

Development of Shaft Friction on Driven Piles in Sand and Clay

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1. Introduction

Despite widespread research effort in recent years, design methods to estimate the shaft resistance of displacement piles continue to rely heavily upon empirical correlations. In an attempt to achieve a more reliable design approach several researchers have assembled databases of instrumented pile load tests and used these to propose new design methods e.g. American Petroleum Institute (API-2000) for piles in sand and Lehane et al. (2000) for piles in clay. The most significant contribution to our understanding of displacement pile behaviour has resulted from field tests using highly instrumented piles, most notably those conducted with the Imperial College (IC) instrumented test pile (Bond 1989, Lehane 1992 and Chow 1997). These findings have been synthesised in the MTD design approach (Jardine et al. 2005), which has been shown to be more reliable than traditional design approaches at estimating the shaft resistance of driven piles. However, Randolph (2003) highlights the continuing

presence of empirical factors in many new design approaches and demonstrates the need to understand the physical mechanisms governing pile capacity.

2. Shaft Capacity of Piles in Sand

Existing design approaches for piles in sand can be broadly classified as follows:

- (i) Methods relating the shaft resistance to the in-situ horizontal stress (earth pressure approach) or,
- (ii) Methods which correlate the shaft resistance to an in-situ test parameter such as SPT N value or CPT q_c .

One of the most widely used design earth pressure approaches API RP2A (2000) relates the local ultimate shaft friction (τ_f) developed on a driven pile in sand to the vertical effective stress (σ'_v) through the equation,

$$\tau_f = K_f \sigma'_v \tan \delta \quad (1)$$

The most difficult parameter to determine is K_f (the coefficient of earth pressure). Field tests indicate that K_f depends on the soil relative density and stress level and that K is not constant along the pile length. In the API method K_f is taken to be unity for closed-ended driven piles, and δ , the interface friction angle between the soil and pile wall is assumed to depend on the material's relative density and vary from 15° for very loose sand to 35° for very dense sand. Although equation 1 indicates that (τ_f) increases indefinitely and linearly with the overburden pressure, the API recommendations specify a limiting maximum (τ_f) value that is dependent on the assessed relative density of the sand. Shaft resistance is often correlated to Standard Penetration Test N values for a range of soil types. The relationships for sand take the form:

$$\tau_f \text{ (kPa)} = \lambda N \quad (2)$$

Many workers have suggested λ values for use in design. Recommended λ values typically range from 2.0-3.0 for displacement piles in sand.

Randolph (2003) notes that "one of the major advances in design approaches for the driven piles in sand during the last decade has been the linking of the key parameters such as base resistance and maximum shaft friction to the cone resistance (q_c). Bustamante & Gianesselli (1982) propose a method, which relates the local ultimate skin friction, τ_f , with the CPT end resistance (q_c). τ_f is calculated by dividing the cone end resistance (q_c) by a coefficient α_s :

$$\tau_f = q_c / \alpha_s \quad (3)$$

Various values of α_s are proposed depending on the nature of the soil and the pile installation methods. Values range from 60 to 150 for concrete piles and from 120 to 200 for steel piles, limiting maximum τ_f stresses are also recommended and range from 35 to 150 kPa.

Recent high quality instrumented pile tests (Lehane 1992, Chow 1997) have shown that τ_f mobilized by a pile is given by:

$$\tau_f = \sigma'_{rf} \tan \delta_f \quad (4)$$

where σ'_{rf} is the horizontal stress at failure and δ_f is the interface friction angle measured in appropriate direct shear or preferably ring shear tests. σ'_{rf} is composed of two components, the stationary horizontal stress (σ'_{rs}) which depends primarily on the in-

situ sand state and installation effects, and a dilational component ($\Delta\sigma'_{rd}$). Lehane and Jardine (1994) show that whilst the latter (dilational) component can dominate the shaft resistance response of model piles, it is much less important for field scale piles with $D > 300$ mm. Therefore in terms of field scale piles, accurate prediction of σ'_{rs} is paramount. σ'_{rs} measurements recorded during the installation of the Imperial College Pile (ICP) in medium dense, dune sand at Labenne are shown in Figure 1. Lehane (1992) noted the similarity between the shape of the q_c and σ'_{rs} profiles at the site, with σ'_{rs} being approximately two orders of magnitude lower than q_c at a given depth. A key observation was that σ'_{rs} at a given depth in the ground, reduced as the distance (h) between that point and the pile tip increased. Chow (1997) reported installation data for the same pile jacked into dense over-consolidated sand at Dunkirk. Although σ'_{rs} profiles were 4-5 times higher than those measured at Labenne, Chow (1997) found that plotting the ratio of σ'_{rs}/q_b for the data at both test sites against h/R , in figure 2, brought the whole data set into a relatively narrow band. Statistical analysis of 306 σ'_{rs} measurements at both sites indicated that the ratio of σ'_{rs}/q_b was not constant for a given h/R , but increased moderately with stress

level. On the basis of the foregoing Jardine et al. (2005) proposed the following equation to predict σ'_{rs} based on readily obtainable parameters, the q_c profile, pile geometry and in-situ vertical effective stress:

$$\sigma'_{rs} = 0.029 q_c (\sigma'_{v}/P_{atm})^{0.12} (h/R)^{-0.38} \quad (5)$$

where $P_{atm} = 100$ kPa

3. Interface dilation

By providing simultaneous measurements of radial effective stress with pressure sensors and local shaft friction, effective stress paths during pile loading can be plotted for the ICP. One such stress path measured during a compression load test is shown in Figure 3. The figure shows at the start of the load test the pile is in equilibrium with stationary radial effective stress is ≈ 60 kPa, and shear stress of -8 kPa as a result of the presence of residual load at the pile base. As the test progresses, the shear stress rises, this is accompanied by an initial drop in σ'_{r} , followed by a large increase, with failure occurring at $\sigma'_{r} \approx 84$ kPa, representing an approximate 45% increase in radial effective stress due to dilation. Lehane and Jardine (1994) collated a database of similar effective stress measurements on model and full scale piles and show that the increase in stress due to interface dilation is dependent on the shear stiffness of the soil (G), the pile radius (R) and radial

displacement of the soil grains during shearing (δ_r). Cavity expansion theory has been used to describe this behaviour:

$$\Delta\sigma'_{rd} = 2G\delta_r/R \quad (6)$$

Equation 6 suggests that dilation is highest on piles installed in dense sand (high shear modulus), with rough surface texture (where δ_r will be greatest) and small diameters.

4. h/R (friction fatigue) effect

The physical mechanism which causes the reduction in radial effective stress with increasing h/R evident in the IC pile tests in sand was explained by performing Constant Normal Stiffness (CNS) direct shear tests in the laboratory (DeJong et al. 2003). In traditional direct shear tests, the normal stress is constant throughout shearing (constant normal stress test). In CNS testing, the stiffness is maintained constant throughout the test in order to model the restraint provided by the stiff zone of soil outside the shear zone. Experiments using this apparatus have shown that shearing cause's densification of the soil in the shear zone and reduction in normal stress (equivalent to radial stress acting on the pile shaft).

