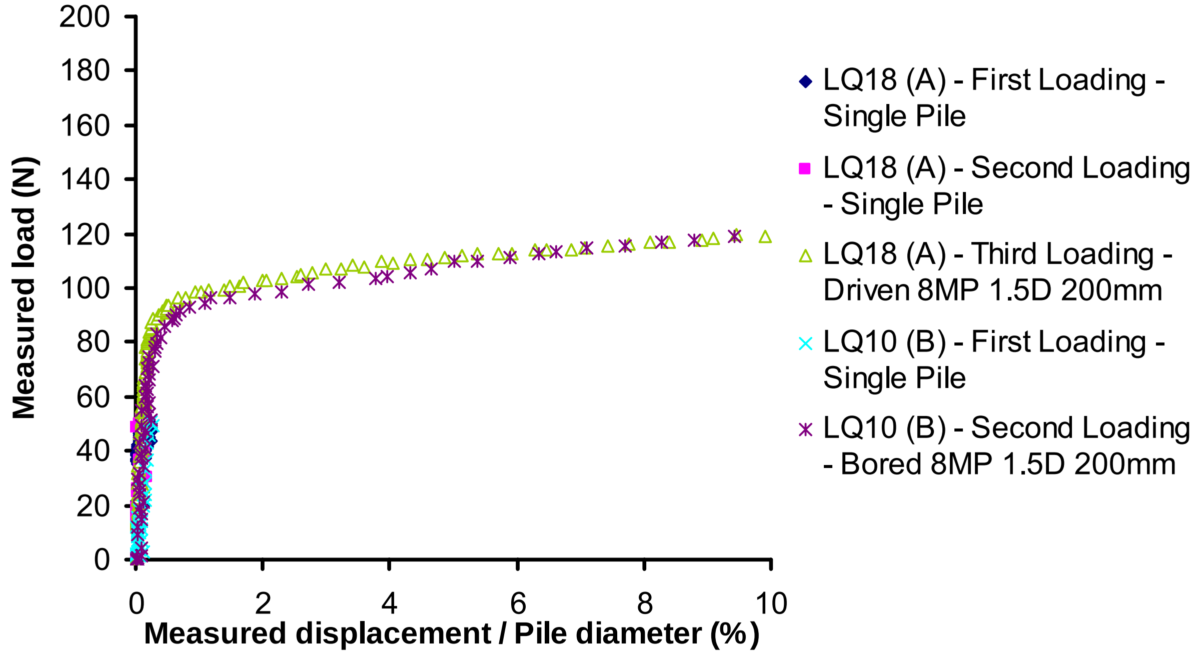


Figure 4.31 Tests LQ10(B) and LQ18(A). 1st loading cycle up to working load – single pile foundation. 2nd loading cycle – Tests LQ10(B) centre pile enhanced with eight bored 200mm long mini-piles with 1.5D spacing; Test LQ18(A) – single pile foundation (centrifuge was not stopped between the first and second loading cycle). 3rd loading cycle – test LQ18(B) centre pile enhanced with eight driven 200mm long mini-piles with 1.5D spacing.

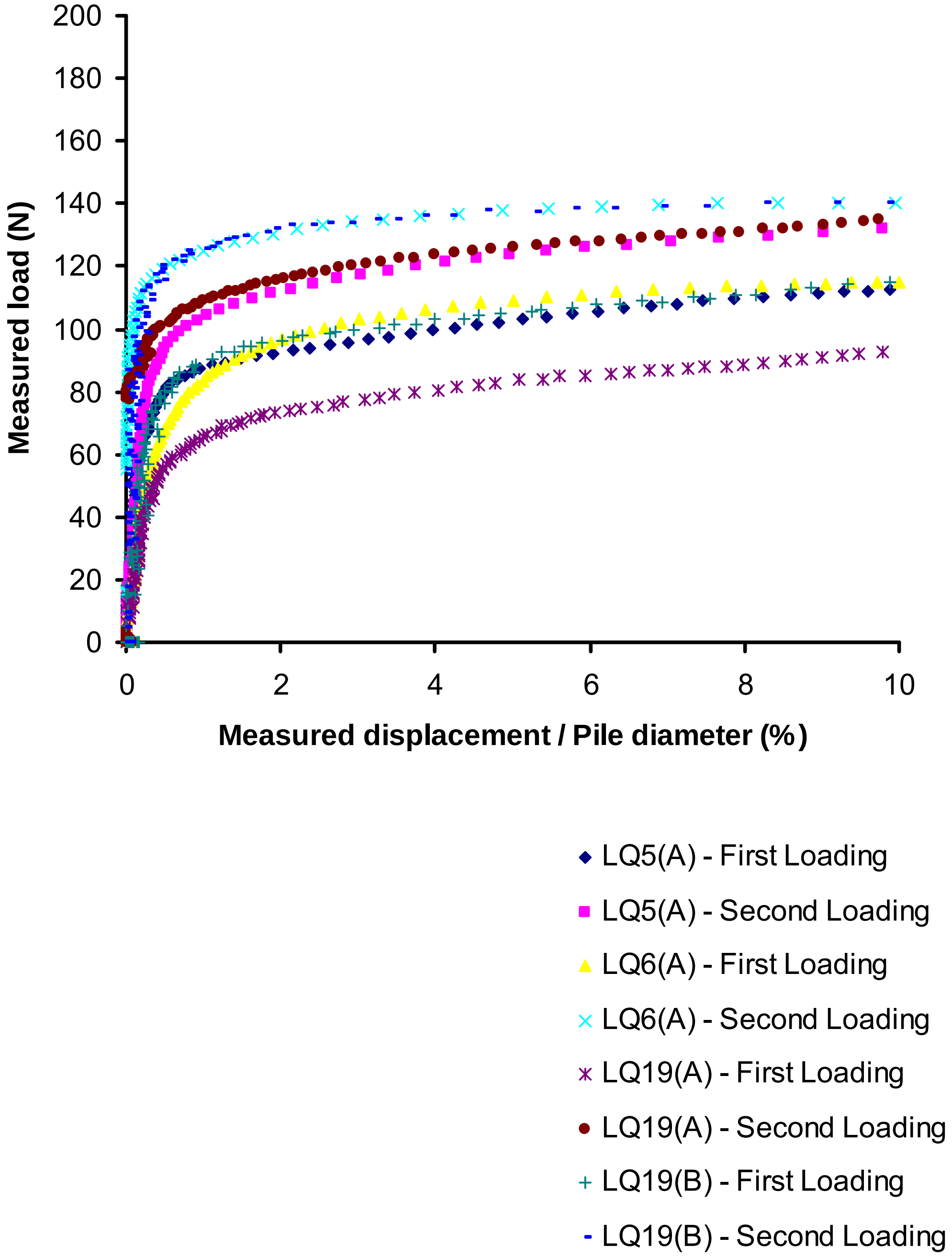


Figure 4.32 Performance of single pile foundations during 1st and 2nd loading. Tests LQ5(A) and LQ6(A) – centrifuge was stopped between the first and second loading. Tests LQ19(A) and LQ19(B) – centrifuge was NOT stopped between the first and second loading.

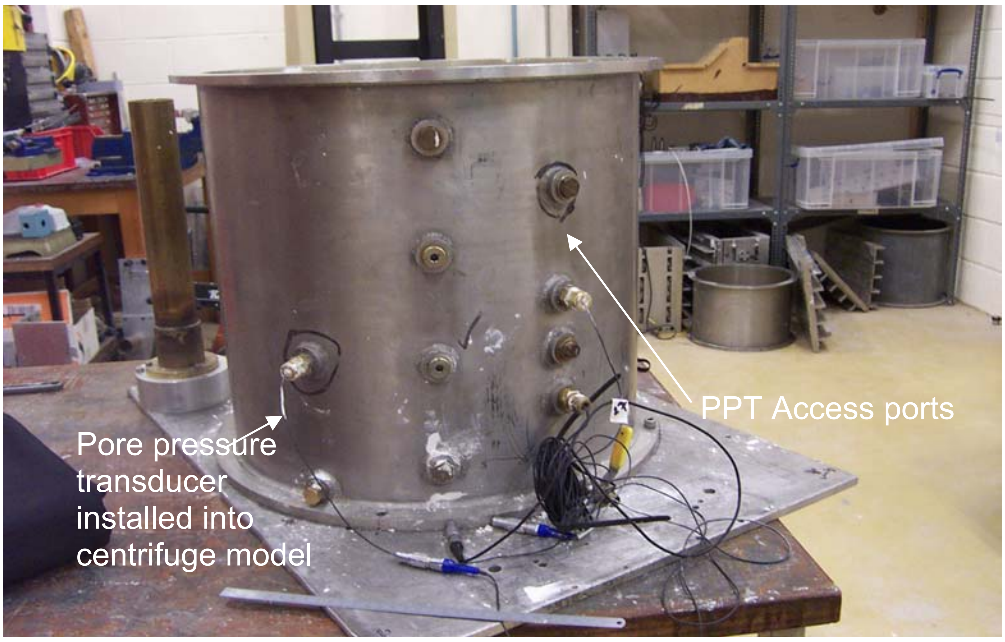


Figure 4.33 Centrifuge stainless steel tub with access ports through which the pore pressure transducers were installed.

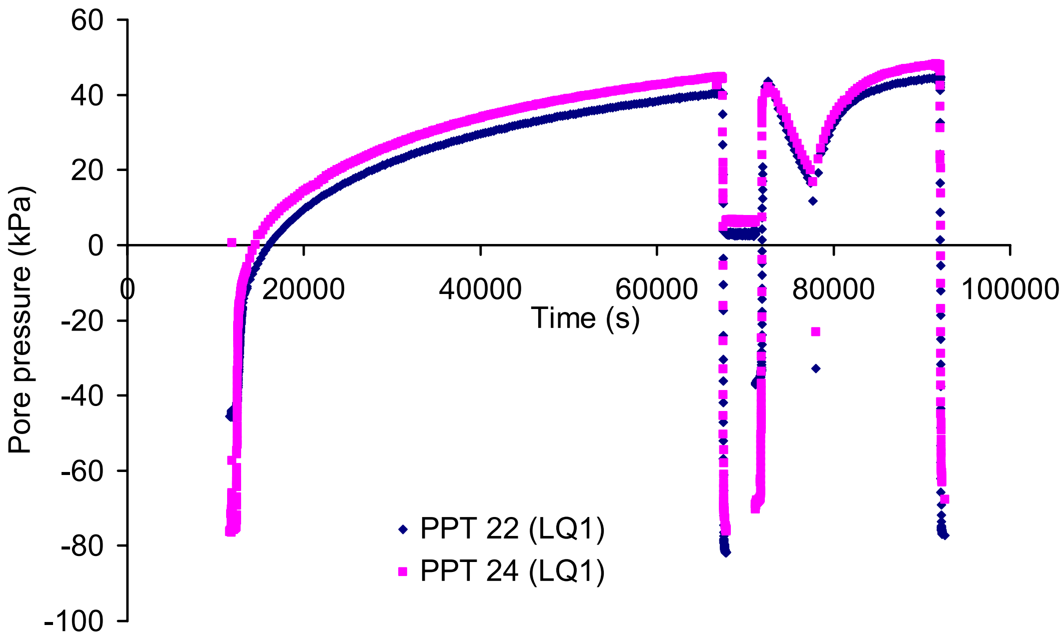


Figure 4.34 Typical pore pressure measurements with time during centrifuge model testing (Test LQ1).

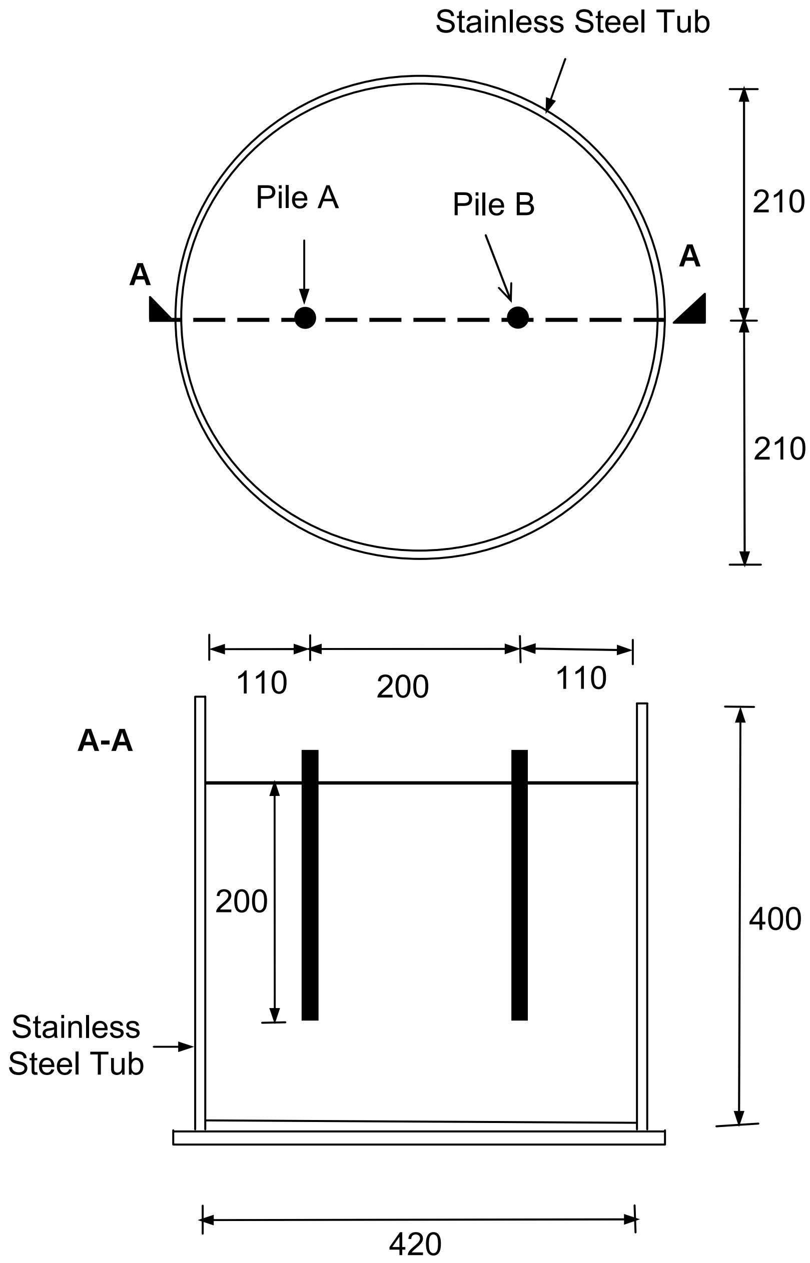


Figure 5.1 Geometry of a typical model used for centrifuge testing.

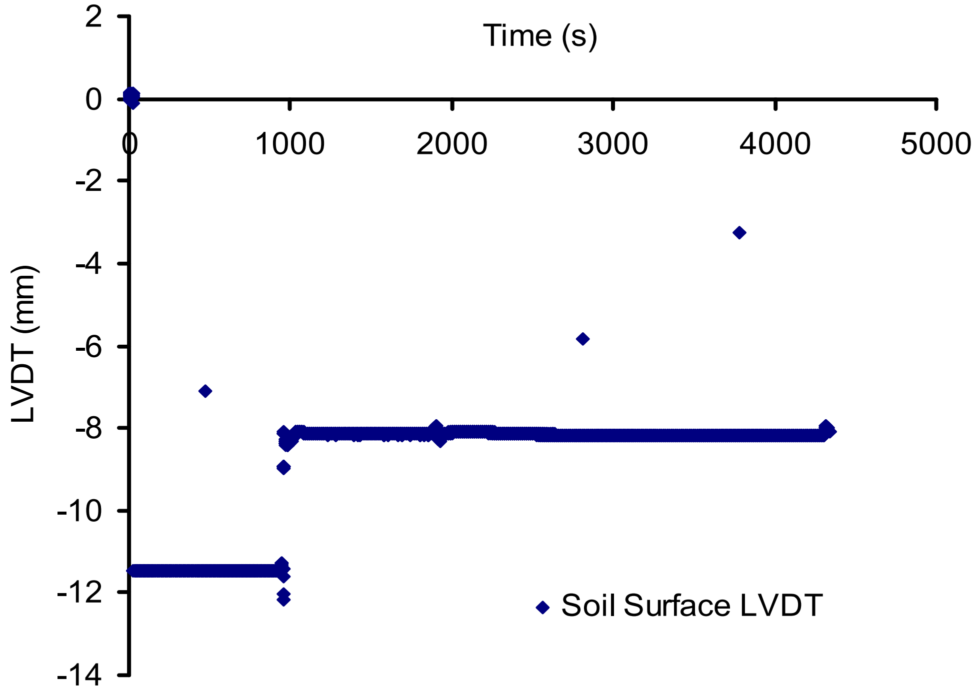


Figure 5.2 Typical soil surface compression and swelling measurements with time during centrifuge model testing (Test LQ6).

