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The development and testing of an instrumented open-ended model pile

David Igoe¹, Paul Doherty² and Kenneth Gavin³

¹David John Paul Igoe, Research Assistant, School of Architecture, Landscape & Civil Engineering, University College Dublin, Newstead Building, Belfield, Dublin 4, Ireland.
Tel: +353 (0) 716 3229, fax: + 353 (0) 7167399, email: david.igoe@ucd.ie

²Paul Doherty, Research Assistant, School of Architecture, Landscape & Civil Engineering, University College Dublin, Newstead Building, Belfield, Dublin 4, Ireland.
Tel: +353 (0) 716 3229, fax: + 353 (0) 7167399, email: paul.doherty@ucd.ie

³Dr. Kenneth George Gavin, Lecturer, School of Architecture, Landscape & Civil Engineering, University College Dublin, Newstead Building, Belfield, Dublin 4, Ireland.
Tel: +353 (0) 716 3222, fax: + 353 (0) 7167399, email: kenneth.gavin@ucd.ie

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Abstract

This paper describes the development of a model instrumented open-ended (pipe) pile. The importance of model geometry and separating the shaft, annular and plug load and horizontal effective stresses are discussed. A detailed description of the construction of the twin-walled open-ended pile is presented. Particular attention was given to protecting the fragile instrumentation from the rigours of installation and the effects of water ingress. Calibration procedures which were used to verify the instrument reliability are also discussed. The final section describes field tests conducted in both a loose sand and a medium-dense sand deposit, which are used to validate the instrument performance.

Nomenclature

Q_T	=	axial pile capacity, N
Q_b	=	end-bearing capacity, N
Q_s	=	shaft capacity, N
q_s	=	unit shaft resistance, Pa
q_{plug}	=	plug resistance, Pa
q_{ann}	=	annular resistance, Pa
q_c	=	CPT cone resistance, Pa
A_{plug}	=	area beneath the pile plug, m ²
A_{ann}	=	area beneath the pile annulus, m ²
A_s	=	area of the pile shaft, m ²
R_a	=	surface roughness, m
R	=	pile radius, m
R_i	=	internal pile radius, m
D	=	pile diameter, m
t	=	pile wall thickness, m
h	=	distance from pile tip, m
L	=	pile length, m
ΔL_p	=	change in plug length, m
ΔL	=	change in pile penetration, m
σ'_r	=	radial effective stress, Pa
σ'_{rc}	=	equalized radial effective stress, Pa
$\Delta \sigma'_{rd}$	=	increase in radial effective stress during loading, Pa

τ_f	=	external unit shaft friction at failure, Pa
τ_s	=	external unit shaft friction, Pa
τ_{si}	=	internal unit shaft friction, Pa
δ	=	interface friction angle
σ'_{v0}	=	in-situ vertical effective stress, Pa
P_{atm}	=	atmospheric pressure = 100 kPa
G	=	soil shear stiffness, Pa
Δr	=	interface dilation, m
IFR	=	Incremental Filling Ratio
CPT	=	Cone Penetration Test
UCD	=	University College Dublin

Introduction

In recent years, a drive to increase the proportion of the world's energy supply derived from offshore renewable and non-renewable sources has led to rapid expansion in the offshore piling industry. Development of these resources brings new challenges for pile designers as higher capacities and deeper penetrations are required. Many of the design methods currently employed in the offshore industry are based on techniques developed for onshore piles, which are typically closed-ended steel or concrete piles with diameters less than 0.4 m. In contrast, offshore displacement piles are usually open-ended steel tubes with diameters between 0.6 m and 5 m. The axial capacity of displacement piles are largely controlled by the stress changes during installation, which in turn are influenced by (i) the high base stresses mobilized by the advancing pile tip and (ii) the increased soil density required to accommodate the pile volume. Due to the difference in end condition, open ended piles typically displace approximately 20% of the volume of soil of a closed-ended pile of equal external diameter (Lehane and Randolph 2002). Therefore, extrapolating design methods developed for closed-ended piles to open-ended geometries requires assumptions which are somewhat questionable.

This paper describes the design, construction, calibration and field testing of an instrumented model pipe pile which was developed to provide information on the plug and annular base resistance and horizontal effective stress at multiple levels along the pile shaft during installation and static load testing.