To investigate this effect on piles White and Lehane (2004) performed

a number of instrumented pile installations using a centrifuge. The 9mm diameter (D) piles were installed to a depth (L) of 120mm. In order to assess the effect of installation cycling, three different installation techniques were adopted, (i) monotonic installation, in which the pile was pushed continuously to half its final depth, where a load test was performed before continuous installation to its final depth. (ii) Jacked installation, in which the pile was pushed downwards 2mm (one jack stroke), following which the pile was unloaded completely. (iii) Pseudo-dynamic installation, in which the pile was first pushed 2mm, before being withdrawn 1.5mm to simulate the elastic rebound experienced during pile driving (i.e. net penetration after each jacking stroke was 0.5mm). A comparison of stationary horizontal stresses measured at three locations along the shaft during installation is shown in Figure 4. It is clear that at any given level on the pile σ'_{hs} values are highest for the monotonically installed pile, intermediate for the jacked pile and lowest for the pseudo-dynamically installed piles.

A key finding of this study was that plotting normalised stationary horizontal stress (σ'_{hs}/q_c) against distance from the pile tip h, in Figure 5, did not result in a unique distribution of σ'_{hs}/q_c , rather the ratio at a given height (h) depended on the

number of loading cycles experienced. Using experimental data from their jacked and pseudo-dynamically installed piles shown in Figure 5, the authors argue that the number of load cycles experienced by a soil element, rather than the absolute distance of that element from the pile tip controls the σ'_{hs} value. Despite these findings recent reviews of design methods (Jardine et al. 2005 and Lehane et al. 2005) have found that design approaches incorporating h/R rather than number of load cycles (N) provides better estimates of shaft resistance for large databases of field tests. In order to investigate the divergence of experimental observation with field behaviour a series of field tests were performed with the UCD pile at a dense sand test bed site in Blessington, County Wicklow.

5. Experimental Programme

An instrumented pile was jacked at 2cm/sec from the base of a 1m deep starter borehole to a depth of 3.5m below ground level (bgl), using a 200kN capacity CPT truck. The pile used during the pile test was designed and built at University College Dublin. The pile is a 73mm diameter closed ended model pile with a flat base. Instrumentation on the pile consists of three levels of sensors described in terms of the distance from the pile base (h) normalized by the pile diameter (D),

see figure 6. The instrumentation consists of a total radial stress sensor and a pore water pressure sensor at three levels, $h/D = 1.5, 5.5$ and 10 . For soft soils two pressure sensors are embedded in the base plate to measure base total and porewater pressure directly. In dense/hard soils such as the Blessington sand, a blank end is used. The base resistance is measured using strain gauges placed near the pile tip. Further strain gauge arrays at $h=1.7m, 2.7, 3.7m$ and at the pile head allow the load distribution in the pile to be measured.

6. Soil Conditions

The pile tests were performed in sand deposited on the bed of a glacial lake. Melting of the ice sheet coupled with varying water table levels in the lake during sand deposition, and quarrying of the upper 15m of sand/gravel, has resulted in over-consolidation of the deposit with best estimates of the pre-consolidation stress of approximately 1MPa. The in-situ sand is at a relative density of close to 100%, has a bulk unit weight of $20.3kN/m^3$ and a degree of saturation of 71%. Eight cone penetration tests (CPT) were performed in the area of the pile installations, although two tests stopped prematurely on a gravel layer at about 1m bgl. The CPT end resistance (q_c) profiles revealed a variable deposit above 1m bgl.

Below this depth the CPT q_c profiles were reasonably consistent. Profiles of minimum, average and maximum q_c values are shown in Figure 7a for clarity. q_c values ≈ 15 MPa were measured from 1-2.5m bgl, increasing to 18 MPa at 2.5m bgl. Sleeve friction (f_s) was measured using a friction sleeve located at $h/D=3$ from the cone tip. The values plotted using a similar format in Figure 7b, are seen to be much more variable than the q_c profiles.

7. Pile Installation

The total pile installation resistance is shown in Figure 8a. Installation was halted when the resistance reached 191kN at 3.5m bgl (the available reaction being 200kN). The mean values of shaft resistance q_{sav} , defined as the shaft load/shaft area, measured during the installation of the instrumented pile (solid symbols) broadly fall within the minimum and maximum f_s profiles, Figure 8b. Many design correlations for shaft friction in sand use the CPT end resistance q_c , rather than f_s , which is a direct measure of the friction on the cone shaft. The difficulty of developing correlations based on cone friction measurements is highlighted by the inherent variability in these measurements. In addition, it is widely recognized that the sleeve friction measurements are affected by the proximity of the sleeve to the cone tip and by scale

effects (dilation) due to small diameter of commercial cones.

8. Static Load Test

Approximately one hour after the installation of the pile, a static compression, maintained-load test was performed. The load-displacement response of the pile is shown in Figure 9. The stiff response observed is typical of small diameter, jacked piles (Gavin and Lehane 2003). The axial resistance measured for a pile head displacement of 10% of the pile diameter (about 7mm) was 157kN. This increased only slightly to 165kN for larger displacements of up to 50% of the pile diameter. The latter load can be considered as the plunging failure load of the pile and is only 85% of the installation resistance measured at 3.5m bgl. The ultimate base resistance mobilized during both the installation of the pile and the static load test does not differ significantly and are approximately equal to the CPT q_c end resistance value. The difference between the installation and the static load resistances arises from differences in the shaft resistance.

9. Horizontal Stress Measurements

In order to fully understand the shaft behaviour, it is necessary to consider both the stationary and peak horizontal effective stresses at the pile-soil interface. During

continuous pile jacking, the stationary horizontal stresses could only be measured when the pile-head load was fully removed. The measurements made during the final jacking stage (including unloading) and during the static load test are shown in Figure 10. The peak installation values (solid symbols) were measured as the pile reached its final penetration depth. σ'_{rf} near the pile tip ($h/D=1.5$) approached 2000kPa, i.e. approximately 10% of the pile base resistance. σ'_{rf} reduced with increasing distance from the pile tip, to 1000kPa at $h/D = 5.5$ and 750kPa at $h/D = 10$. The stationary installation stresses represent the values due to one unload cycle only. These are seen to be approximately 50% of the σ'_{rf} values and exhibit a clear h/D trend with values remote from the pile tip being approximately 40% of those measured at $h/D = 1.5$. This suggests the presence of a zone of highly stressed sand near the pile tip after unloading. The large residual stresses recorded for this and similarly installed piles with residual base stress of up to $0.5q_c$ corroborate this.

When the pile was loaded for the second time during the static load test the peak σ'_{rf} load test values measured near the pile tip were much lower than the values measured during installation, 50% lower at $h/D = 1.5$ and 10% lower at $h/D = 5.5$. These reductions are such that the

h/D trend evident in the load test is greatly reduced in comparison to measurements made during pile installation. Unloading after the load test resulted in σ'_{rs} (stationary load test) reducing at all levels. Reductions were similar at all h/D levels (approximately 25%) with the result that σ'_{rs} values measured near the pile tip remained approximately double those measured at $h/D = 5.5$ and 10.

10. Discussion

The field measurements of horizontal stress in dense sand suggest that friction fatigue depends on both number of loading cycles and h/D . Large reductions in horizontal effective stress in a given soil horizon occur as the highly stressed zone of soil surrounding the base of the pile moves away from the zone under consideration. Additional reductions can occur due to load cycling. Evidence from multiple pile installations by Chatta (2005) suggest that the largest such reductions occur during the first load cycle and σ'_{hs} tends to stabilize after a relatively small number of load cycles. The importance of measuring the horizontal effective stress on model piles rather than the local or average shear stress is clear since the effects of stress relaxation can be masked by the tendency for dilation to occur during loading, and in particular the tendency for this

dilational component to be proportionally higher in situations where relaxation is greatest. Care should be taken when extrapolating shaft resistance values measured on small diameter piles or sleeve friction measurements to full-scale piles.