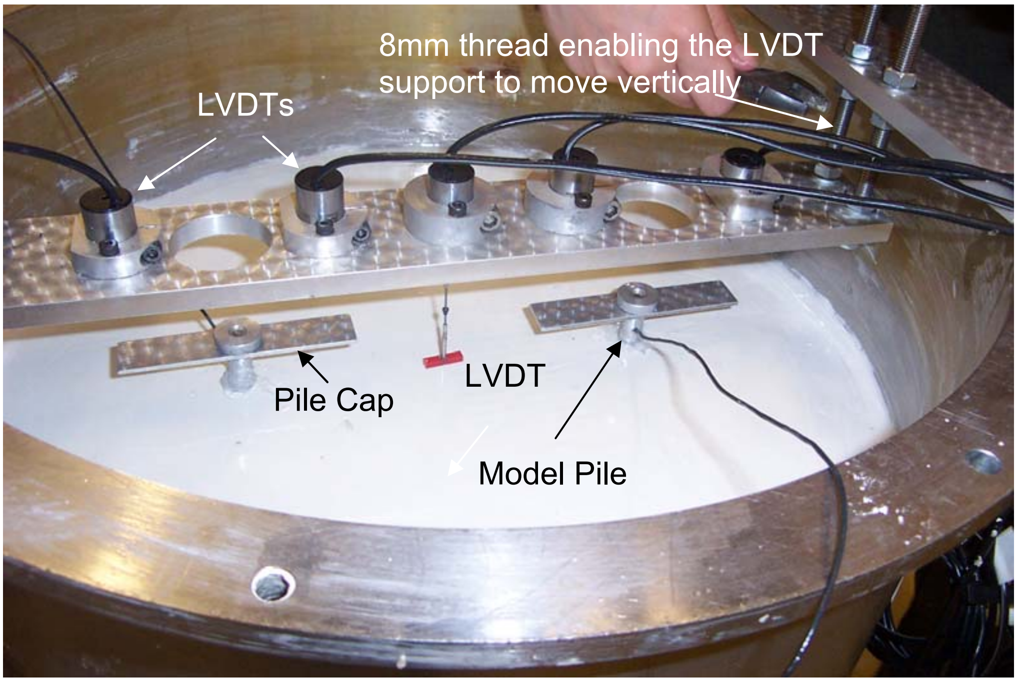


Figure 5.3 The LVDTs used to measure pile settlements during loading and changes in the model soil surface during testing.

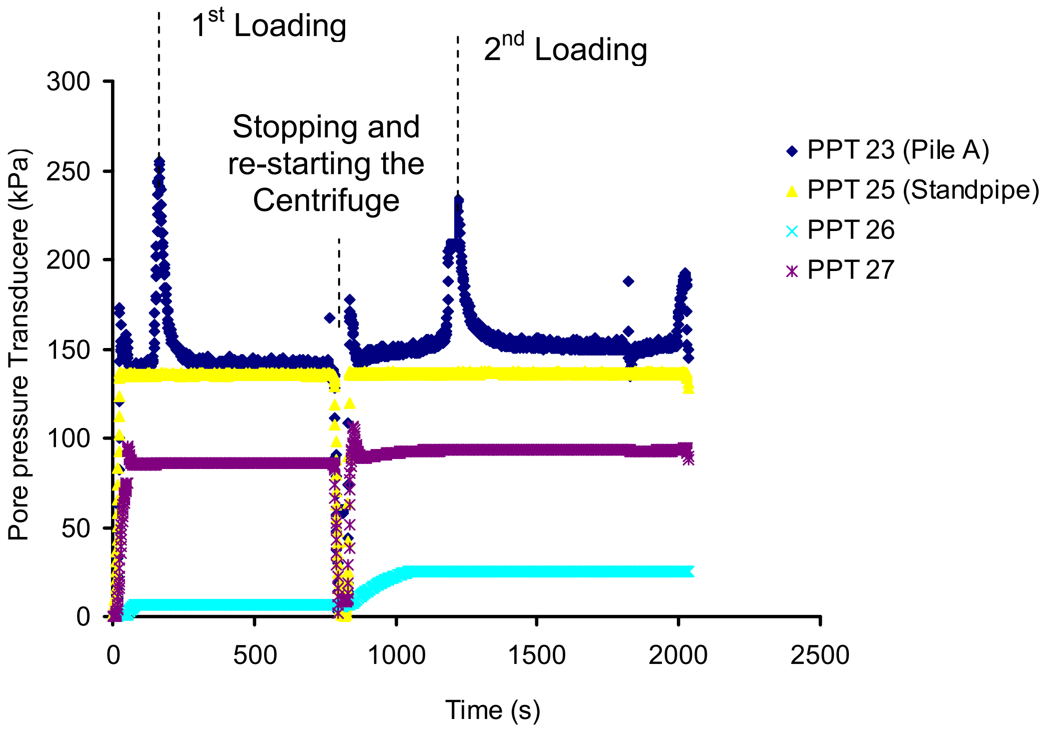


Figure 5.4 Pore pressure transducer response during centrifuge model testing.

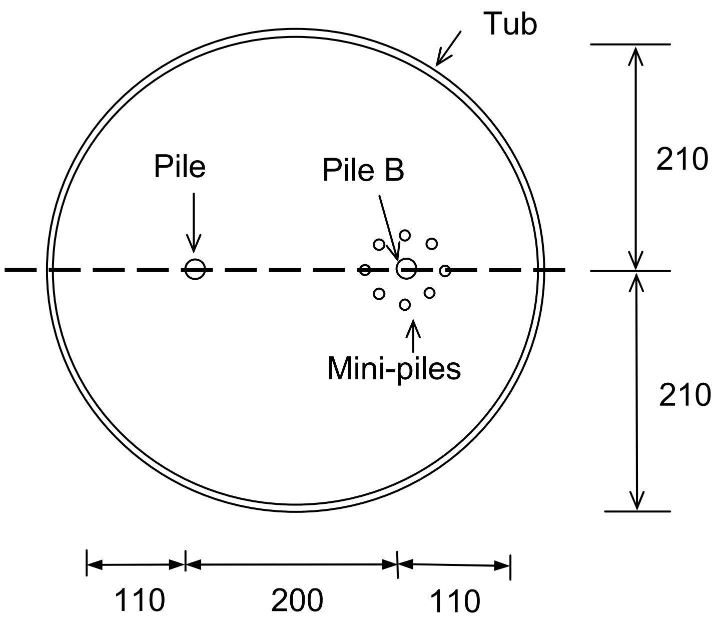


Figure 5.5 Typical geometry of model used to investigate the performance of enhanced pile foundation.

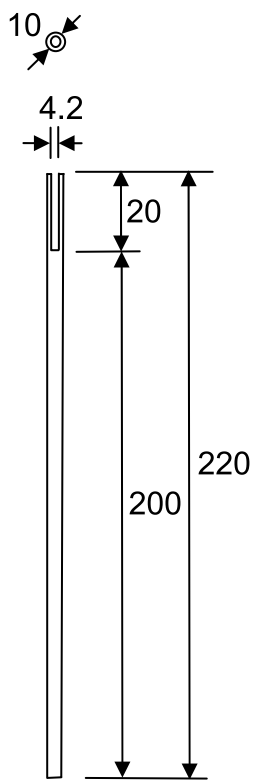


Figure 5.6 Aluminium model pile used for centrifuge model testing.

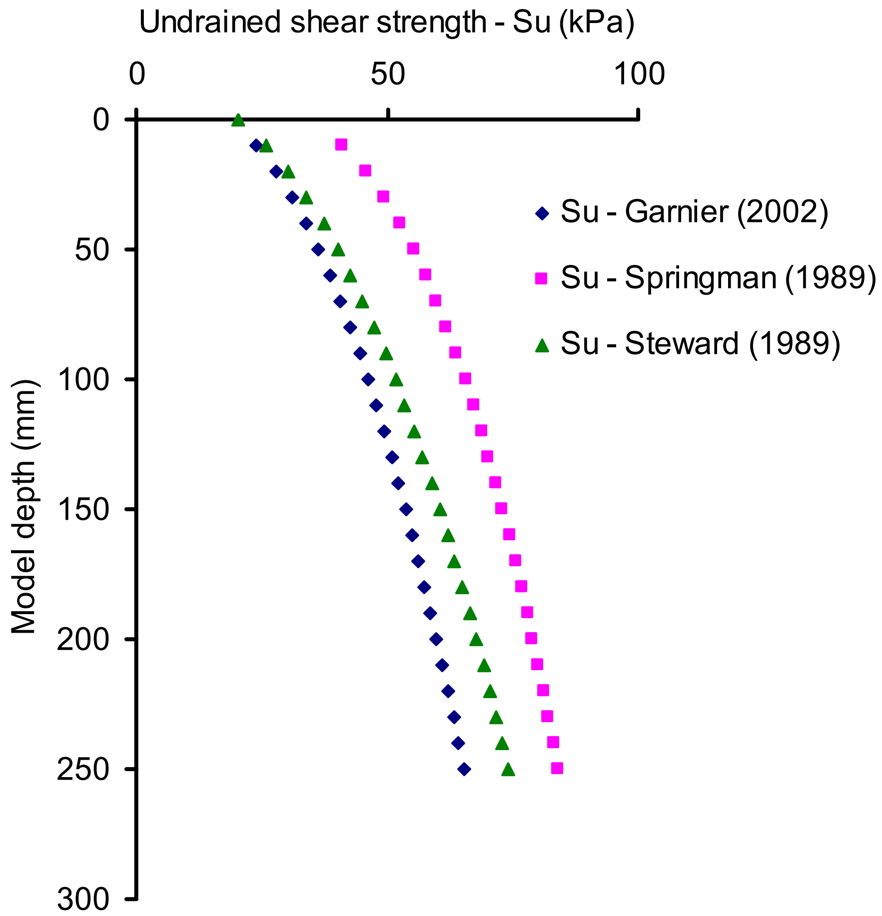


Figure 5.7 Distribution of undrained shear strength after equilibrium of the centrifuge model was reached at 60g based on the findings by Garnier (2002), Springman (1989) and Steward (1989).

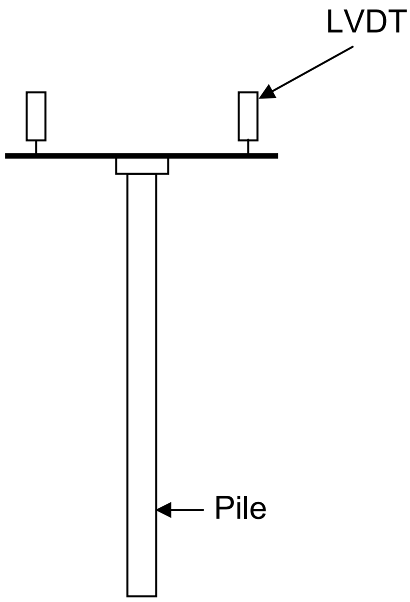


Figure 5.8 Geometry of the single pile model and the LVDTs used to measure displacements during testing.

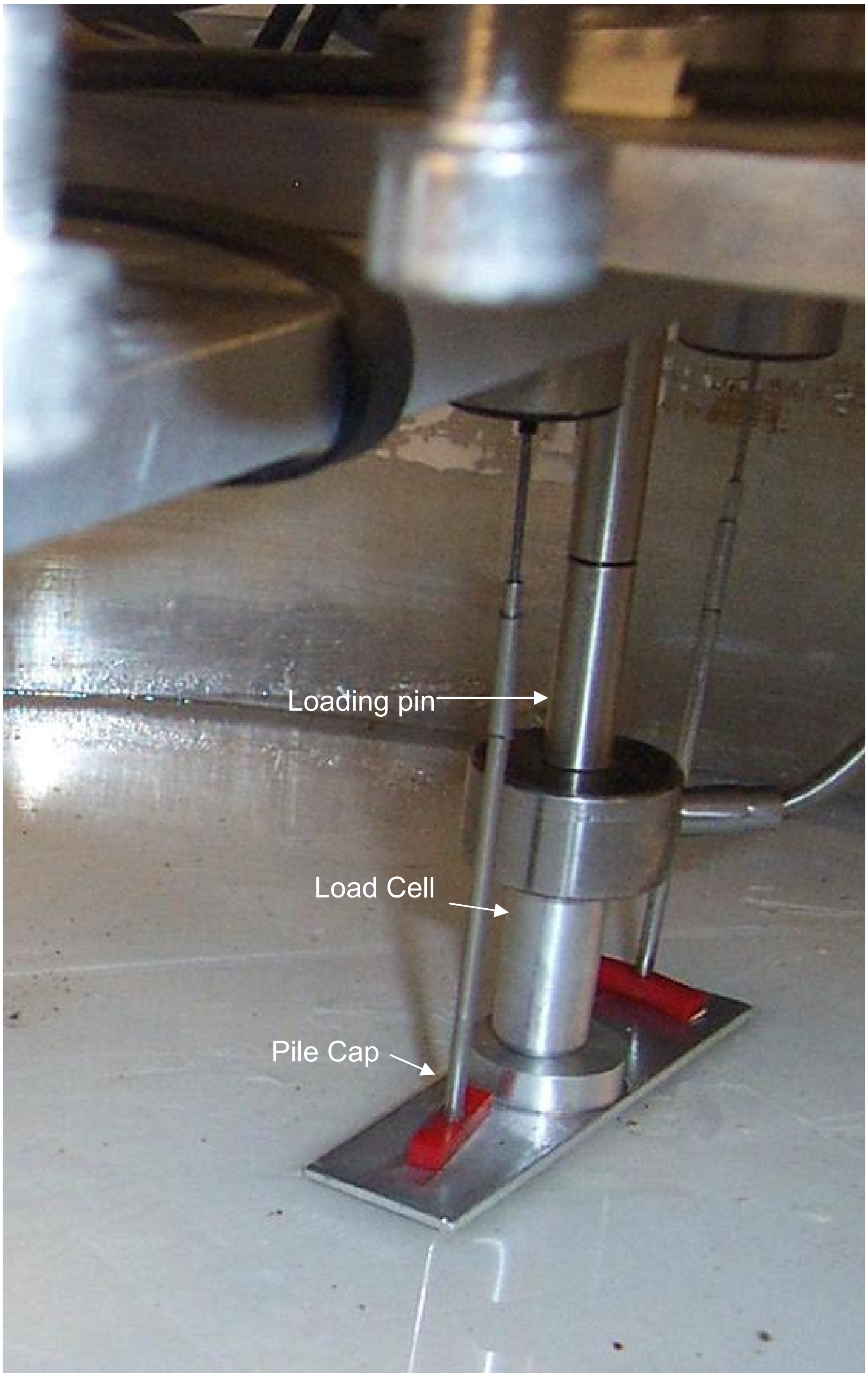


Figure 5.9 Loading pin used to axially load piles during centrifuge model testing and load cell used to measure the load applied.

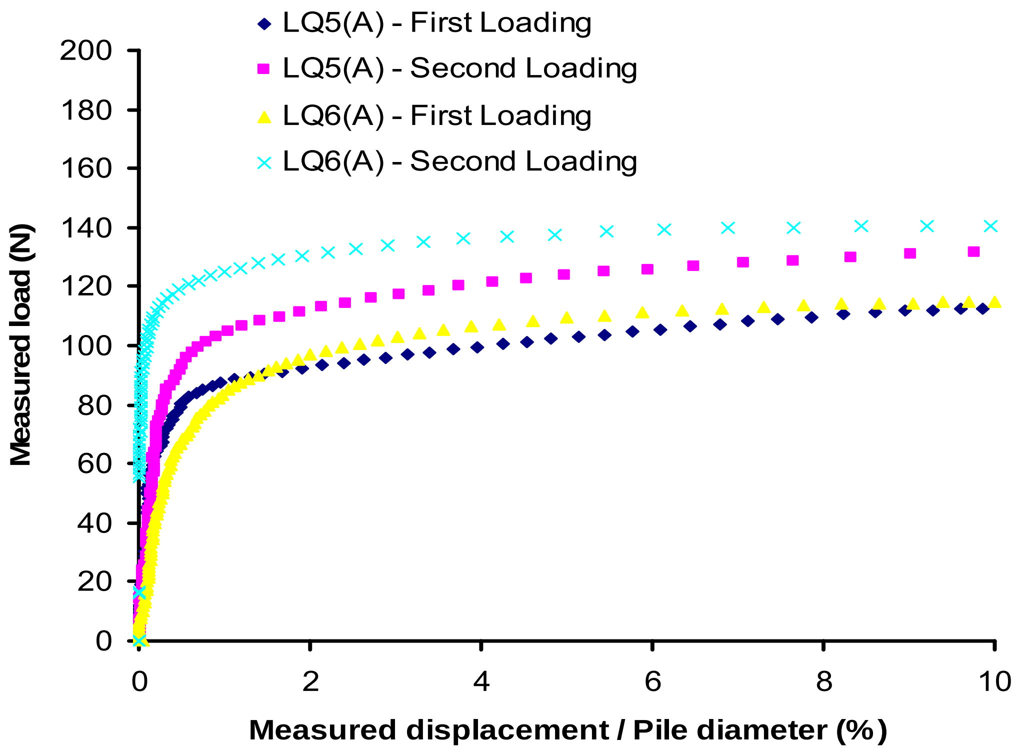


Figure 5.10 Test LQ5(A) and LQ(6) single pile foundations subjected to 1st and 2nd loading to failure. An increase in pile capacity of 20% was observed during 2nd loading.