Background

The axial capacity of a pile (Q_T) is derived from end bearing resistance (Q_b) and external shaft resistance (Q_s).

$$Q_T = Q_s + Q_b \quad (1a)$$

$$Q_s = \tau_s A_s \quad (1b)$$

$$Q_b = (q_{\text{plug}} A_{\text{plug}} + q_{\text{ann}} A_{\text{ann}}) \quad (1c)$$

Where A_s is the area of the pile shaft, and A_{plug} and A_{ann} are the areas beneath the pile plug and annulus respectively. In the case of open-ended piles, the unit base resistance is composed of two components, namely; the plug resistance (q_{plug}), which is the unit end bearing resistance which can develop beneath the pile plug as a result of the internal shear stress (τ_{si}) developed between the pile plug and pile wall, and the bearing resistance developed under the annular area at the pile base (q_{ann}) as shown in Fig. 1. Since different mechanisms control both these components of base resistance, the instrumentation provided on a pile must be capable of measuring both components separately.

Instrumented pile tests reported by Gavin and Lehane 2003, and Paik and Salgado 2003 have shown that the base resistance of pipe piles is strongly affected by the mode of penetration or amount of plugging experienced during installation. The degree of plugging is best understood by comparing the incremental change in the soil plug length (ΔL_p) during installation to the change in pile penetration (ΔL) using the Incremental

Filling Ratio ($IFR = \Delta L_p / \Delta L$). IFR can vary from 1 when the pile is fully coring to 0 when the pile is fully plugged (a condition which is analogous to closed-ended pile penetration). This is also illustrated schematically in Fig. 1. Similarities between the trends observed from model and database studies of full scale piles (See Gavin and Lehane 2005 and Xu et al. 2008) suggest that once the effect of plugging is considered, scale-effects due to the diameter of piles on the mobilized base resistance measurements are unimportant, and model pile tests can be used to infer field scale behavior.

The base resistance (q_b) developed during installation also affects the horizontal effective stress regime around the pile shaft, which controls the shaft resistance. Recent research into the behavior of full displacement piles has led to the development of reliable CPT based effective stress design methods such as IC-05 (Jardine et al. 2005) and UWA-05 (Lehane et al. 2005). A particular advantage of the research described by Jardine et al. 2005, which underpins both approaches' estimation of the shaft resistance of closed-ended piles is the results from a series of instrumented pile tests. The closed-ended Imperial College Pile (ICP) measured the radial effective stress (σ'_r) at a number of locations along the shaft during installation and static loading. The equalized radial effective stresses (σ'_{rc}) measured during installation were found to be dependent on the CPT end-bearing q_c ($\approx q_b$) values. The effects of friction fatigue (whereby the σ'_{rc} value in a given horizon reduced as the pile was driven further past the point) were incorporated using the h/D term in both the IC-05 and UWA-05 design methods (Jardine et al. 2005 and Lehane et al. 2005 respectively):

$$\tau_f = (\sigma'_{rc} + \Delta\sigma'_{rd}) \tan \delta \quad (2a)$$

$$\sigma'_{rc} = 0.027 \cdot q_c \cdot (h/R)^{-0.38} (\sigma'_{v0} / P_{atm})^{-0.12} \quad \text{IC-05} \quad (2b)$$

$$\sigma'_{rc} = 0.03 \cdot q_c \cdot (h/D)^{-0.5} \cdot \quad \text{UWA-05} \quad (2c)$$

where $\Delta\sigma'_{rd}$ is the increase in horizontal stress during loading, δ is the interface friction angle, R is the pile radius, σ'_{v0} is the in-situ vertical effective stress, and P_{atm} is the atmospheric pressure ($\approx 100\text{kPa}$).

The $\Delta\sigma'_{rd}$ term in Eqn 2a relates to the horizontal stress increase at the pile-soil interface during loading. The amount of dilation experienced is dependent on the surface roughness of the pile, and the shear modulus of the soil, and is inversely proportional to the pile diameter, leading Lehane 1992 to observe that dilation can dominate the results of model pile tests. $\Delta\sigma'_{rd}$ is usually estimated using a cavity expansion approach:

$$\Delta\sigma'_{rd} = 4 \cdot G \cdot (\Delta r / R) \quad (3)$$

where G is the soil shear stiffness and Δr is the interface dilation (which may be taken as twice the pile surface roughness). Attempts to minimize scale-effects can include using smooth-walled stainless steel piles. The dominant effect of pile diameter means that in order to quantify dilation and remove this scale-effect from the interpretation of model

pile test results, it is essential that horizontal stress sensors be provided on model piles designed to investigate shaft resistance.

By adopting different assumptions with regard to differences between the horizontal stress regimes set up by closed and open-ended piles, both the IC-05 (Chow 1997) and UWA-05 (White 2005) extended their equation for closed-ended piles to predict the shaft resistance of pipe piles. However, no comparable measurements of σ'_r from pipe piles were available to assess the validity of these assumptions. In an effort to increase our understanding of the mechanisms controlling the shaft and base resistance developed by offshore pipe piles, it is essential that such measurements are obtained.