11. Driven Piles in Clay

The importance of the effect of stress changes during pile installation on the final capacity of piles is well acknowledged. Careful field experiments by Bond (1989), and Lehane (1992) suggest the principal factors governing the equalized lateral effective stresses in clays and silts include a measure of the soil's consistency i.e. $YSR = \sigma'_{vy}/\sigma'_{vo}$ determined in an oedometer test, or preferably q_c (Lehane et al 2000), a composite measure of the soil sensitivity and YSR (called the relative void index I_{vr}) and height above the pile tip (h). However, design approaches, which incorporate high quality field tests (CPT) and require good quality samples to determine consolidation characteristics, are not particularly amenable to everyday design.

Randolph (2003) found that while similarities existed in the normalized radial total stresses (σ_r/q_c) mobilized during the installation of piles in over-consolidated London Clay and Cowden Till, wide variations in the behaviour of piles driven into soft

clay were noted (see Figure 11). With this in mind, a series of pile tests were carried out at a soft silt test bed site to supplement the available database of installation measurements and investigate a simpler design estimate of equalised radial effective stress in clay using CPT data.

12. Soil Conditions

The test site is located at Kinnegar on the eastern shore of Belfast Lough, 10 kilometres from Belfast city centre. The site comprises a layer of recent fill about 1.2m deep overlying a thick sandy silt layer, approximately 2m thick, that in turn overlies a layer of very soft organic clayey silt, which is approximately 6m thick. The water table at the site is tidal, varying from 1 to 1.3m below ground level (bgl). The estuarine silt, known locally as sleet, was laid down over the past 3000 years in water depths between 3m and 9m Manning et al. (1970). Between 1m and 3m bgl. the proportion of sand and clay is 20% and 10% respectively, whilst below this level these proportions reverse (see Figure 12a). A summary of six CPT q_{net} profiles shown in Figure 12b, revealed that whilst q_{net} is quite variable in the sandy silt, the clayey silt is surprisingly uniform, with q_{net} varying from 175 kPa at the top, to 350 kPa at the bottom of the deposit. Uncorrected vane shear strengths were found to follow a similar trend

with depth, with s_{vane} increasing from 20 to 25 kPa with depth. The clayey silt has a relatively high plasticity index (PI=35%), a constant volume friction angle (ϕ'_{cv}) of 34°, and a yield stress ratio, which varies, from 1.5 at 3m bgl to 1.1 at 6m bgl. Detailed soil properties can be found in Lehane (2003), McCabe & Lehane (2003) and McCabe (2002).

13. Pile Installation

The instrumented pile used for testing was designed and constructed at University College Dublin. The pile was described previously for the pile tests in sands. The pile was hydraulically jacked into the ground at a rate of 2cm/sec, in increments varying from 100-400mm in length, to a total depth of 4.25m below ground level. As the pile is in sections ranging in length from 1 to 1.7m long, pause periods were required during installation to allow additional sections to be added. These resulted in a total installation time of 117 minutes. During installation the total stress and pore pressure sensors were monitored continuously. A load cell was used at the pile head to measure the axial load mobilized during pile jacking. Prior to installation a 100mm diameter 1.3m deep borehole was pre-augured through the fill. Static load tests, involving maintained load type compression and tension tests were performed nine days after pile installation. This paper will deal only

with the radial and pore pressures developed during installation and equalization.

The radial total stresses less ambient pore water pressure ($\sigma_r - u_0$) measured during installation by the three sensors, normalized by the net CPT end resistance, q_{net} value at that depth are shown in Figure 13(a). It appears from the figure that there is a tendency for $\sigma_r - u_0 / q_{\text{net}}$ to increase with depth, with values in the clayey silt (below 3m) being markedly higher than those in the sandy silt. Although there is some scatter in the data, the h/D effect described by Bond & Jardine (1991), Lehane & Jardine (1994) and others, where radial stresses at a given level reduce as the pile tip moves further away from the point are evident, with stresses mobilized at h/D=1.5 being generally $\approx 50\%$ higher than values recorded as the pile tip moves further from the point under consideration.

The excess pore pressures ($u - u_0$) generated during installation are plotted in a similar format in Figure 13(b). Again the pore pressures increased with depth, with pore pressures similar in magnitude to the total stresses being mobilized near the toe of the pile at the final penetration depth. As with the radial total stresses the h/D effect is also apparent for pore pressures. The pore pressures plotted in Figure 13(b) are the stationary pore pressures, measured immediately after the pile

has been pushed through a jacking increment. A tendency for pore pressures to fall during load application was noted. Immediately following removal of load the pore pressures rose rapidly, with the effect that near zero effective stress were recorded near the toe of the pile immediately after each loading increment. Similar behaviour was noted by Lehane and Jardine (1994) during installation of a jacked pile in soft clay.

14. Equalization

Once the pile reached its final penetration depth, all load was removed and the excess pore pressures ($\Delta u = u - u_0$) that developed during installation were allowed to dissipate to their ambient hydrostatic levels (u_0). The progress of dissipation is shown in Figure 14(a) where the excess pore pressures are normalized by the q_{cnet} value and plotted against time on a log scale to allow the full dissipation to be displayed. Dissipation of excess pore pressures around the small diameter pile were relatively rapid, with 50% dissipation occurring within the first 10 hours after installation was completed. The rate of dissipation at the furthest location from the base ($h/D = 10$) is slightly slower than the other two sensors, however, all levels reach full dissipation after 8-9 days. Pore water pressure dissipation results in reductions of the total stresses surrounding the pile.

Additional stress changes, which occur due to volume changes associated with consolidation, cause a redistribution of stresses surrounding the pile. Figure 14(b) shows the magnitude of the total radial stress reduction over a 9-day period. The radial total stress at the end of equalization is seen to be approximately 50% of the peak value at all locations. Final total stresses correspond to 20-25% of the q_{cnet} value at that depth.

15. Effective stress during Equalization

The variations in radial effective stress (σ'_r) with time in the Belfast sleech are compared in Figure 15 to similar measurements from a soft clay site (Bothkennar), and a glacial till (Cowden), both reported in Lehane and Jardine (1994). Initially large falls in radial effective stress are observed at Belfast during the start of equalisation. σ'_r is then seen to increase steadily from minima ($<0.1q_{cnet}$), to final values of approximately $0.25q_{cnet}$ similar to the end of installation values. An overall increase in radial effective stress is evident at $h/D=1.5$ at Belfast while sensors more remote from the pile base ($h/D=5.5, 10$) show slight reductions. These fully equalized values show little h/D dependence, with σ'_r / q_{cnet} reducing only slightly with increasing h/D . When compared with similar equalized data from

other sites it appears that despite large differences in normalized installation total radial stresses σ_r/q_{cnet} , σ'_r/q_{cnet} at all sites lie in the range 0.15-0.25.

16. Discussion

A closed ended pile experimented is presented for a clay site. The experiments have shown that very high porewater pressures develop during pile installation in Belfast sleech. A consequence of this is that σ_r/q_{cnet} values measured during pile installation are high relative to stiffer soils, where pore pressures may be low or even negative during pile installation. Measurements of equalized radial effective stress at a number of sites suggest that direct relationships between shaft resistance and CPT q_{cnet} for piles in silt and clay may be possible. Effective stresses in clay soils were influenced by the h/D effect close to the pile base.