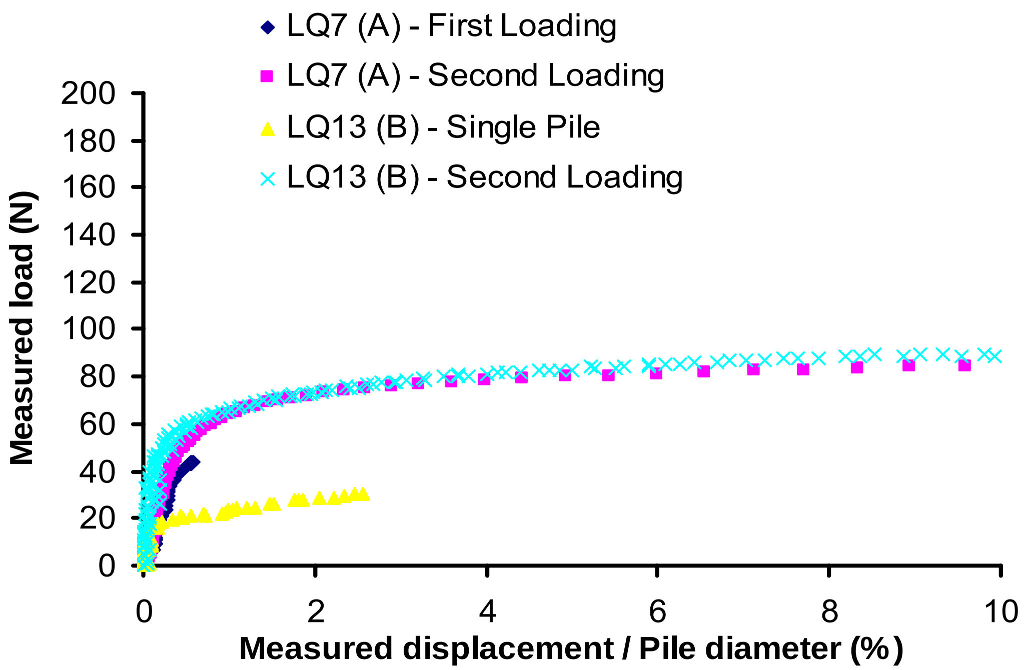


Figure 5.11 Tests LQ7(A) and LQ13(B) single pile foundations subjected to 1st and 2nd loading. The piles were subjected to working load during the 1st cycle. The piles were loaded to failure during the 2nd cycle.

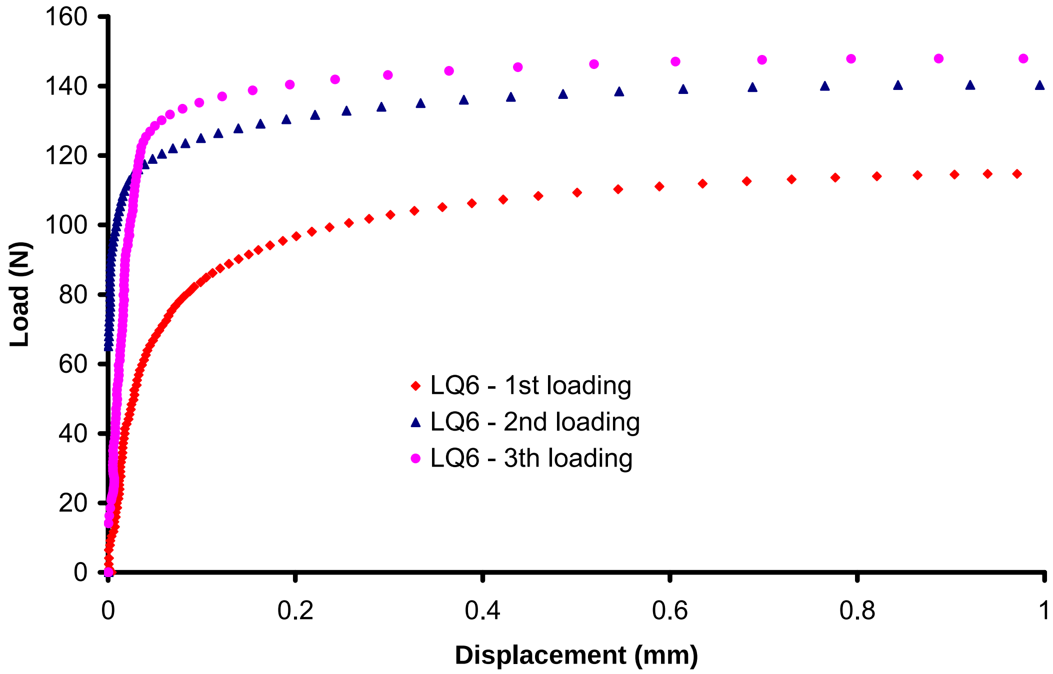


Figure 5.12 Tests LQ6 – Single pile foundations subjected to three loading cycles.

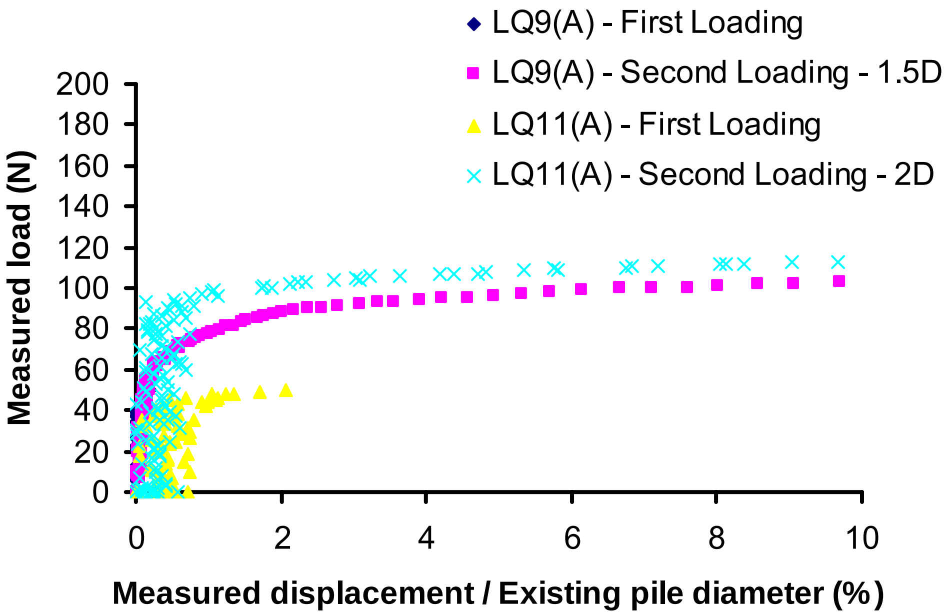


Figure 5.13 1st loading cycle - single pile foundation loaded up to working load (FOS=2.0), tests LQ9(A) and LQ11(A). 2nd loading cycle – enhanced pile foundations with 8 mini-piles 100mm long, test LQ9(A) with 1.5D spacing and test LQ11(A) with 2D spacing.

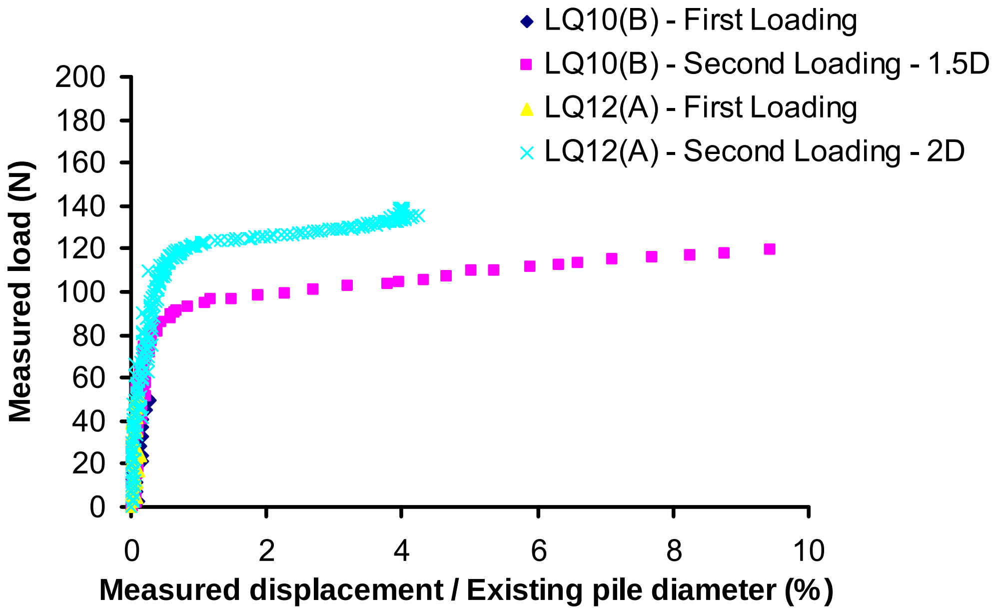


Figure 5.14 1st loading cycle - single pile foundation loaded up to working load (FOS=2.0), tests LQ10(B) and LQ12(A). 2nd loading cycle – enhanced pile foundations with 8 mini-piles 200mm long, test LQ10(B) with 1.5D spacing and test LQ12(A) with 2D spacing.

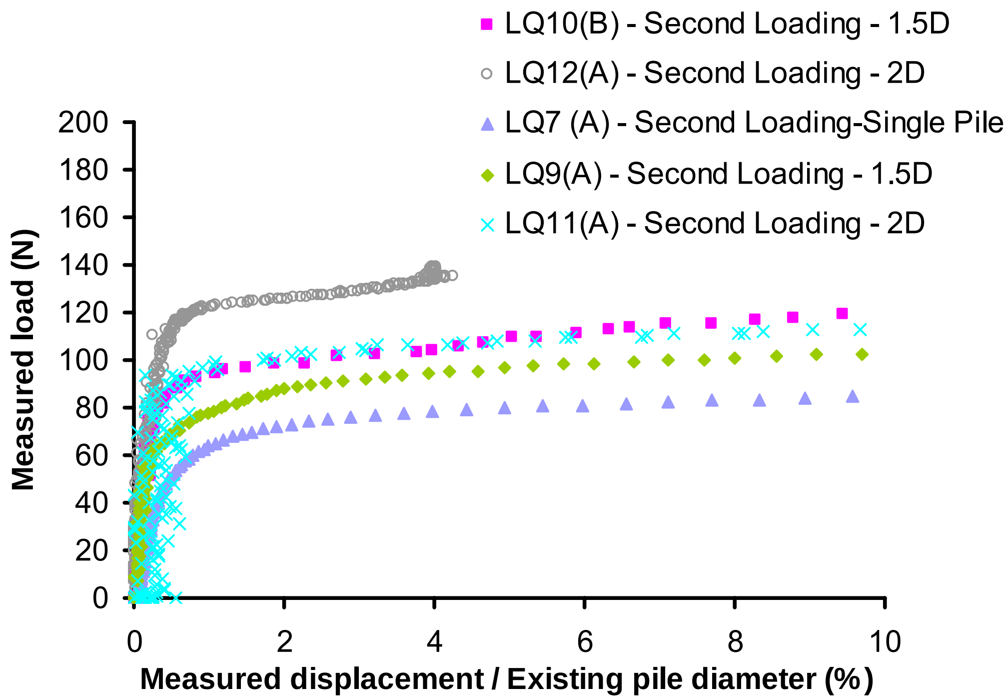


Figure 5.15 The performance of tests LQ7(A), LQ9(A), LQ10(B) and LQ12(A) during second loading cycle. During testing only the existing centre pile was loaded.

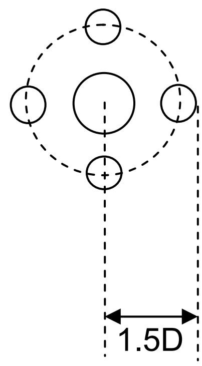


Figure 5.16 Details of the geometry of the novel pile groups used to investigate the influence of 1.5D centre to centre spacing between the centre pile and the mini-pile group (D – diameter of centre pile).

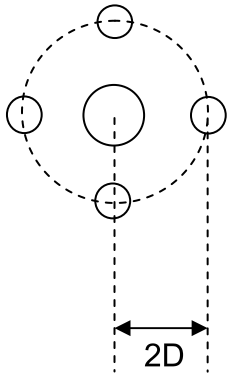


Figure 5.17 Details of the geometry of the novel pile groups used to investigate the influence of 2D centre to centre spacing between the centre pile and the mini-pile group (D – diameter of centre pile).

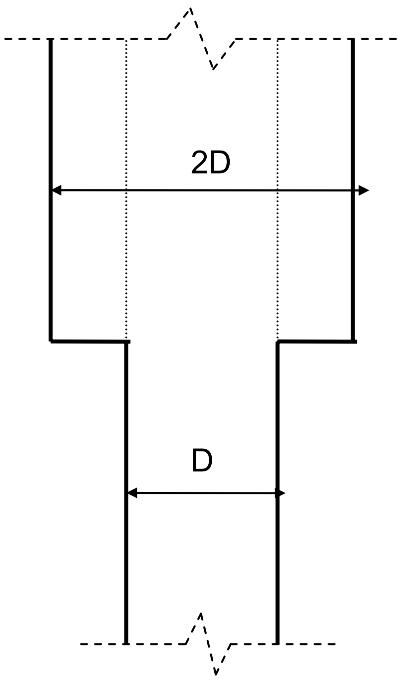


Figure 5.18 Effective geometry of the enhanced centre pile with 100mm long mini-piles installed at 1.5D centre to centre spacing with the existing centre pile.

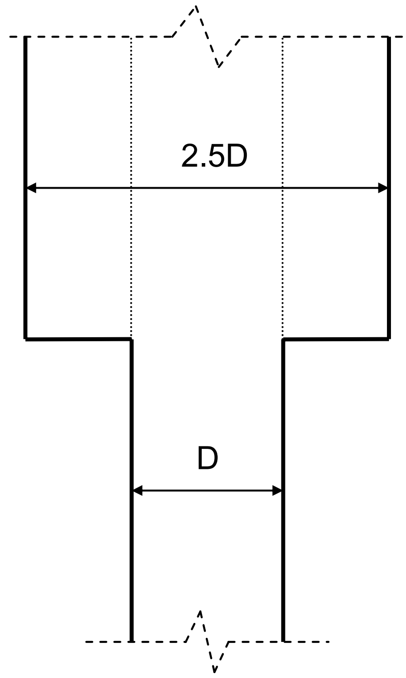


Figure 5.19 Effective geometry of the enhanced centre pile with 100mm long mini-piles installed at 2D centre to centre spacing with the existing centre pile.

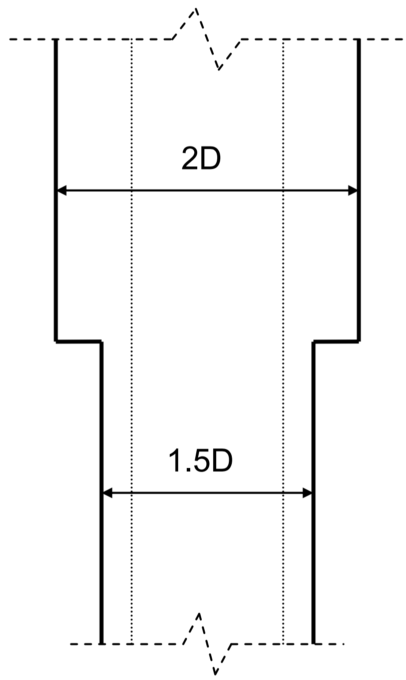


Figure 5.20 Effective geometry of the enhanced centre pile with 200mm long mini-piles installed at 1.5D centre to centre spacing with the existing centre pile.

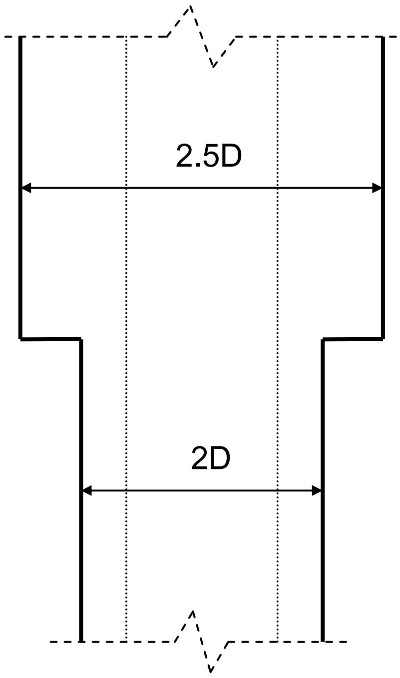


Figure 5.21 Effective geometry of the enhanced centre pile with 200mm long mini-piles installed at 2D centre to centre spacing with the existing centre pile.