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References

- American Petroleum Institute (API). 2000. RP2A: Recommended practice of planning, designing and constructing fixed offshore platforms- working stress design, 21st edition, Washington 59-61.
- Bond A.J. (1989), 'Behaviour of displacement piles in overconsolidated clays', PhD Thesis, Univ. of London (Imperial College).
- Bond, A. J. & Jardine, R. J. 1991. Effects of installing displacement piles in a high OCR clay. *Géotechnique* 41(3): 341-363.
- Bustamante M. & Gianceselli L. 1982. Pile bearing capacity prediction by means of static penetrometer CPT. Proc. 2nd European Symposium on Penetration Testing. (1):493-500
- Chatta, I. (2005) "Experimental investigation of the response of displacement piles to cyclic loading" MEng.Sc., University College Dublin.
- Chow F. (1997), "Investigations into the behaviour of displacement piles for offshore structures", PhD Thesis, Univ. of London (Imperial College).
- DeJong,J.T., Randolph M.F and White D.J. (2003) "Interface load transfer degradation during cyclic load: a microscale investigation" *Soil & Foundations* No. 43, pp81-93.
- Gavin K.G and Lehane B.M, (2003), 'End Bearing Resistance of Small Diameter Pipe Piles in Dense Sand', Proceedings of the International Conference on Foundation Behaviour, pp-321-330. Dundee, September.
- Jardine R.J, Chow, F.C, Overy, R and Standing, J (2005) *ICP Design Methods for Driven Piles in Sands and Clays*, Publishers Thomas Telford.
- Lehane B.M.(1992), "Experimental investigations of pile behaviour using instrumented field piles", PhD Thesis, Univ. of London (Imperial College).
- Lehane B.M. and Jardine R.J. (1994), "Shaft capacity of driven piles in sand: a new design approach", Proceedings of Conference on the Behaviour of Offshore Structures, Boston, 1, 23-36.
- Lehane, B.M. & Jardine, R. J. 1994. Displacement pile behavior in a soft marine clay. *Can Geotech. J.* 31(2): 181-191.
- Lehane, B.M, Chow, F, McCabe, B & Jardine, R. 2000, 'Relationships between shaft capacity of piles and CPT end resistance' *Geotechnical Engineering*, Volume GT143, Issue 2, pp 93-101.
- Lehane, B.M. 2003. Vertically loaded shallow foundation on soft clayey silt. Proceedings of the Institute of Civil Engineers, *Geotechnical Engineering* 156(1): 17-26.
- Lehane B.M., Scheider J.A, and Xu X., (2005) "CPT based design of driven piles in sand for offshore structures", Internal report No. GEO: 05345, University of Western Australia.
- Manning P.I., Robbie J.A. and Wilson H.E. 1970. *Geology of Belfast and the Lagan Valley*. Belfast: HMSO.
- McCabe B.A., 2002. *Experimental Investigations of Pile Group Behaviour in Soft Silt*. PhD thesis, University of Dublin (Trinity College).
- McCabe, B.A. & Lehane, B.M. 2003. Stress changes associated with driving pile groups in clayey silt. Proceedings of the 13th European Conference on Soil Mechanics and Geotechnical Engineering, Prague, 25-28 August 2003. Vol. 2: 271-276.
- Randolph, M.F. (2003) 'Science and empiricism in pile foundation design' Rankine Lecture, *Geotechnique* 53, No.10, pp847-875.
- White D.J and Lehane B.M. (2004), 'Friction fatigue on displacement piles in sand', *Geotechnique* 54, No.10 pp645-658.

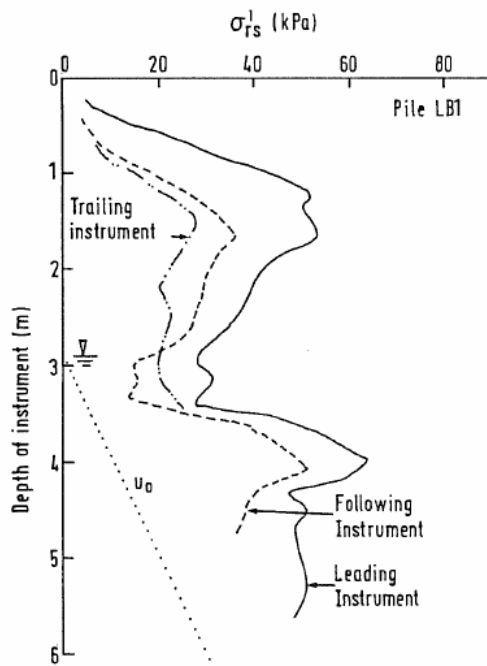


Figure 1: Pile installation at Labenne (after Lehane 1992)

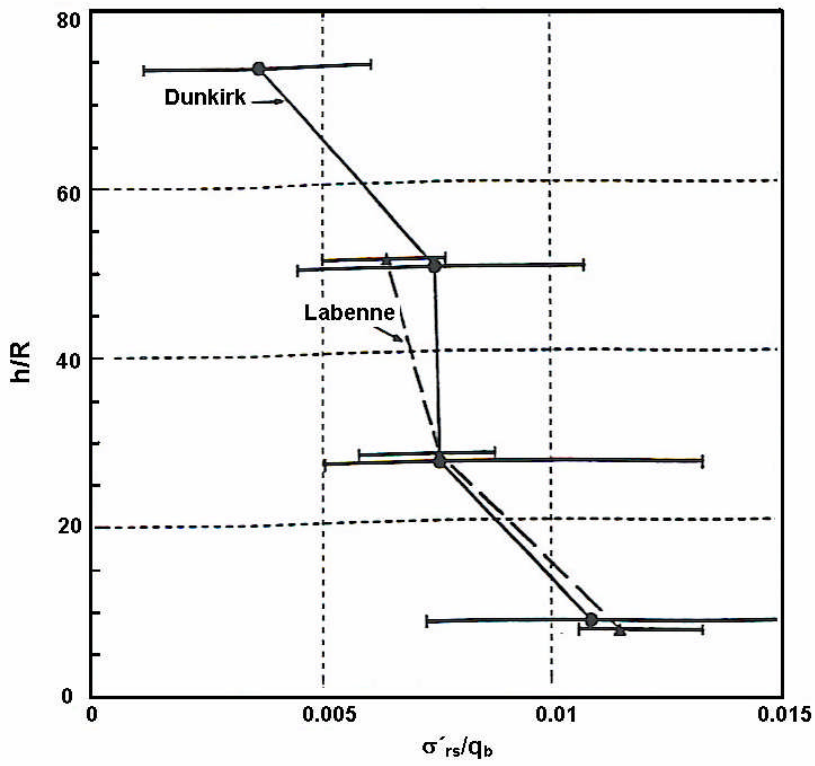


Figure 2: Normalised radial effective stresses (after Chow 1997)

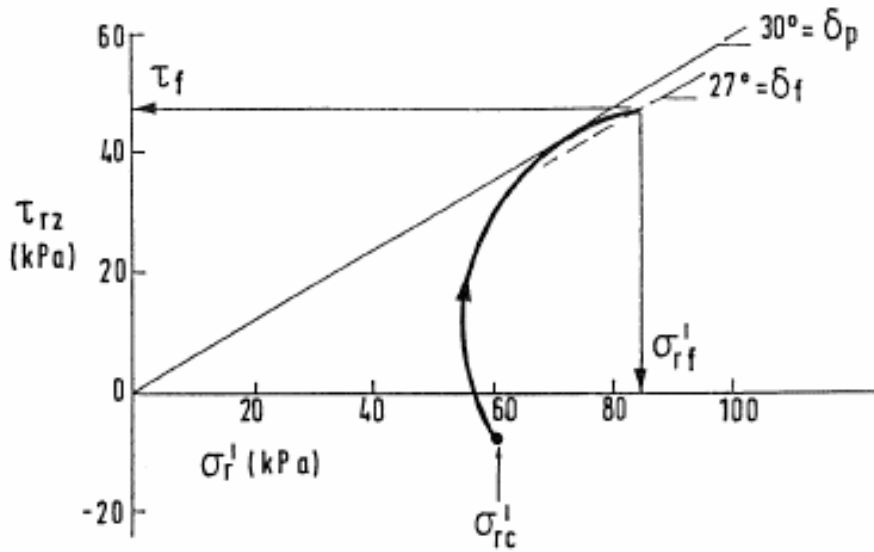


Figure 3: Effective stress path during loading of ICP (after Lehane 1992)

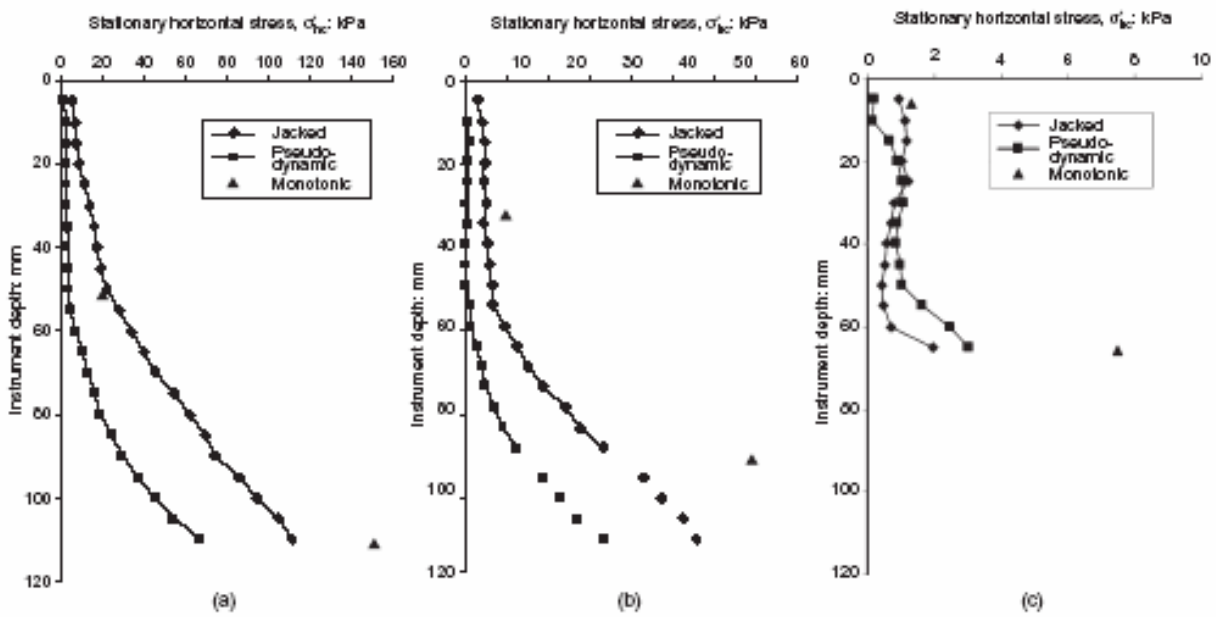


Figure 4: Stationary horizontal total stresses (after White and Lehane 2004)

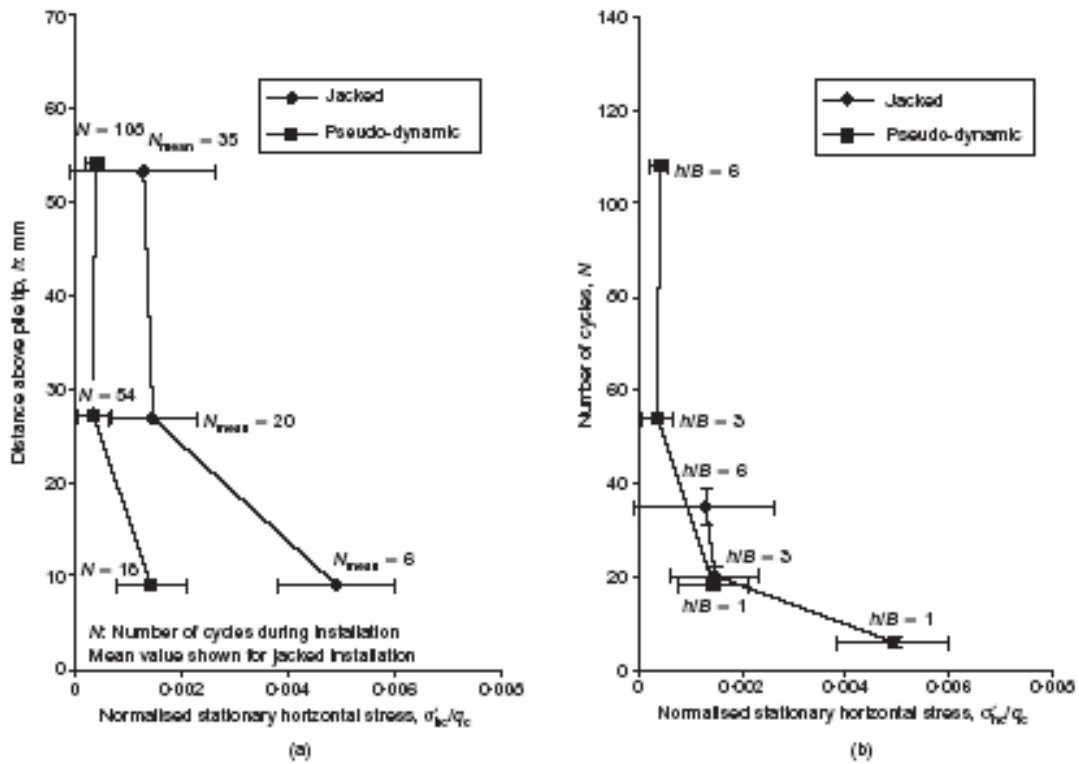
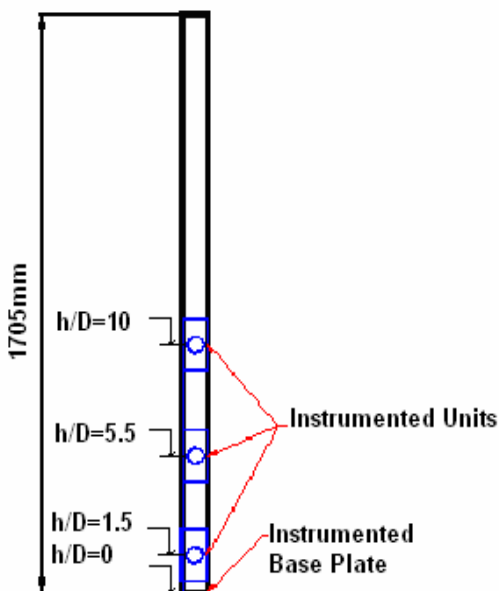


Figure 5: Normalised stationary horizontal stresses (after White and Lehane 2004)



The pile can be pushed to 6m bgl using a series of 1m long spacer sections with the same external diameter as the instrumented section. Each tube has strain gauge arrays at the top and bottom to measure load.