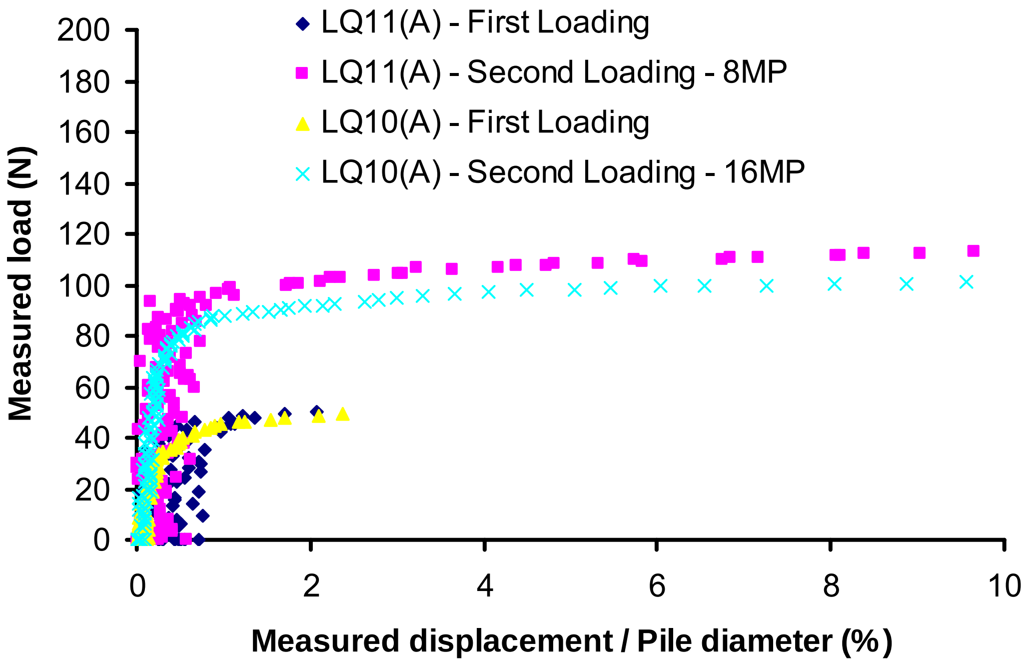


Figure 5.22 Tests LQ11(A) and LQ10(A). 1st loading cycle – single pile foundation loaded up to working load. 2nd loading cycle – enhanced pile foundation: Test LQ11(A) eight 100mm long mini-piles with 2D spacing; Test LQ10(A) sixteen 100mm long mini-piles with 2D spacing.

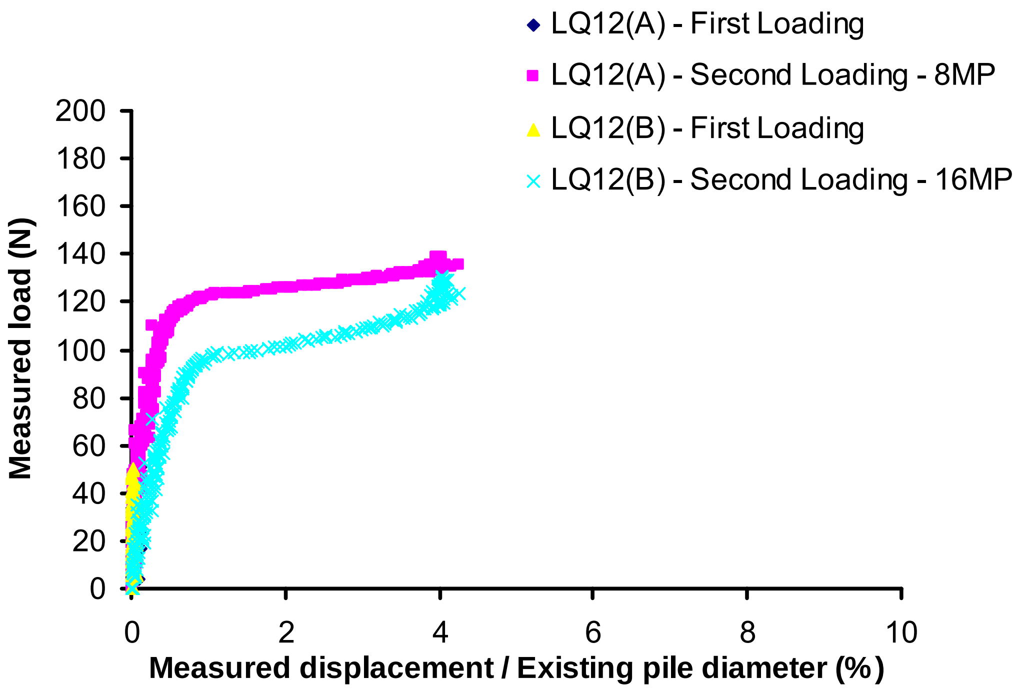


Figure 5.23 Tests LQ12(A) and LQ12(B). 1st loading cycle – single pile foundation loaded up to working load. 2nd loading cycle – enhanced pile foundation: Test LQ12(A) eight 200mm long mini-piles with 2D spacing; Test LQ12(B) sixteen 200mm long mini-piles with 2D spacing.

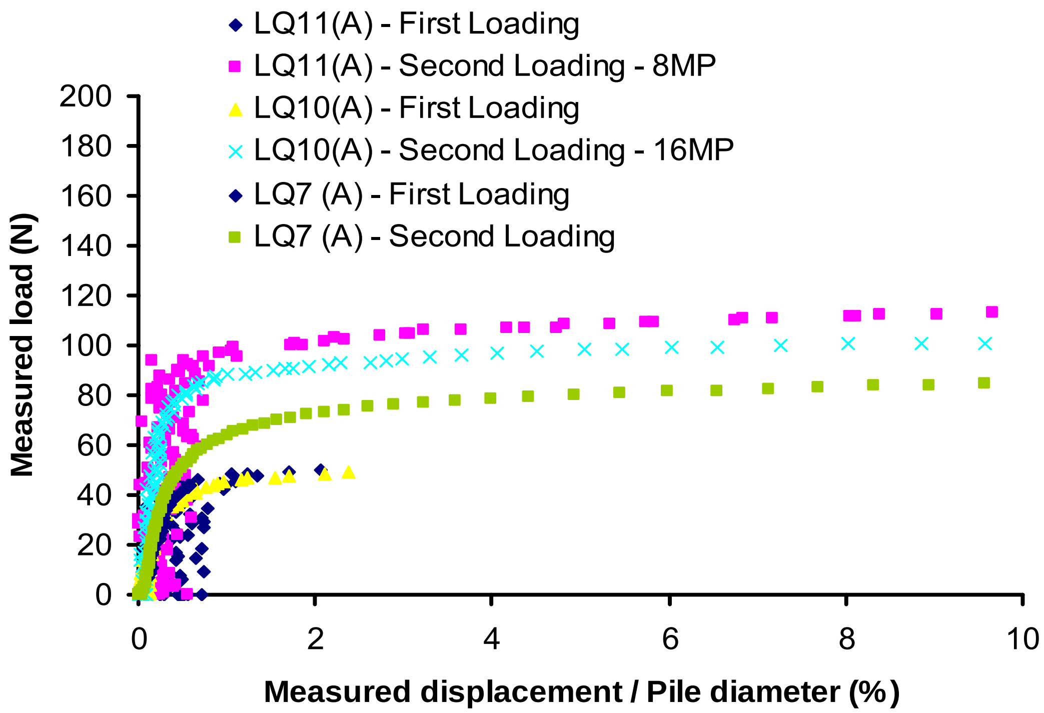
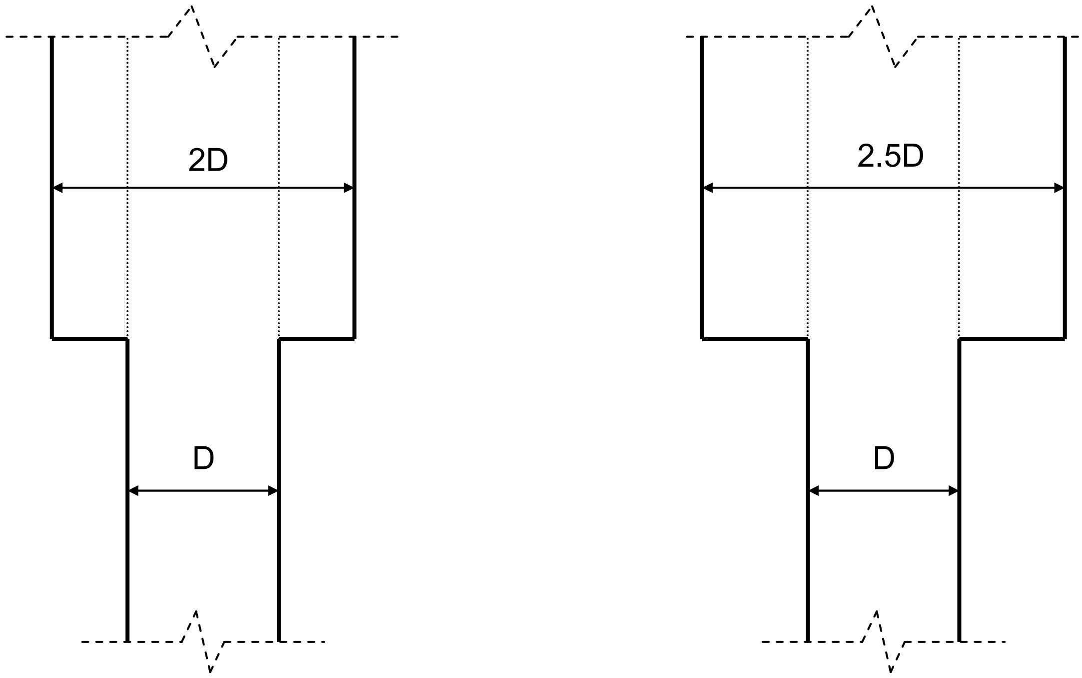


Figure 5.24 Tests LQ7(A), LQ10(A) and LQ11(A). 1st loading cycle – single pile foundations loaded up to working load. 2nd loading cycle – enhanced piled foundations loaded to failure: test LQ7(A) – single pile, test LQ10(A) enhanced pile foundation with sixteen 100mm long mini-piles at 3D spacing and test LQ11 enhanced pile foundation with eight 100mm long mini-piles at 3D spacing.



(a) (b)

Figure 5.25 Effective geometry of the enhanced centre pile with 100mm long mini-piles installed at 2D centre to centre spacing with the existing centre pile:
(a) Mini-pile group of 8 (Test LQ11(A))
(b) Mini-pile group of 16 (Test LQ10(A)).

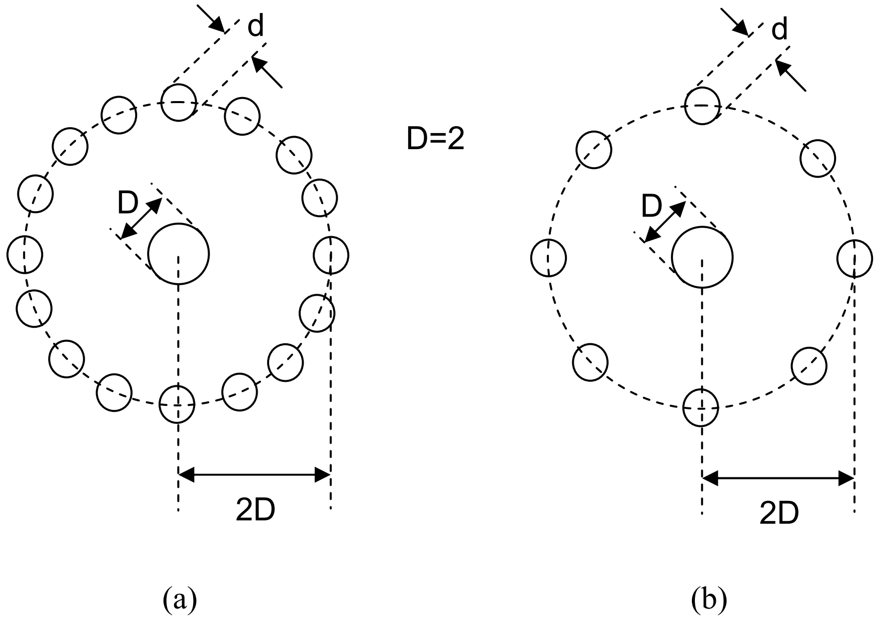


Figure 5.26 Details of the geometry of the model of novel pile groups:
(a) Mini-pile group of 8
(b) Mini-pile group of 16.

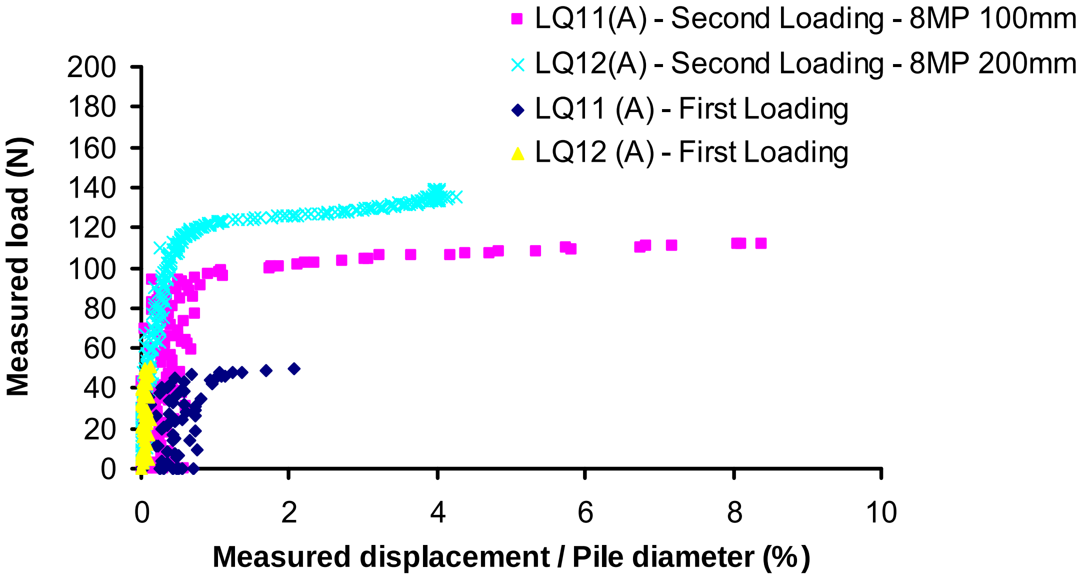


Figure 5.27 Tests LQ11(A) and LQ12(A). 1st loading cycle – single pile foundation loaded up to working load. 2nd loading cycle – enhanced pile foundation: Test LQ11(A) eight 100mm long mini-piles with 2D spacing; Test LQ12(A) eight 200mm long mini-piles with 2D spacing.

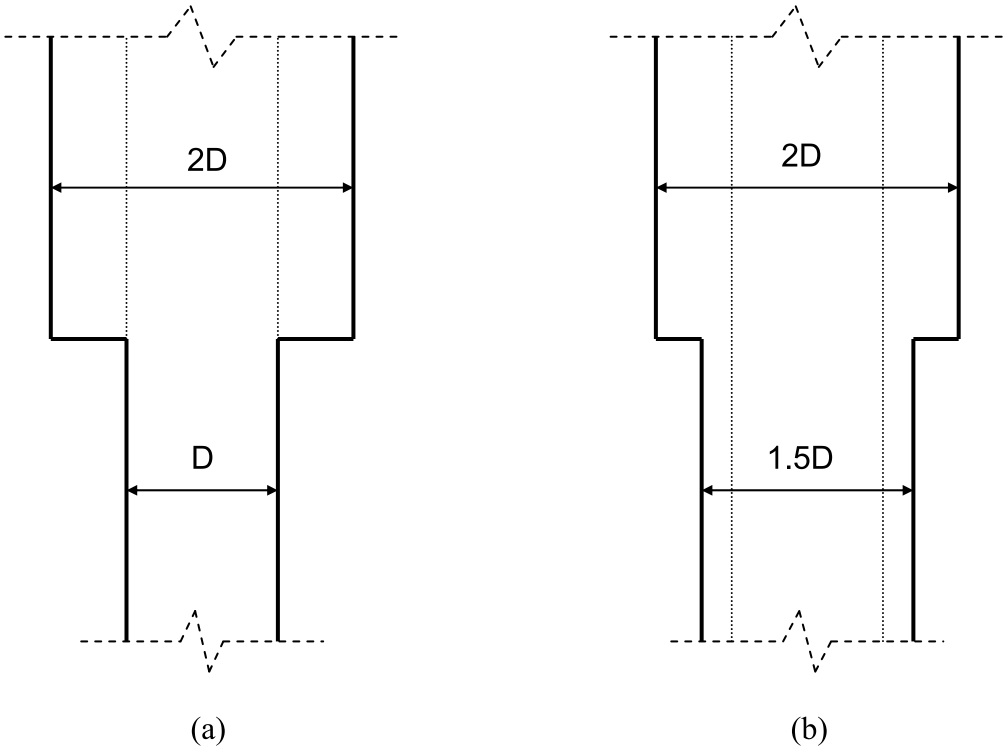


Figure 5.28 Tests LQ9(A) and LQ10(B). Enhanced pile foundations with eight mini-piles at 1.5D centre to centre spacing.
(a) 100mm long mini-piles (Test LQ9(A))
(b) 200mm long mini-piles (Test LQ10(B)).

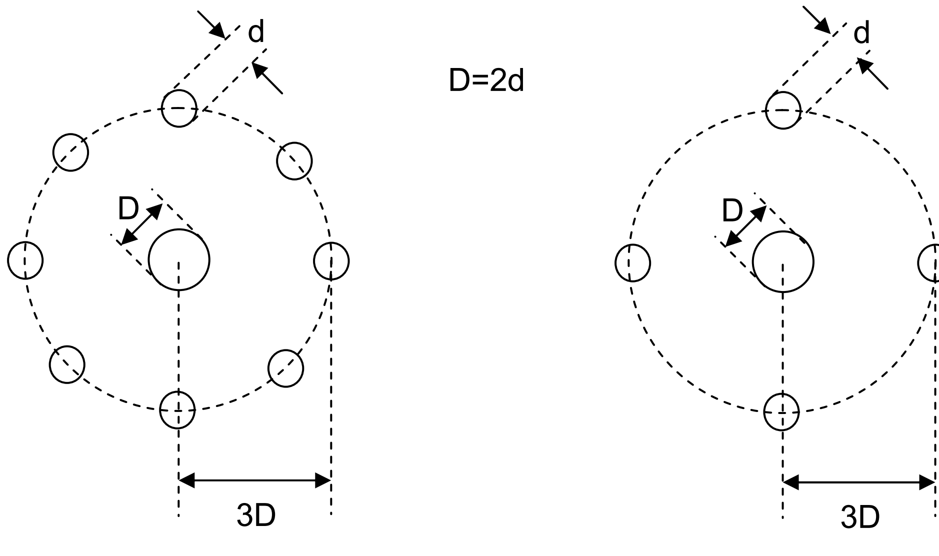


Figure 5.29 Examples of geometry of centrifuge model used to investigate caisson effect.