Figure 6: UCD closed ended pile

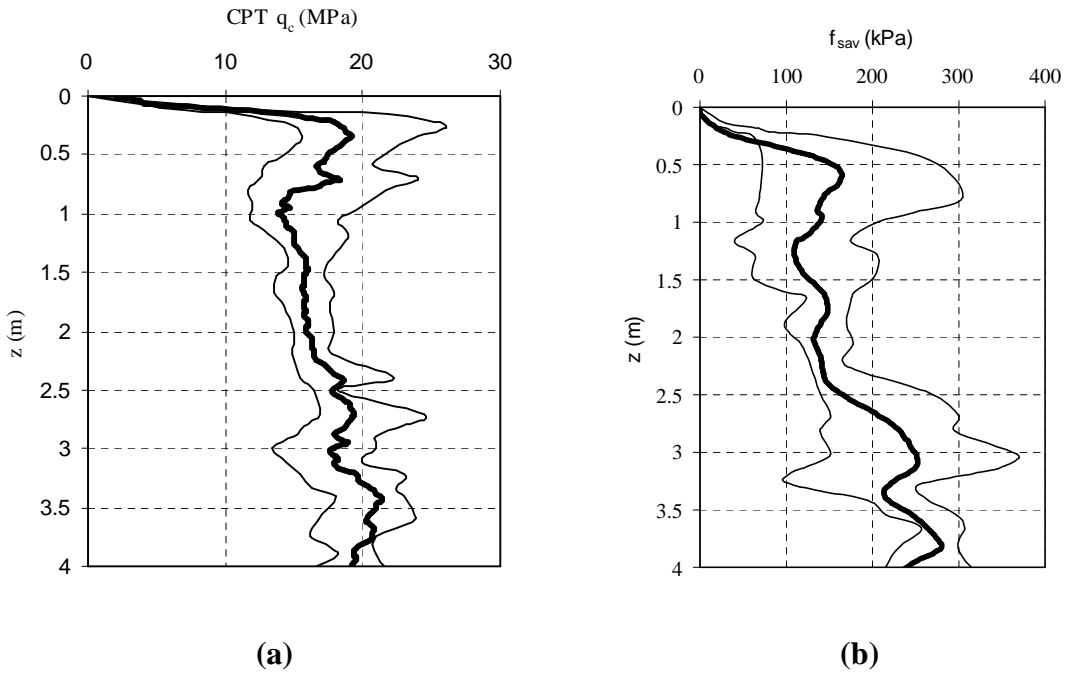


Figure 7: Cone Penetration Test CPT q_c and f_s data

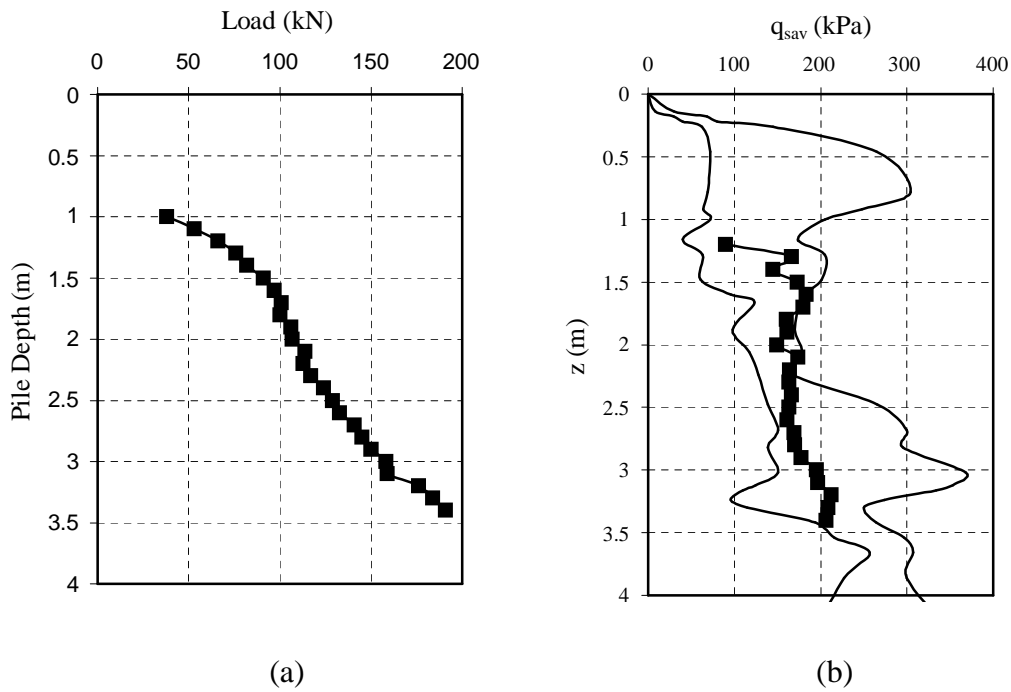


Figure 8: Pile head load and average shaft resistance during pile installation.

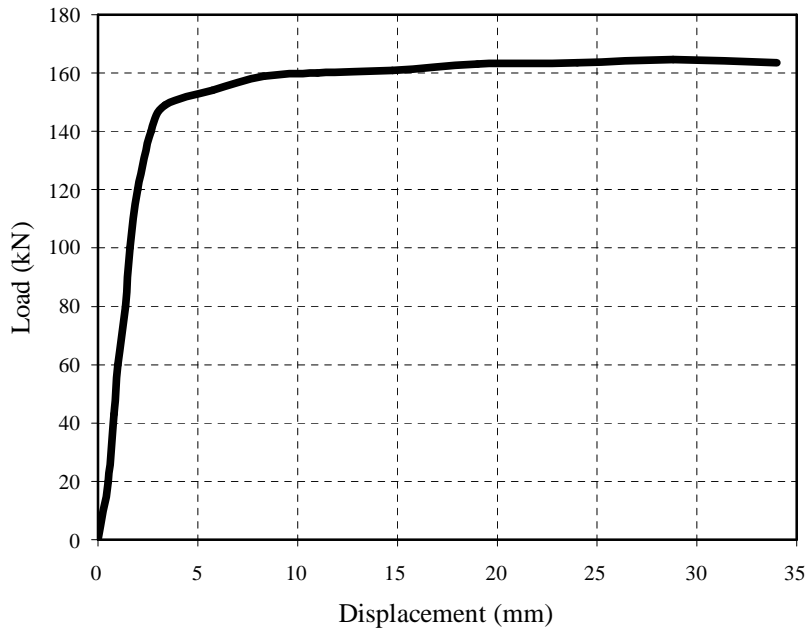


Figure 9: Load-displacement response during maintained-load test.

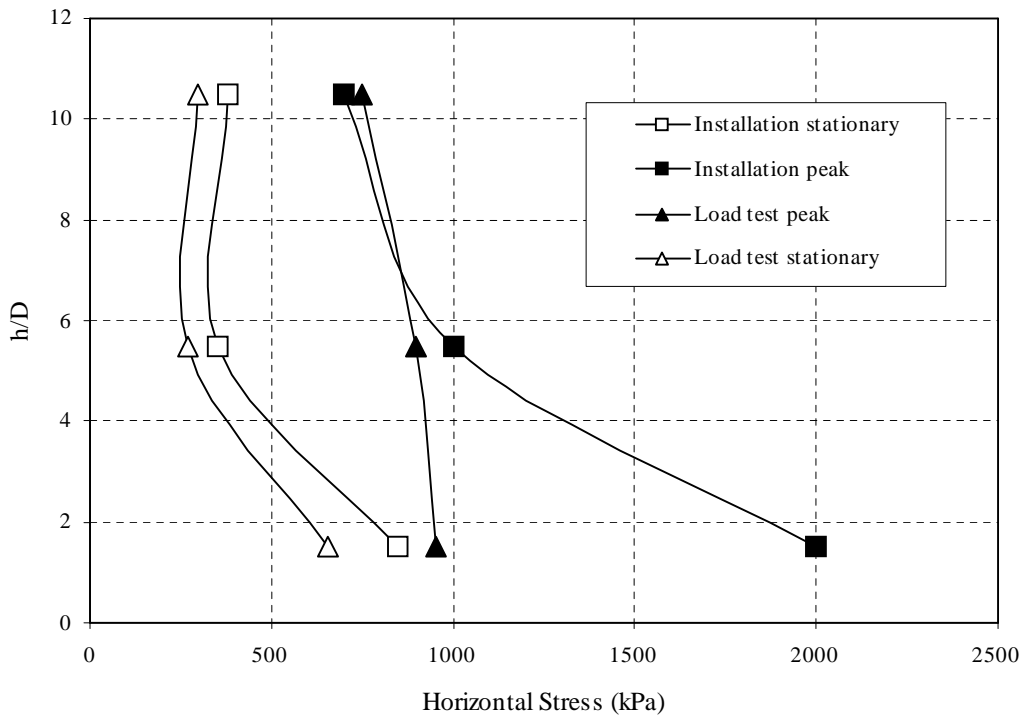


Figure 10: Horizontal stress mobilized at different h/D ratios

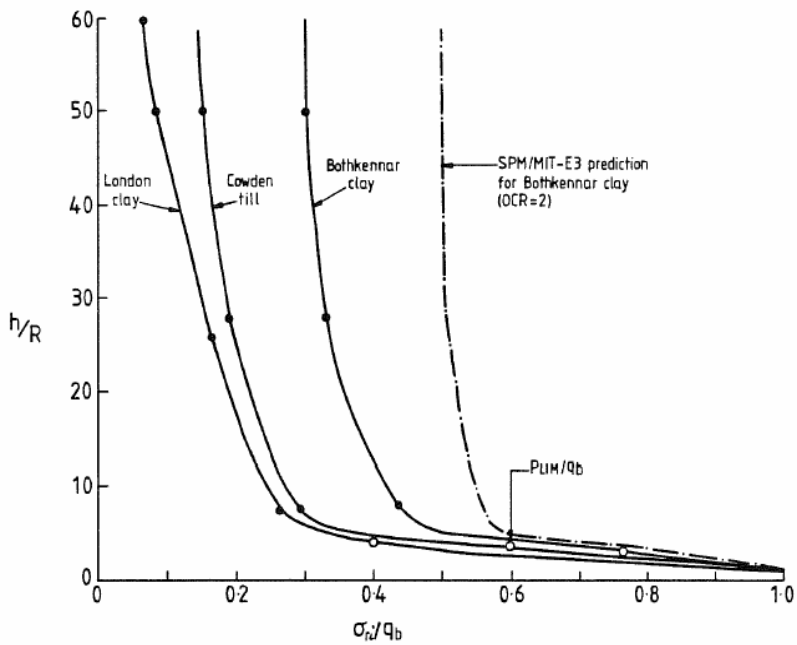


Figure 11: IC normalised radial stresses (after Lehane and Jardine, 1994)

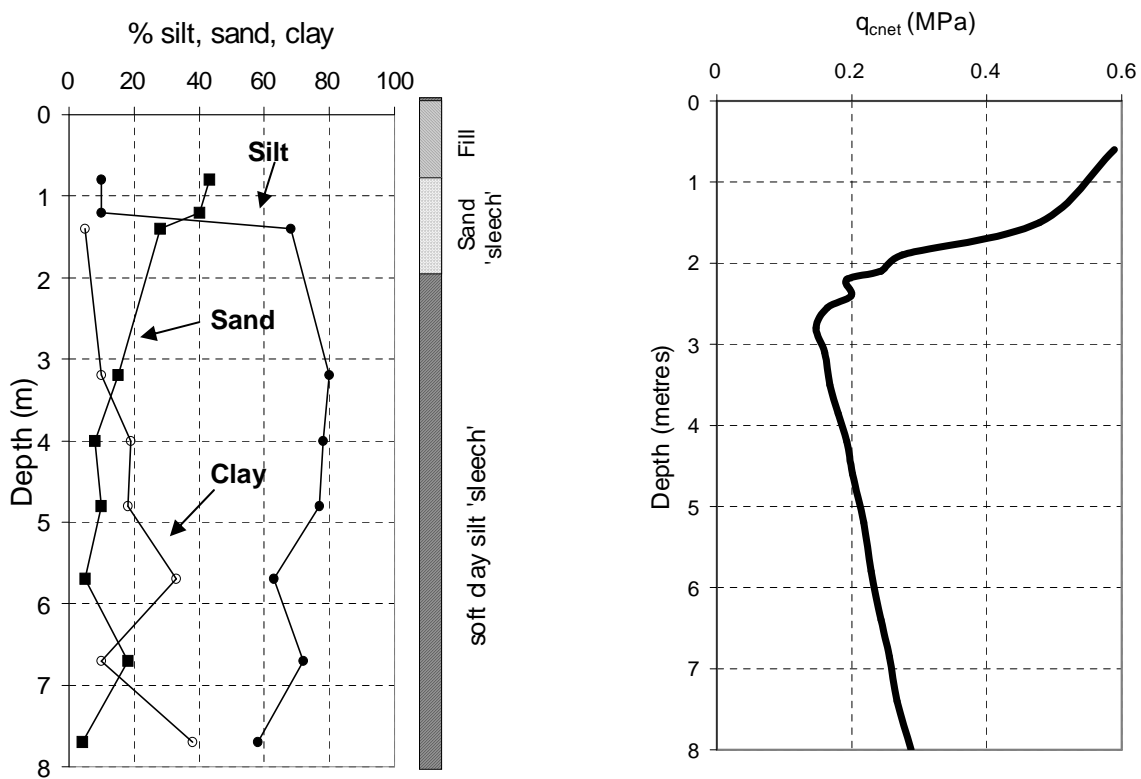


Figure 12: (a) PSD distribution (b) q_{cnet} profile for the test bed site at Belfast