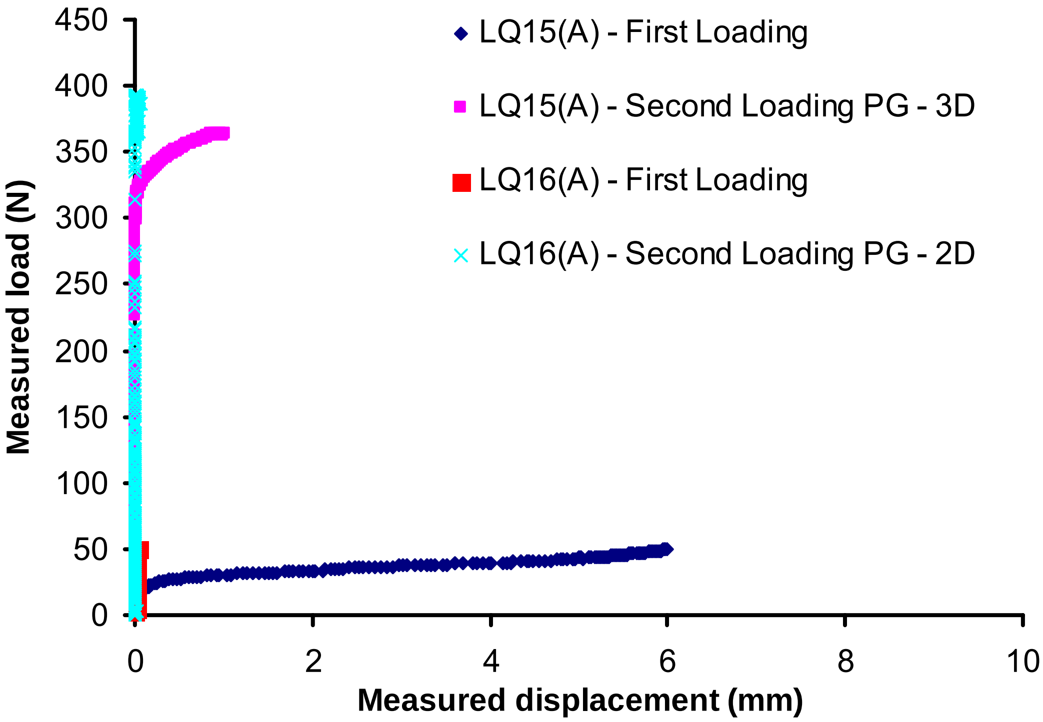


Figure 5.30 Tests LQ15(A) and LQ16(A). 1st loading cycle – single pile foundations loaded up to working load. 2nd loading cycle – novel pile group: Test LQ15(A) eight 200mm long mini-piles with 3D spacing; Test LQ16(A) eight 200mm long mini-piles with 2D spacing.

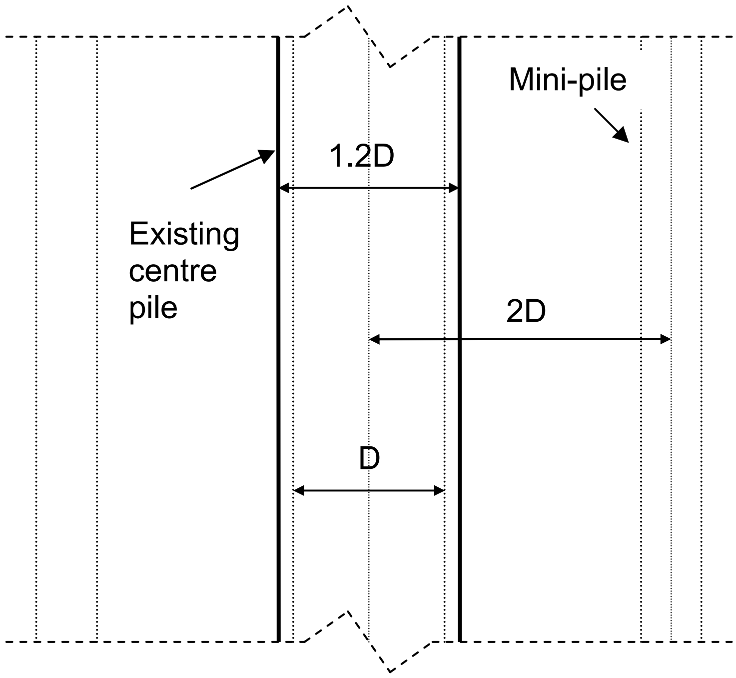
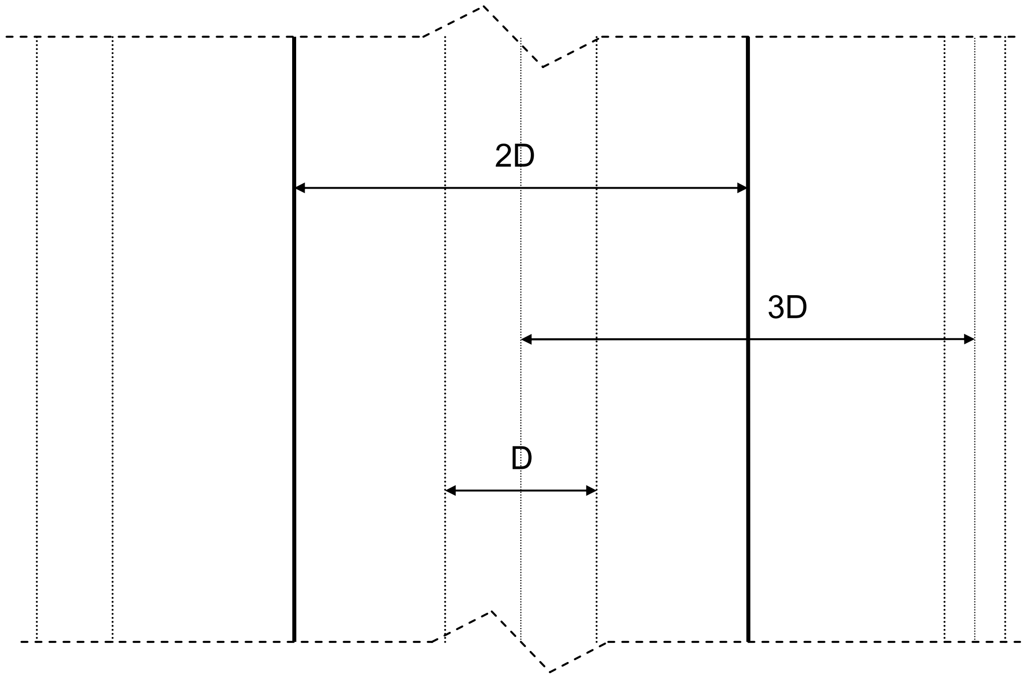
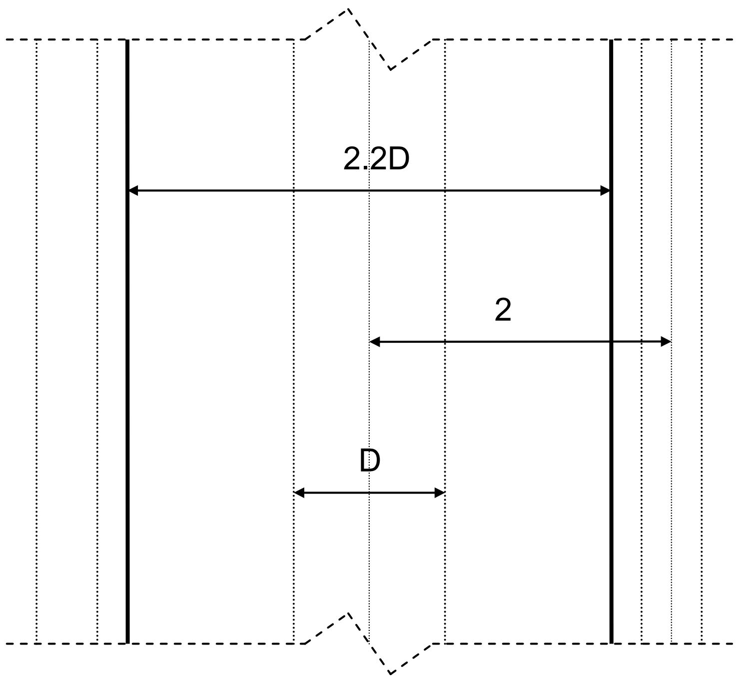


Figure 5.31 Test LQ17(A). Novel pile group with four 200mm mini-piles at 2D centre to centre spacing with the existing centre pile.



(a)



(b)

Figure 5.32 Tests LQ15(A) and LQ16(A). Novel pile group with eight 200mm mini-piles.
(a) Spacing between the mini-pile group and existing centre pile of 3D.
(b) Spacing between the mini-pile group and existing centre pile of 2D.

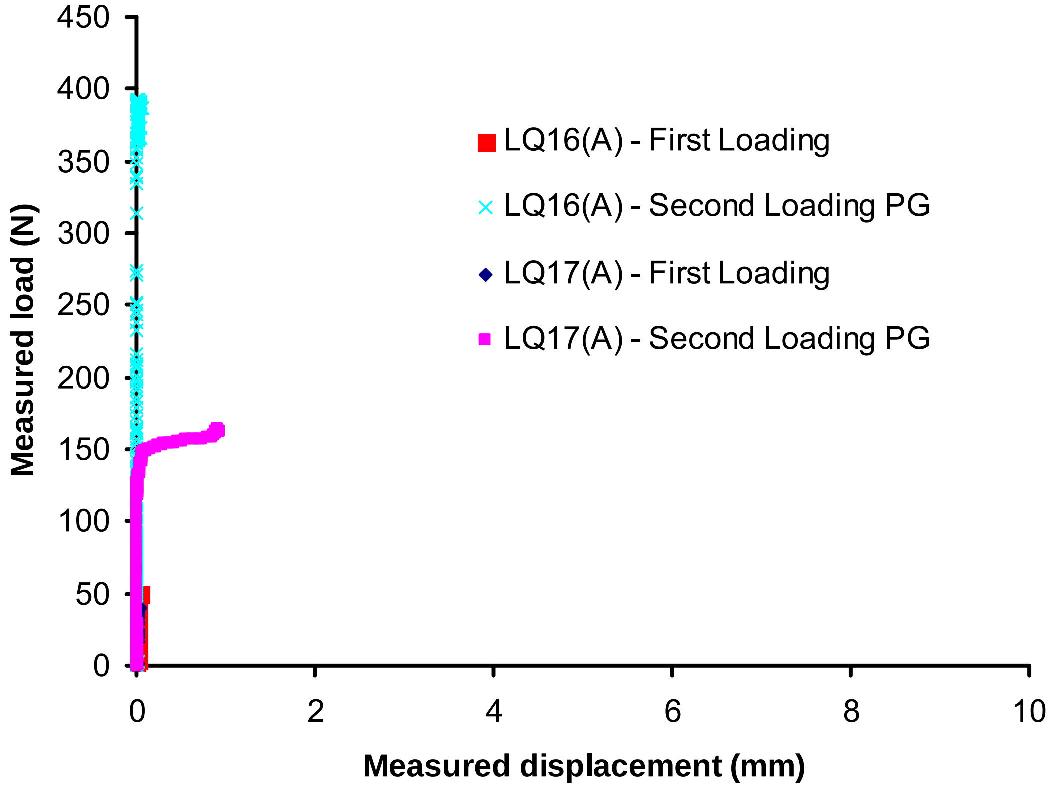


Figure 5.33 Tests LQ16(A) and LQ17(A). 1st loading cycle – single pile foundations. 2nd loading cycle – novel pile group: Test LQ16(A) eight 200mm long mini-piles with 2D spacing; Test LQ17(A) four 200mm long mini-piles with 2D spacing.

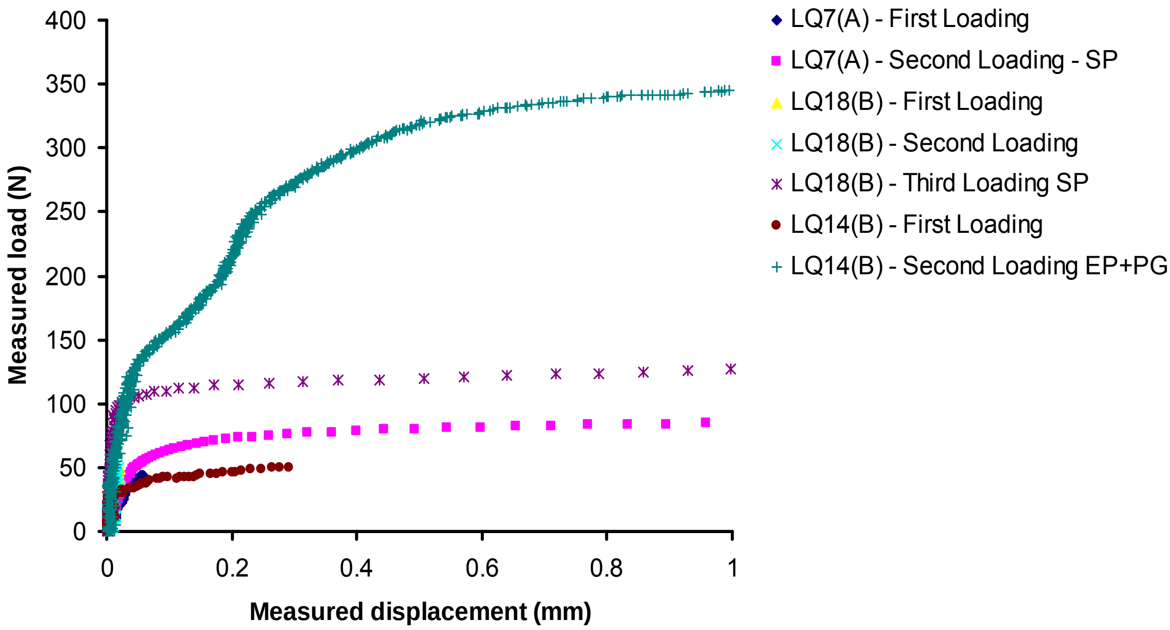


Figure 5.34 Tests LQ7(A), LQ14(B) and LQ18(B). 1st loading cycle – single pile foundation. 2nd loading cycle – Test LQ7(A) single pile foundation; Test LQ14(B) novel pile group (centre pile enhanced with eight 100mm long mini-piles with 3D spacing; Test LQ18(B) single pile foundation (centrifuge was not stopped between the first and second loading cycle). 3rd loading cycle – test LQ18(B) centre pile enhanced with eight 100mm long mini-piles with 3D spacing.

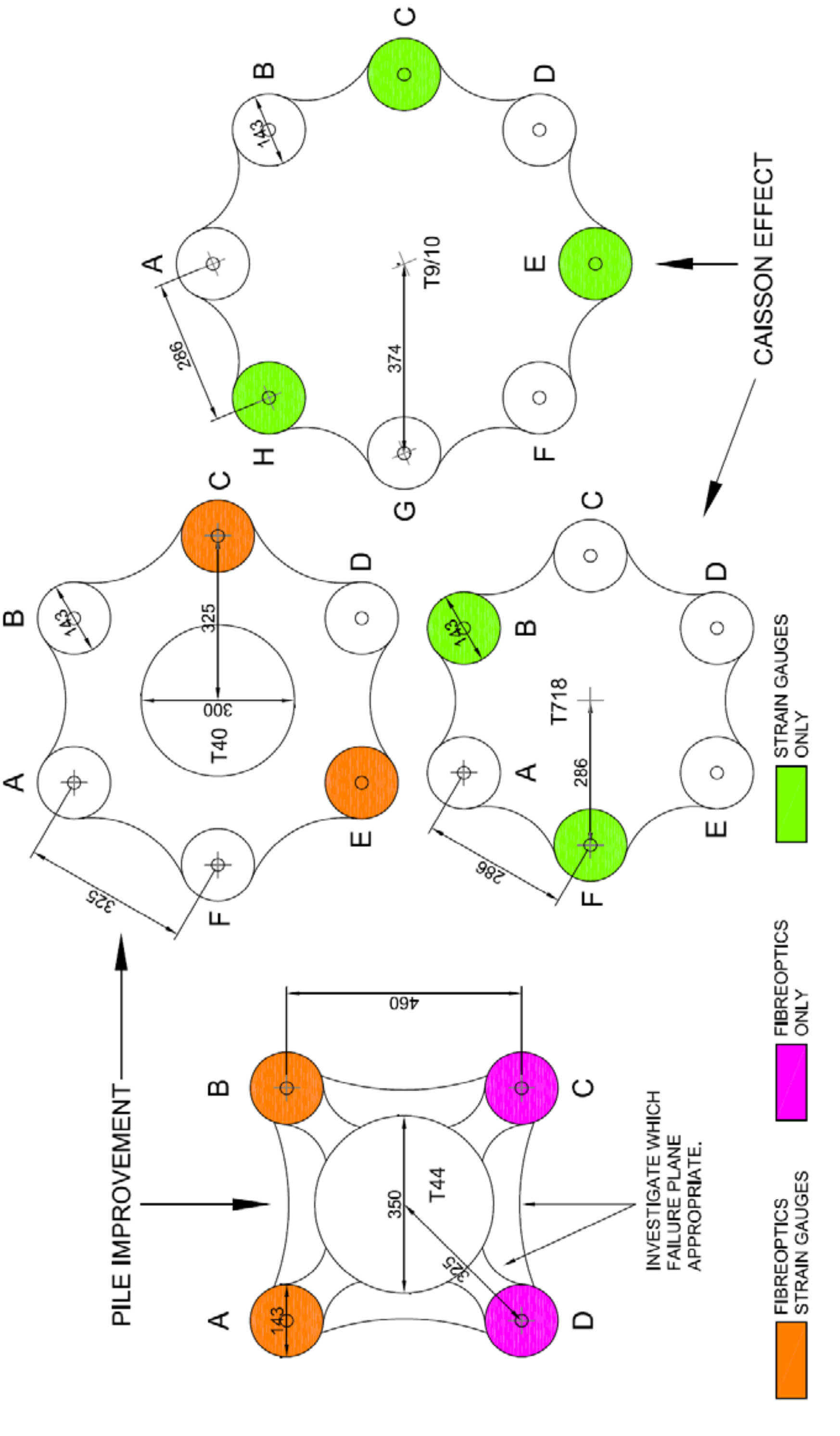


Figure 5.35 Pile improvement and caisson effect using mini-piles, trials at Chattenden, Kent, UK (Fernie et al., 2006).

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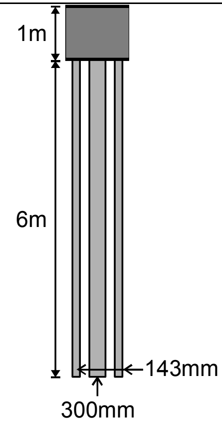
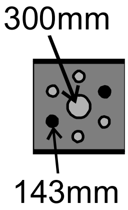
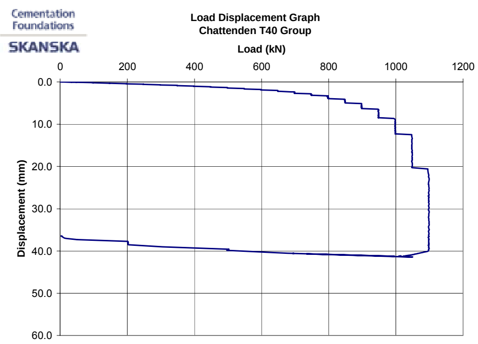
<tr><td>e</td><td>
</td><td>
</td><td>
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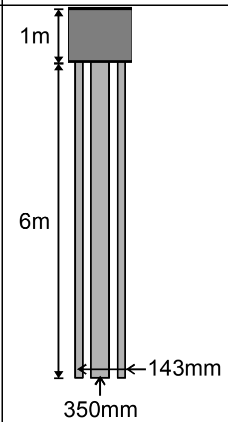
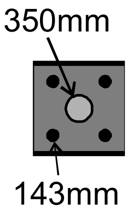
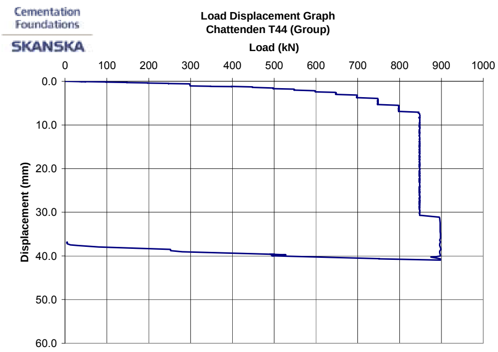
<tr><td>f</td><td>
</td><td>
</td><td>
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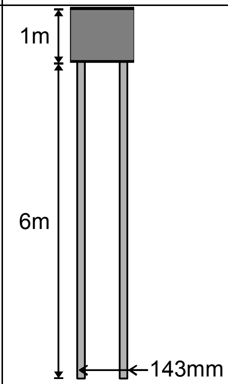
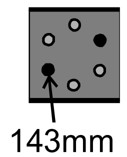
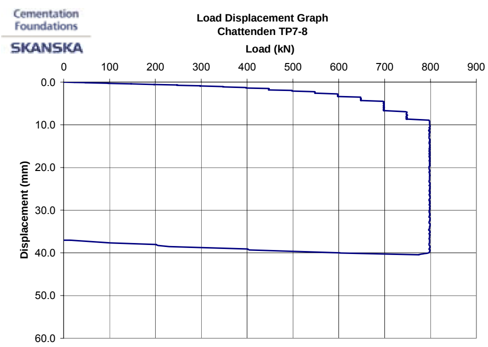
<tr><td>g</td><td>
</td><td>
</td><td>
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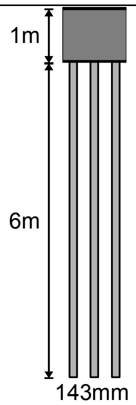
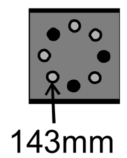
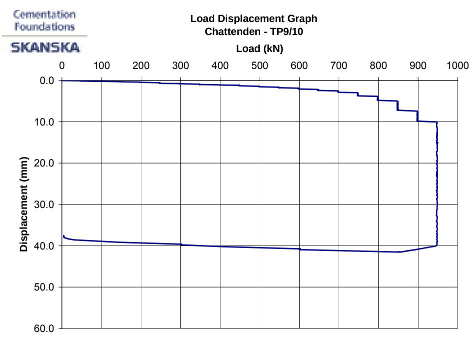
<tr><td>h</td><td>
</td><td>
</td><td>
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Figure 5.36 Geometry of test carried out and their performance during load testing, trials at Chattenden, Kent, UK (Fernie et al., 2006).

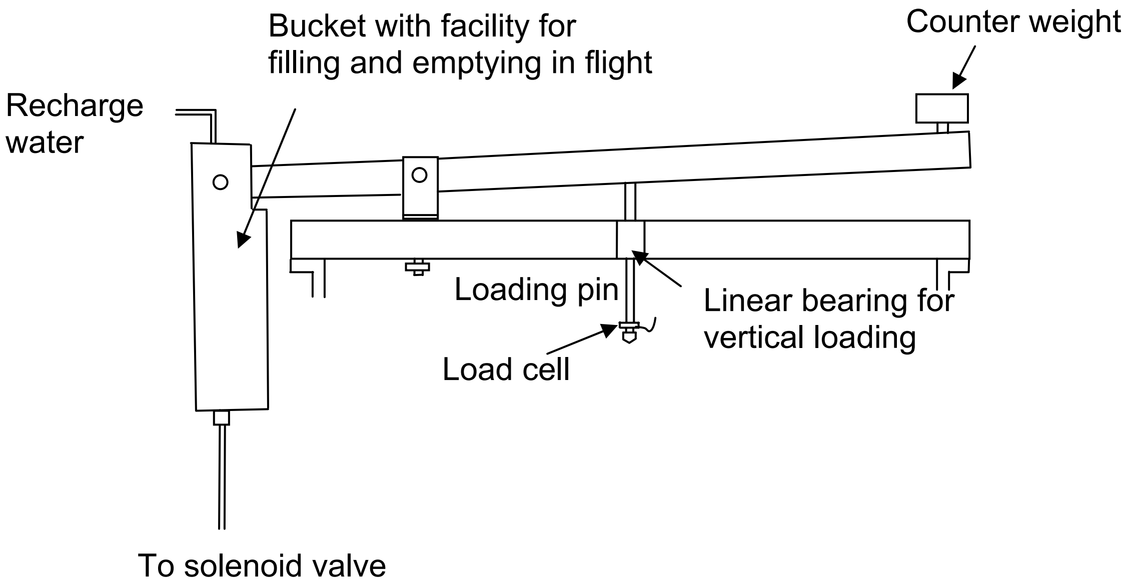

Figure 6.1 Axial loading pile foundation device used in centrifuge model testing (Morrison, 1994).

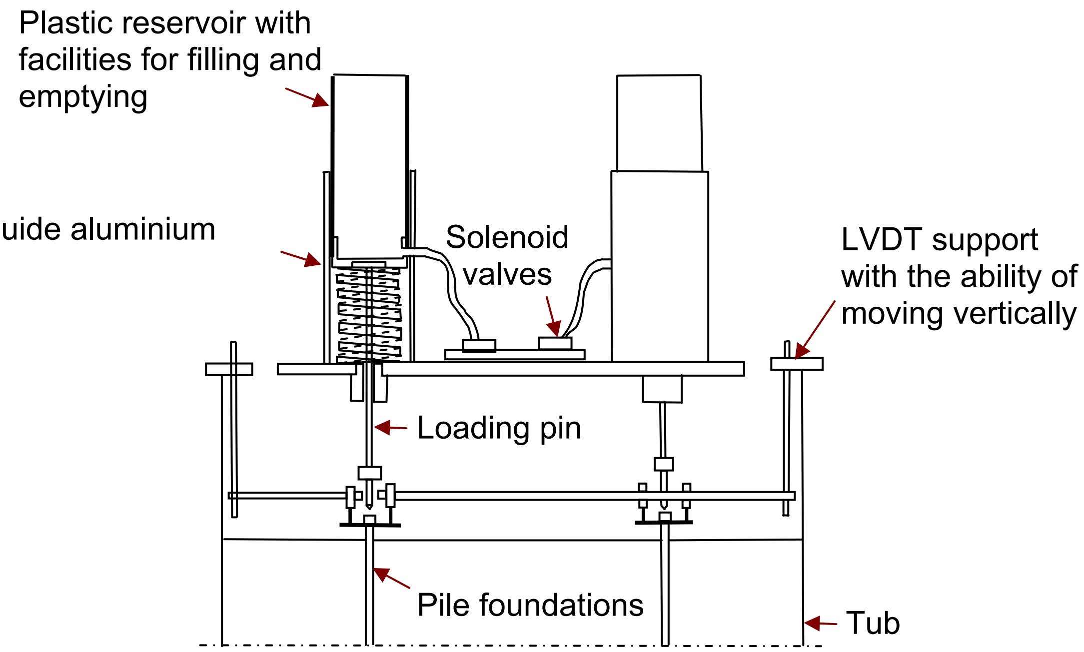

Figure 6.2 New loading apparatus developed for centrifuge model testing.

Figure 6.3 Installation of 10mm diameter model piles at 1g.

Figure 6.4 Model on centrifuge swing and ready for spin up.

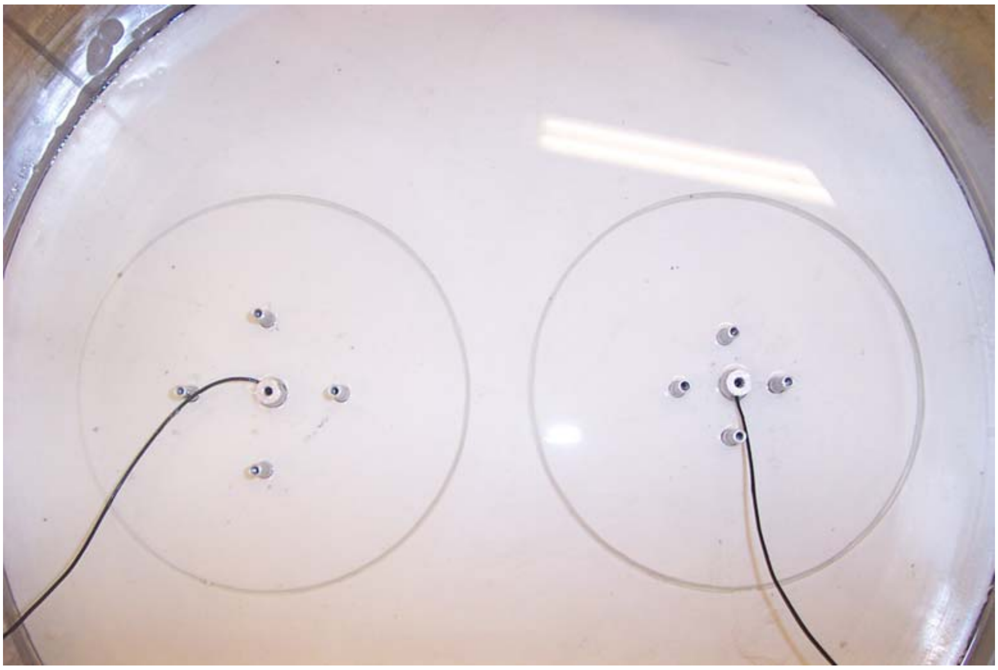

Figure 6.5 Enhanced existing model pile foundations, after initial loading, with mini-pile groups.

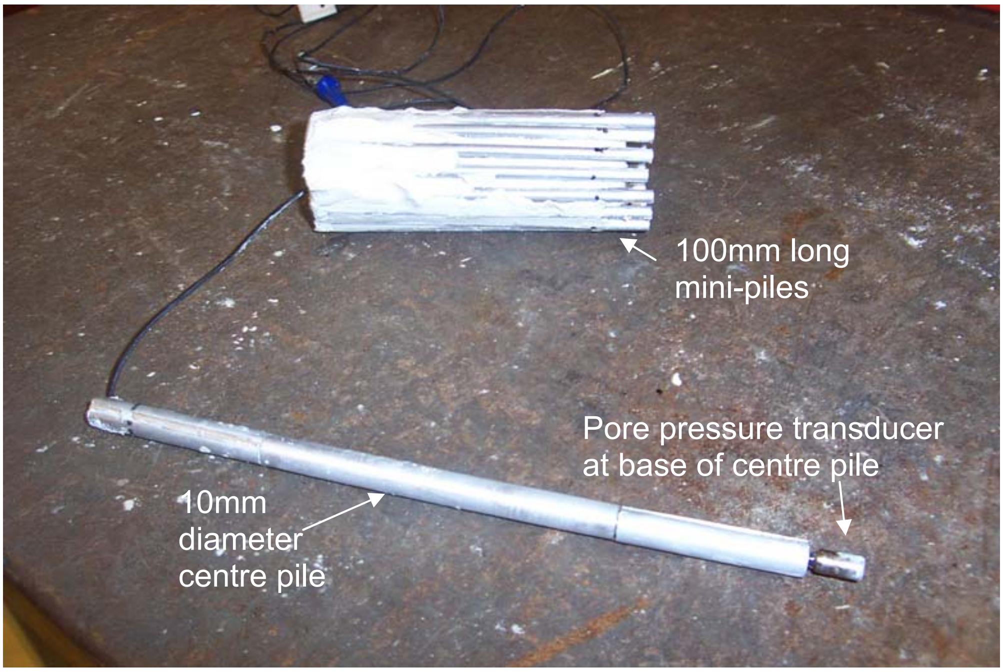

Figure 6.6 Pore pressure transducer at the tip of the model pile used in centrifuge model testing (Test LQ19).

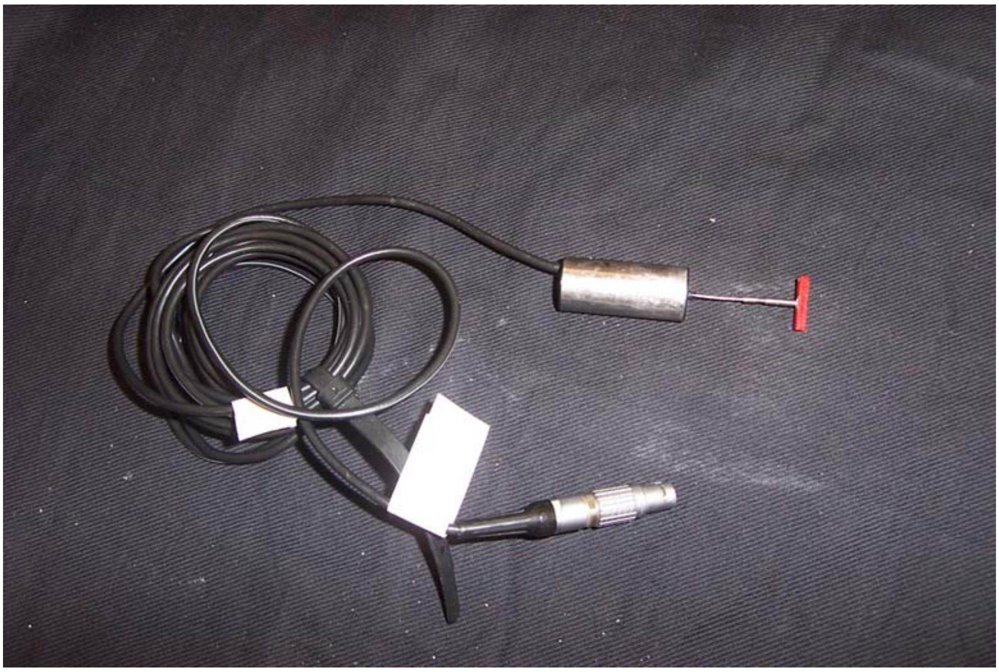

Figure 6.7 LVDT used to measure model pile displacement during centrifuge testing.