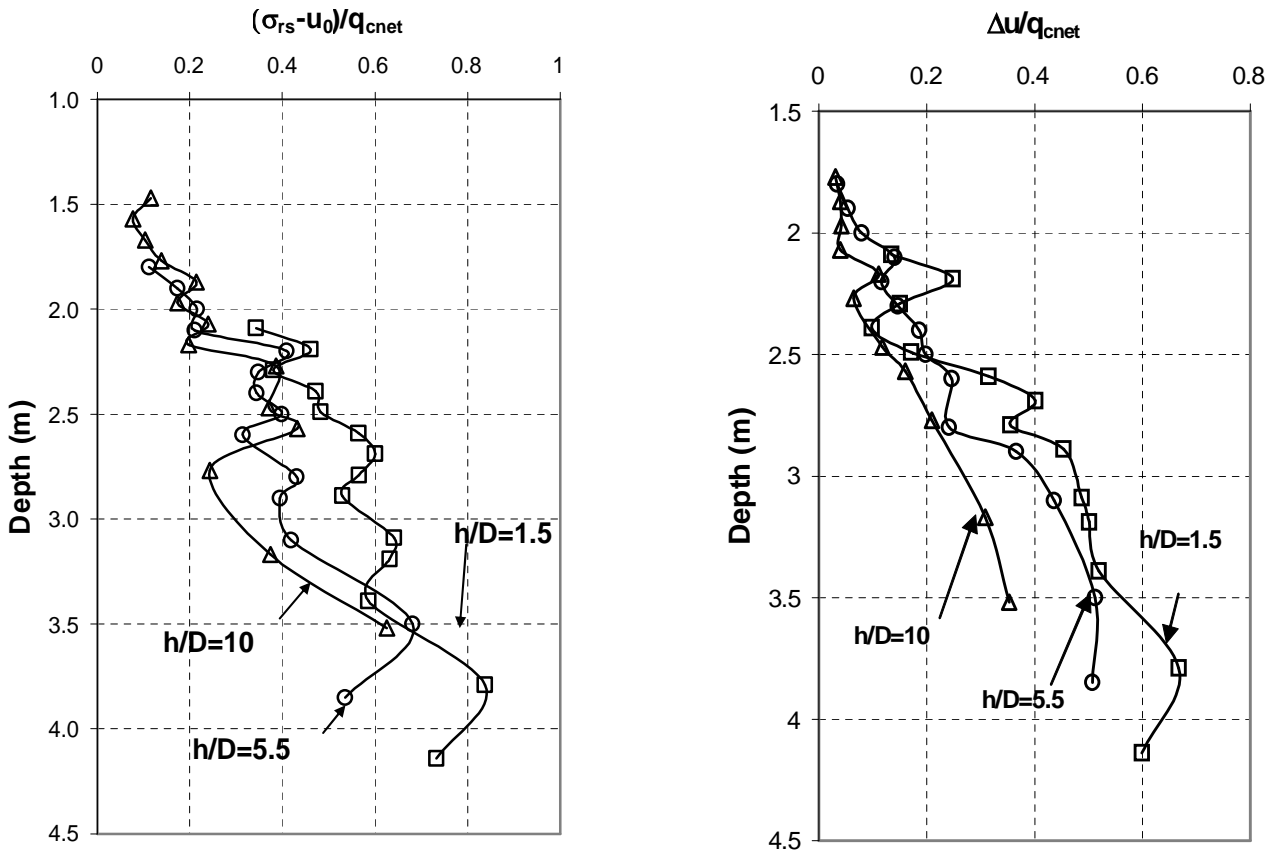


Figure 13: Installation (a) radial total stresses and (b) pore pressures

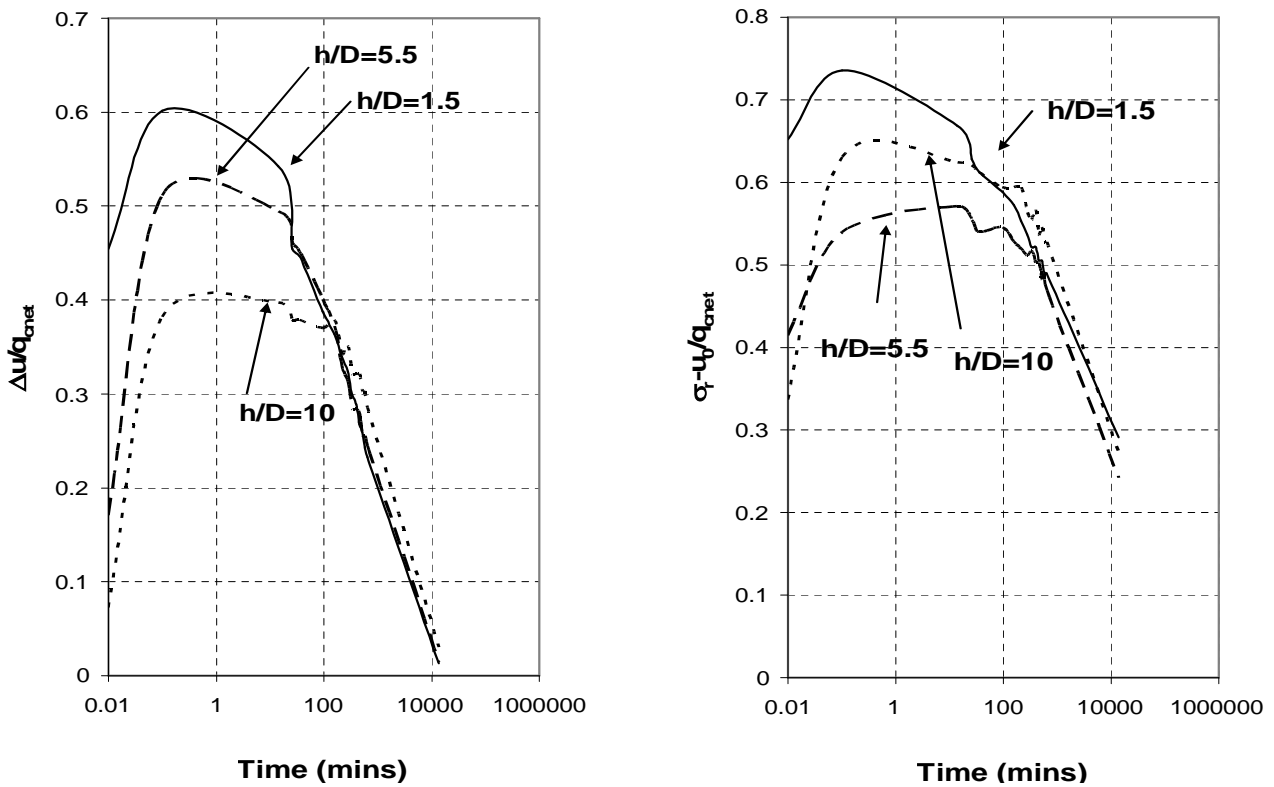


Figure 14: (a) pore pressure dissipation and (b) radial stress reduction.

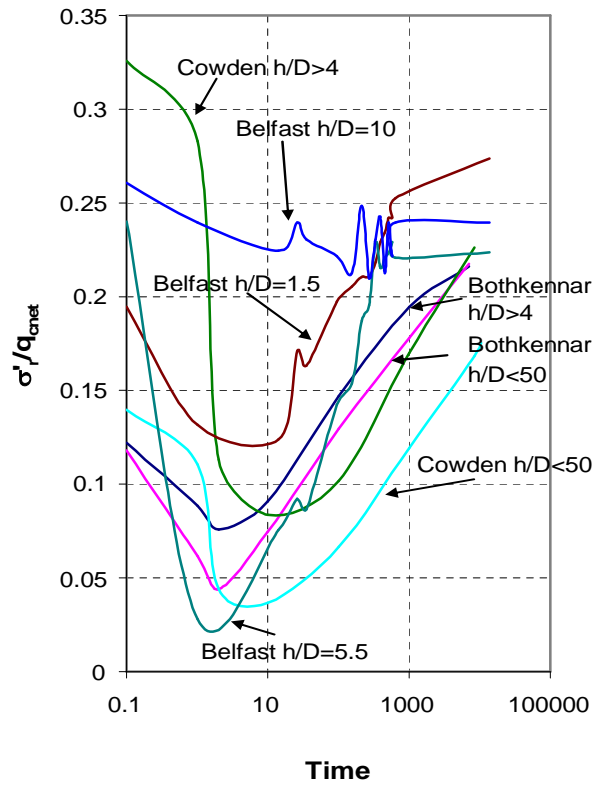


Figure 15: Equalization of normalized effective stresses