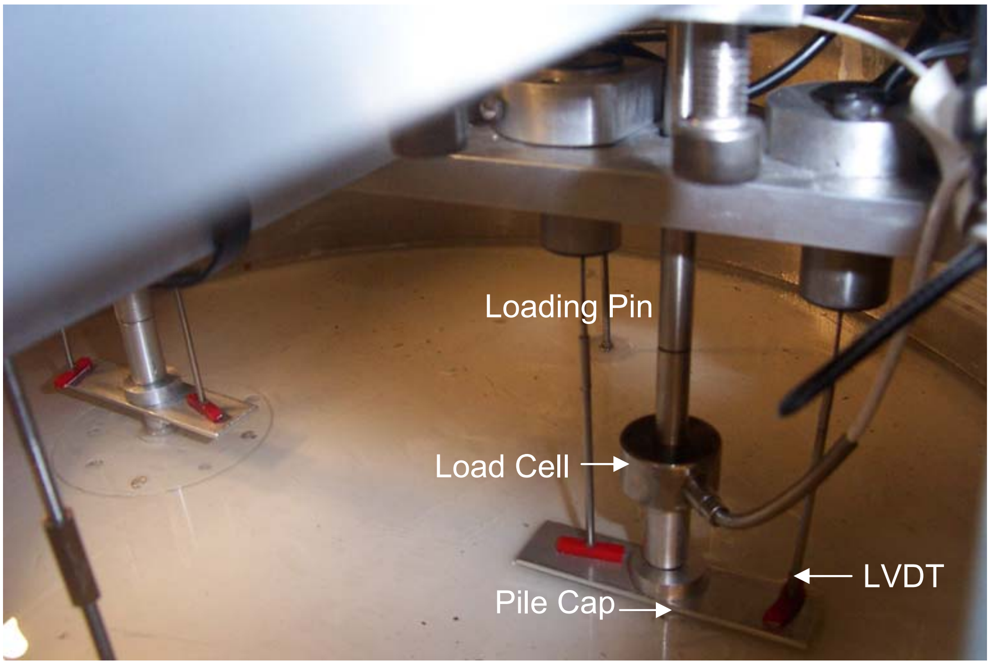

Figure 6.8 Load cell connected to a loading pin used in centrifuge model testing.

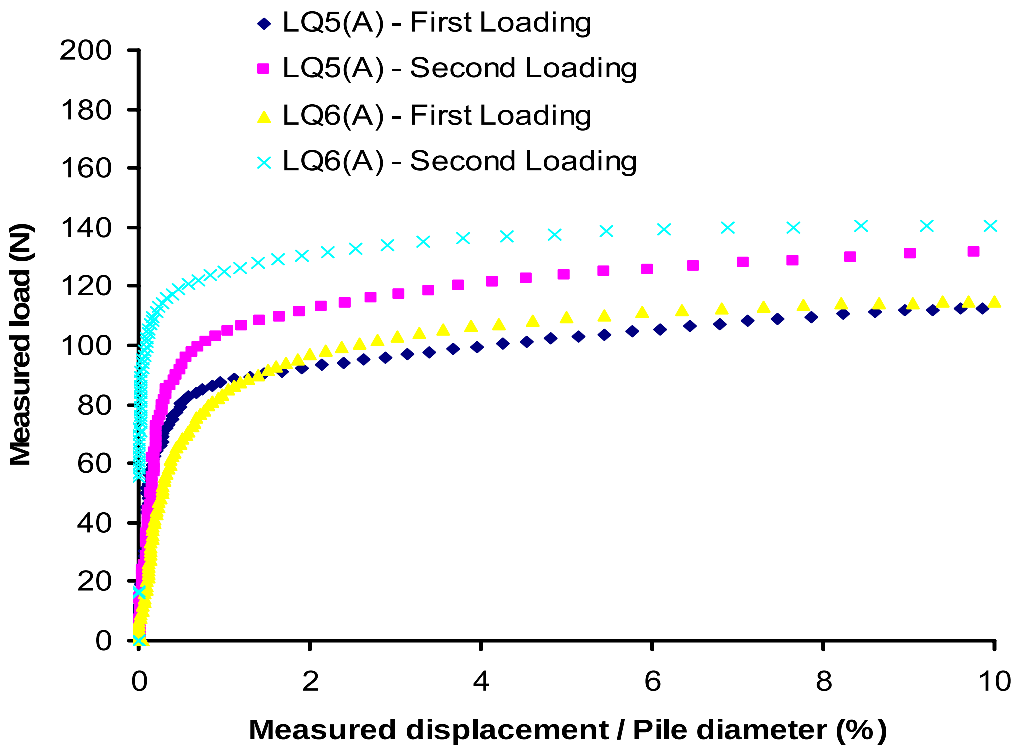

Figure 6.9 Test LQ5(A) and LQ(6) single pile foundations subjected to 1st and 2nd loading to failure. An increase in pile capacity of 20% was observed during 2nd loading.

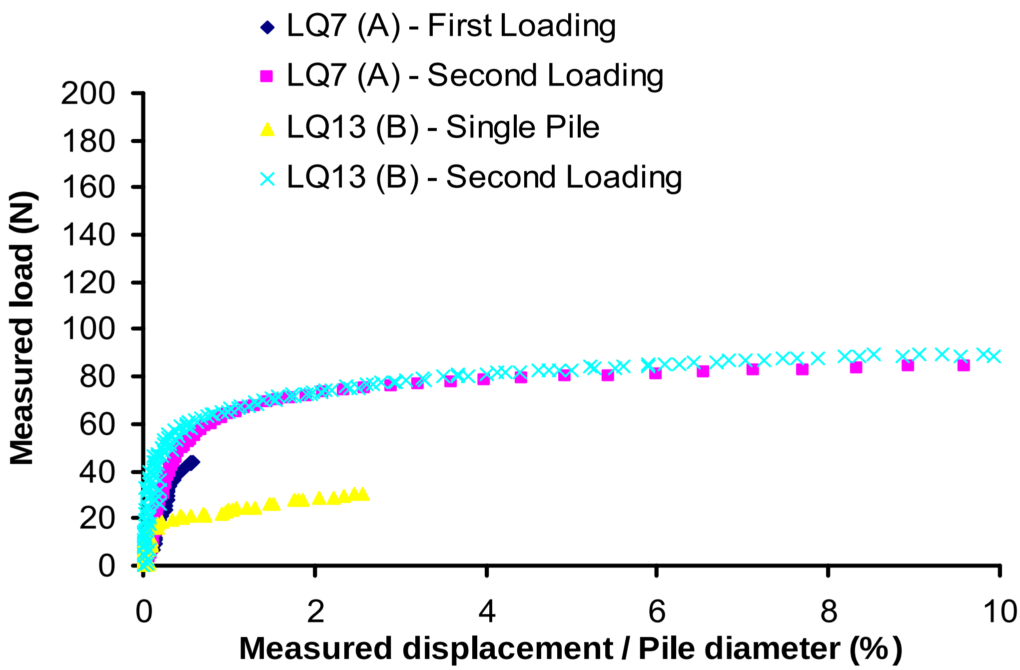

Figure 6.10 Tests LQ7(A) and LQ13(B) single pile foundations subjected to 1st and 2nd loading. The piles were subjected to working load during the 1st cycle. The piles were loaded to failure during the 2nd cycle.

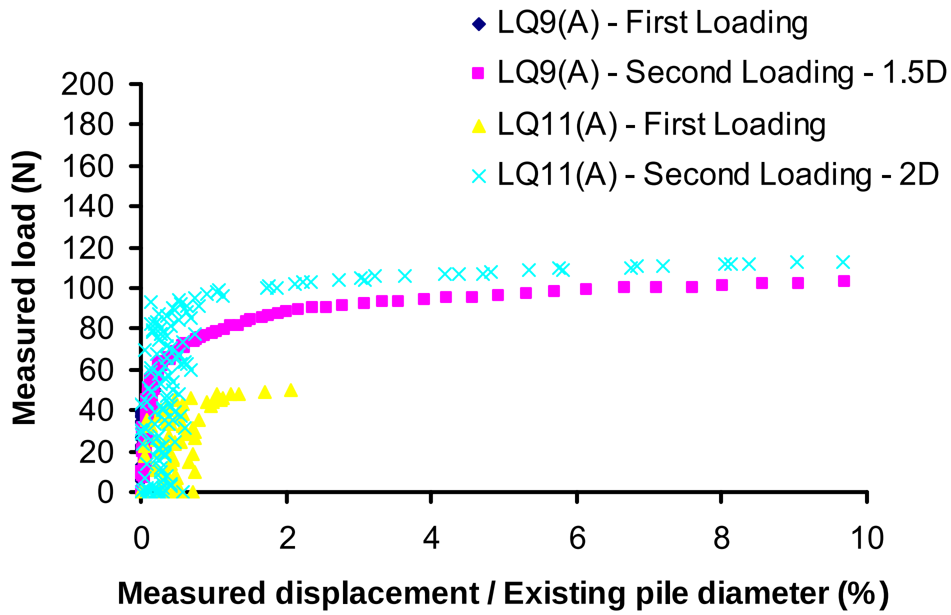

Figure 6.11 1st loading cycle - single pile foundation loaded up to working load (FOS=2.0), tests LQ9(A) and LQ11(A). 2nd loading cycle – enhanced pile foundations with 8 mini-piles 100mm long, test LQ9(A) with 1.5D spacing and test LQ11(A) with 2D spacing.

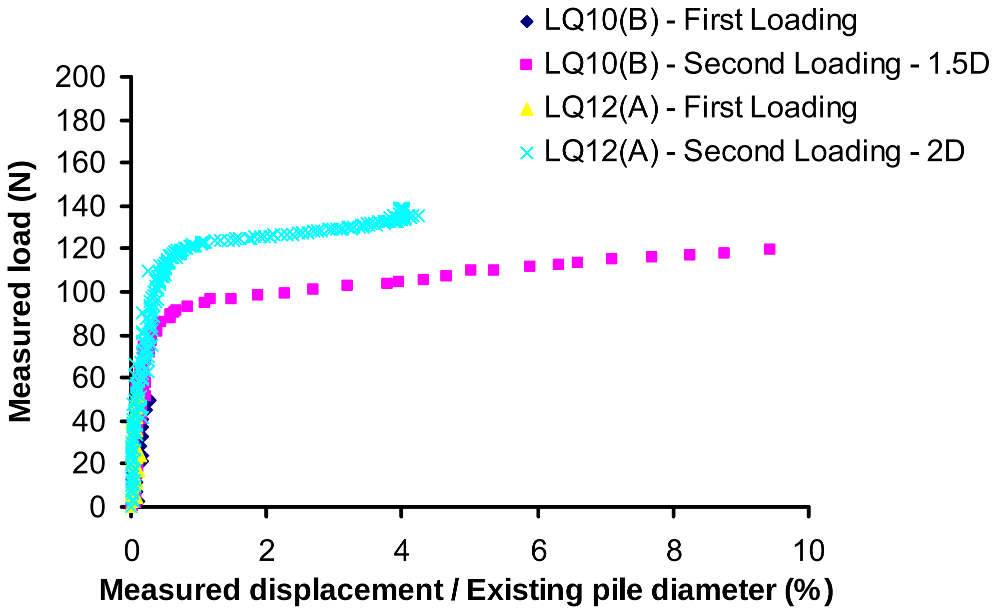

Figure 6.12 1st loading cycle - single pile foundation loaded up to working load (FOS=2.0), tests LQ10(B) and LQ12(A). 2nd loading cycle – enhanced pile foundations with 8 mini-piles 200mm long, test LQ10(B) with 1.5D spacing and test LQ12(A) with 2D spacing.

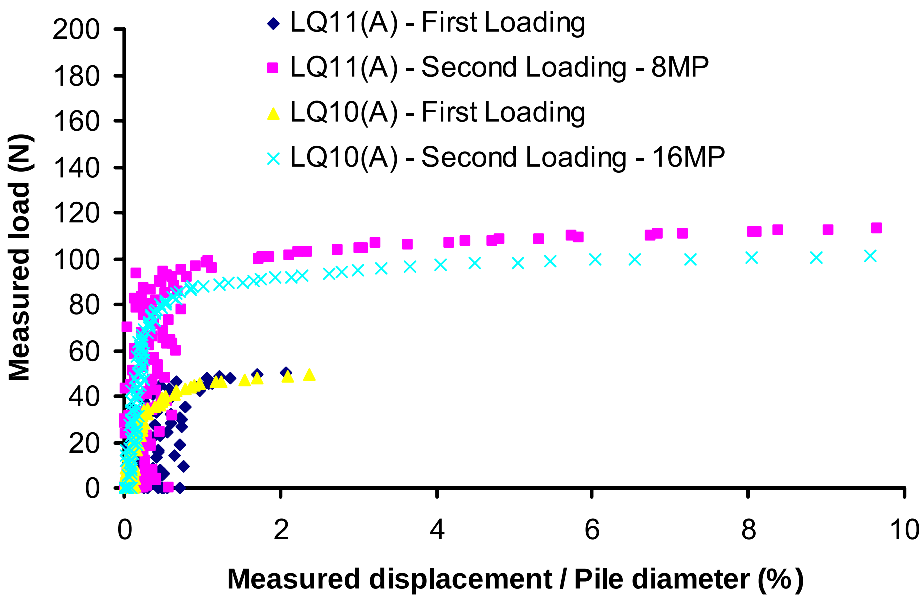

Figure 6.13 Tests LQ10(A) and LQ11(A). 1st loading cycle – single pile foundations loaded up to working load. 2nd loading cycle – enhanced piled foundations loaded to failure: test LQ10(A) enhanced pile foundation with sixteen 100mm long mini-piles at 3D spacing and test LQ11 enhanced pile foundation with eight 100mm long mini-piles at 3D spacing.

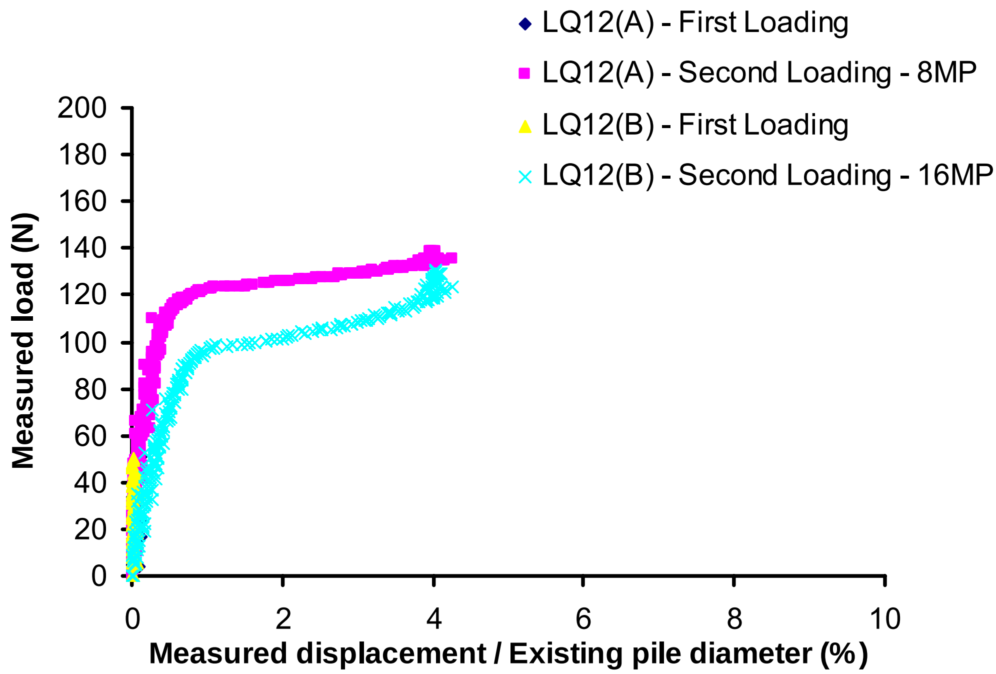

Figure 6.14 Tests LQ12(A) and LQ12(B). 1st loading cycle – single pile foundation loaded up to working load. 2nd loading cycle – enhanced pile foundation: Test LQ12(A) eight 200mm long mini-piles with 2D spacing; Test LQ12(B) sixteen 200mm long mini-piles with 2D spacing.

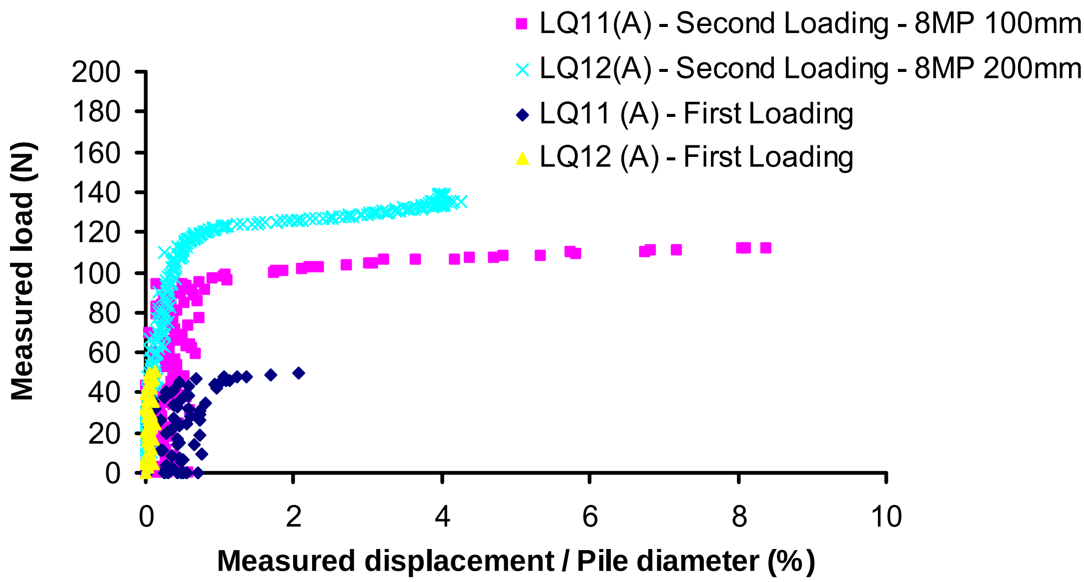

Figure 6.15 Tests LQ11(A) and LQ12(A). 1st loading cycle – single pile foundation loaded up to working load. 2nd loading cycle – enhanced pile foundation: Test LQ11(A) eight 100mm long mini-piles with 2D spacing; Test LQ12(A) eight 200mm long mini-piles with 2D spacing.

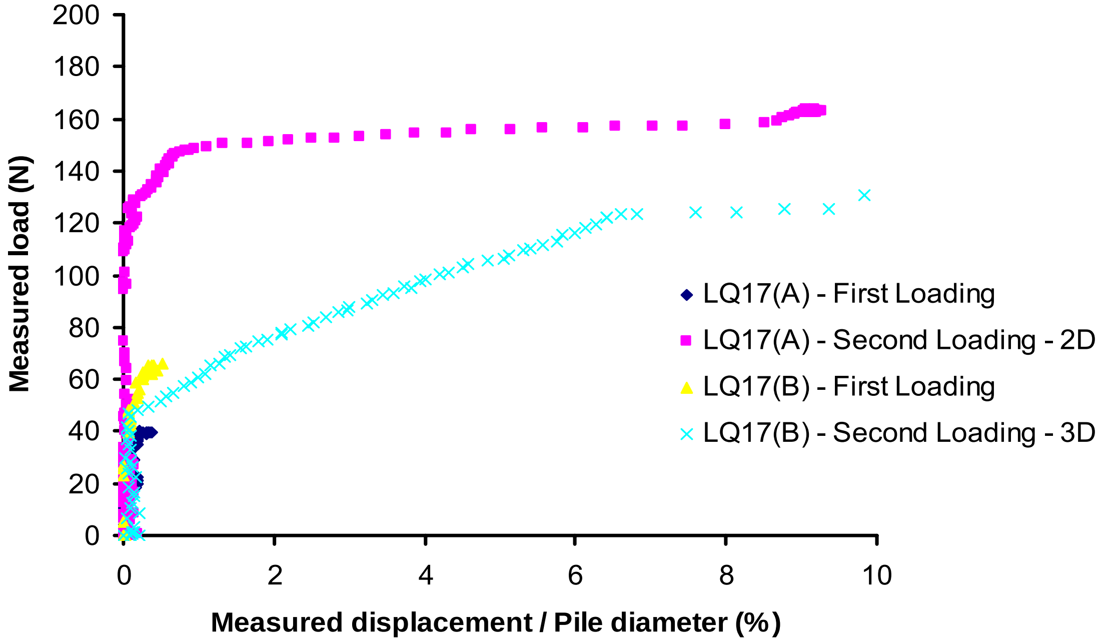

Figure 6.16 Tests LQ17(A) and LQ17(B). 1st loading cycle – single pile foundations loaded up to working load. 2nd loading cycle – novel pile group: Test LQ16(A) eight 200mm long mini-piles with 3D spacing; Test LQ17(B) four 200mm long mini-piles with 3D spacing.

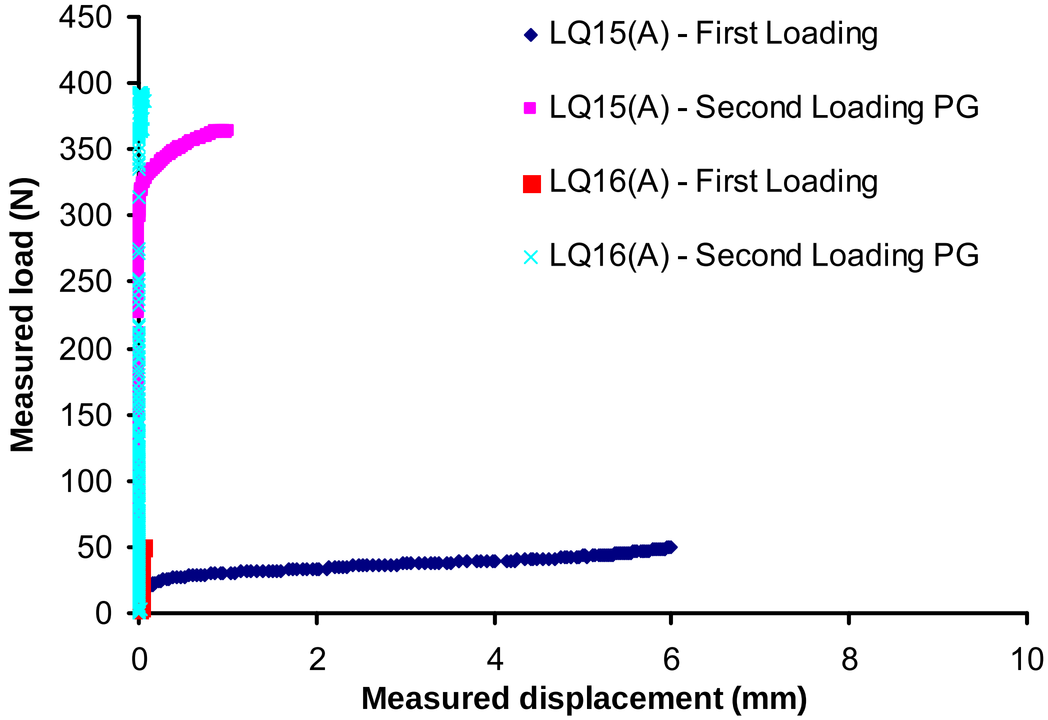

Figure 6.17 Tests LQ15(A) and LQ16(A). 1st loading cycle – single pile foundations loaded up to working load. 2nd loading cycle – novel pile group: Test LQ15(A) eight 200mm long mini-piles with 3D spacing; Test LQ16(A) four 200mm long mini-piles with 3D spacing.

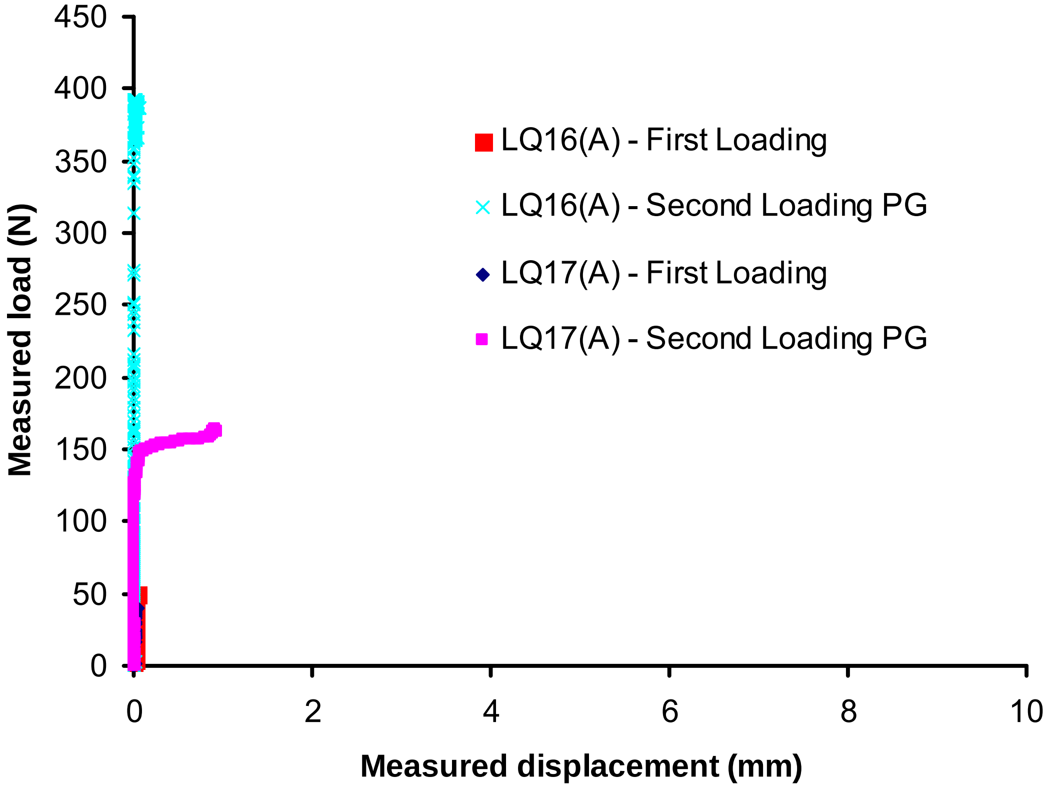

Figure 6.18 Tests LQ16(A) and LQ17(A). 1st loading cycle – single pile foundations. 2nd loading cycle – novel pile group: Test LQ16(A) eight 200mm long mini-piles with 3D spacing; Test LQ17(A) four 200mm long mini-piles with 3D spacing.

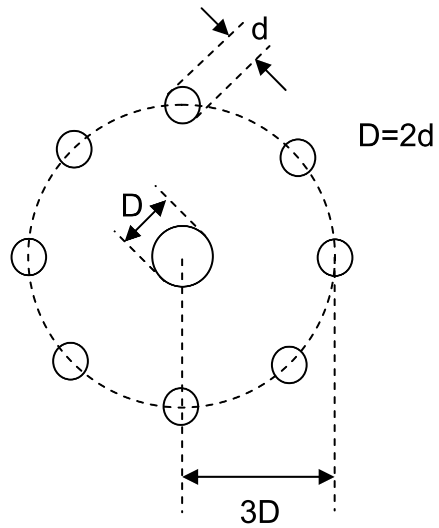

Figure 6.19 10mm diameter pile foundation enhanced with eight mini-piles of 100mm long with 2D spacing, where D is the diameter of the centre existing pile.

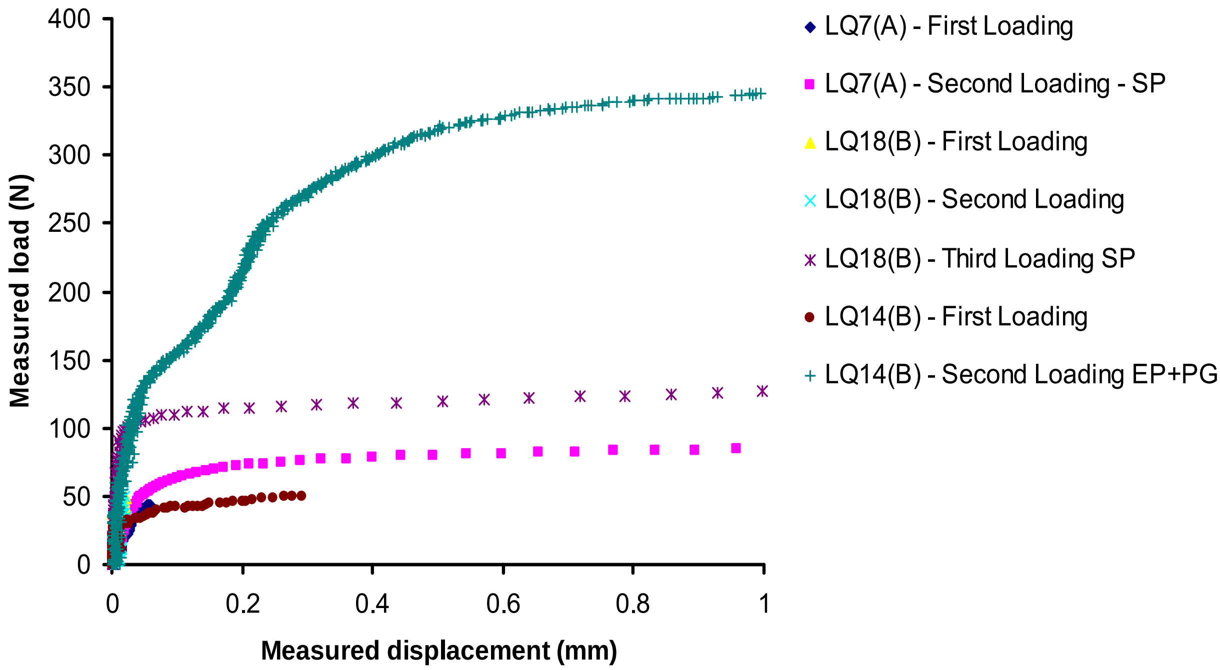

Figure 6.20 Tests LQ7(A), LQ14(B) and LQ18(B). 1st loading cycle – single pile foundation. 2nd loading cycle – Test LQ7(A) single pile foundation; Test LQ14(B) novel pile group (centre pile enhanced with eight 100mm long mini-piles with 3D spacing; Test LQ18(B) single pile foundation (centrifuge was not stopped between the first and second loading cycle). 3rd loading cycle – test LQ18(B) centre pile enhanced with eight 100mm long mini-piles with 3D spacing.

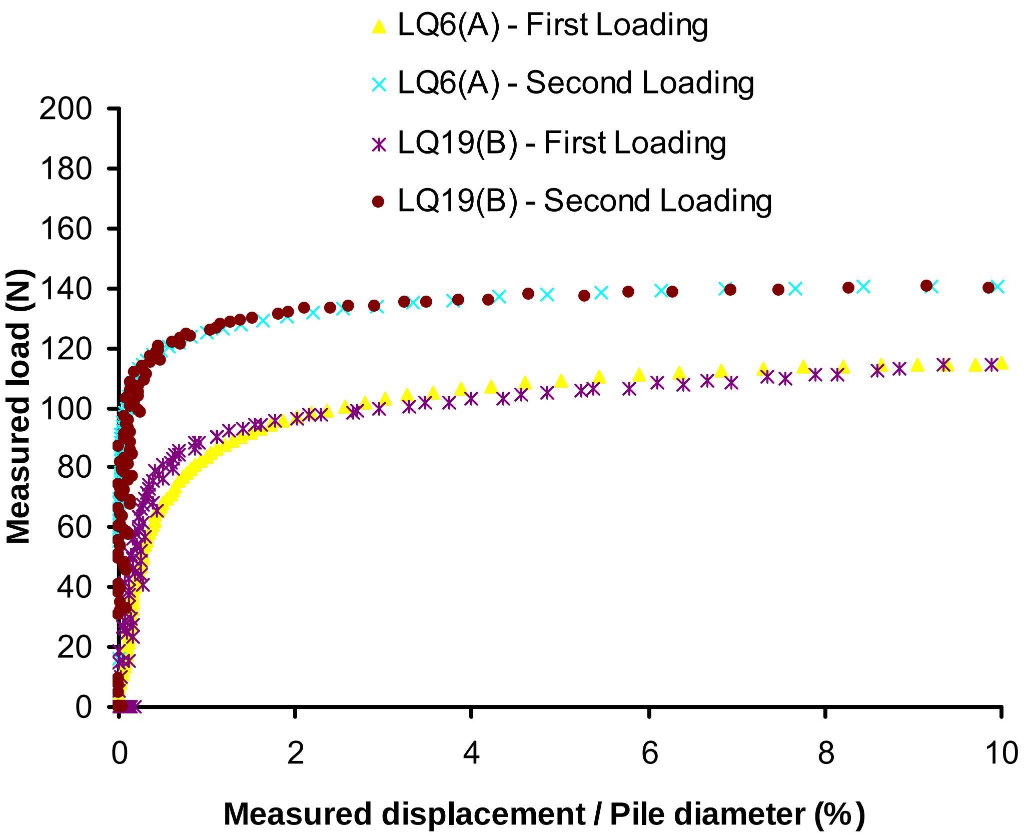

Figure 6.21 Performance of single pile foundations during 1st and 2nd loading. Test LQ6(A) – centrifuge was stopped between the first and second loading. Test LQ19(B) – centrifuge was NOT stopped between the first and second loading.

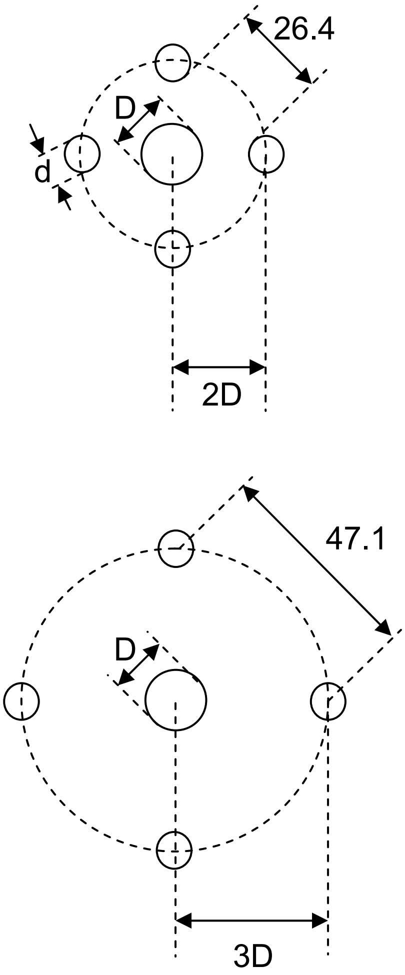

Figure 6.22 The effects on centre-to-centre spacing of the mini-piles as a result of diameter change of the mini-piles and the distance from the centre pile.