The UCD Open Ended Pile

Design Criteria

The UCD open ended model pile was designed to be representative of typical offshore piles while considering the physical constraints associated with transport, manual handling and testing. Portable jacking rigs or a CPT truck were used for installation, and as such, the UCD Open-Ended (OE) pile was designed to be maneuverable by hand with a minimum number of site personnel. Multiple tests scheduled for a range of soil types, in both clay and sand, required that the pile was reusable, robust, and watertight. In order to

be characteristic of offshore piles, an appropriate ratio of both diameter to wall thickness (D/t) and Length to Diameter (L/D) had to be chosen. While piles with diameters as large as 5.1 m have been employed for offshore wind turbines, these are the exception. Recent trends in offshore foundation use indicate that mid-range diameter piles are more common. Overy 2007 reports on the range of pile sizes employed in the North Sea noting that diameters ranged from 0.66 to 2.134 m and penetration lengths from 26 m to 87 m. Jardine and Chow 2007 suggest that D/t ratios vary between 15 and 45, with an average of 27.

Since the primary difference between the open- and closed-ended piles lies in the manner in which the pile carries the applied load, the open-ended pile must allow for separate measurement of the total pile resistance and for three components namely: (i) the annular resistance (ii) the internal shaft load and (iii) the external shaft capacity. In addition, measurements of shear stress, radial total stress and pore pressures at different levels along the shaft wall allow the fundamental mechanisms concerning the pile capacity to be determined.

General Description

The primary component of the UCD-OE pile is a fully sealed instrumented unit which is 2 m long and has an outside diameter of 168 mm. The instrumented section adopts the twin-walled construction pioneered by Paik and Lee 1993, which consists of two cylindrical hollow stainless steel tubes, where the smaller tube slides into the larger tube

and all the instrumentation is housed in the internal void between (see Fig. 2). To meet both the geometric and maneuverability requirements various combinations of outer and inner tubes were considered. To achieve the optimum D/t ratio, an outer tube with an external diameter of 168 mm and a wall thickness of 3.34 mm was chosen, while the inner tube had an external diameter of 154 mm and a wall thickness of 2 mm. The resulting void space between the two tubes was 3.66 mm. The total wall thickness of the twin-walled pile was 9 mm and the internal diameter was 150 mm. The pile has a D/t ratio of 18.7 which falls within the range of typical offshore piles suggested by Jardine and Chow 2007. A stainless steel annular ring was welded to the inner tube. A small gap was maintained between the annular ring and the external wall of the pile to ensure full separation of external shaft and base loads. The top of the instrumented section consists of two separate stainless steel capping plates welded to the inner tube and outer tube respectively. Once the outer and inner tubes were fitted together, the two top caps were bolted to form a rigid connection. A nut welded to the inside of the lower cap allowed insertion of a driving bar, or an intermediate connection section to allow extension pieces to be added. The extension pieces are 0.75 m in length and fit together via an interlocking toothed system, which utilizes 8 mm diameter bolts to rigidly connect two sections. A total of five extension pieces were fabricated which allow a maximum penetration depth of 5.75 m ($L/D = 40$).

Instrumentation

The void between the two tubes houses all the instrumentation required to measure radial total stress, shear stress and pore pressure. A schematic of the instrumentation depicting the sensor layout is illustrated in Fig. 2. The outer tube contains three radial stress and three pore pressure sensors along the shaft. The sensor locations are shown in terms of distance from the pile base (h) normalized by the pile diameter (D). The h/D values used are 1.5 for the leading sensors, 5.5 for the trailing, and 10.5 for the lagging pair of sensors. The total stress and pore pressure sensors are positioned diametrically opposite each other, to allow effective stresses to be determined at each level. 120Ω uniaxial foil strain gauges were used to measure the axial shaft load distribution in each tube during installation and load testing.

The strain gauge clusters located at $h/D = 0$ and 12 on both the internal and external tube each contained four gauges, whilst all other levels have three gauges per set. The strain gauges were used to separate the load distribution through the inner and outer tube. They were installed by preparing the steel tubes to remove any discontinuities and ensuring a suitable bonding surface. Careful abrasion using emery paper and swabbing with acetone provided a clean, flat mounting surface. The lead wires were then taped to the metal surface in pre-designated routes, to insure no crossing while the tubes were sliding together, ultimately leading to the cable exit slots located in the pile caps. Gauges located at a particular level were positioned equidistant from each other to negate any bending effects.

In addition, the base section which was welded to the inner tube houses total stress and pore pressure transducers mounted flush with the base which allows the annular load and effective stress to be determined. Pressure measurements were conducted using KyowaTM PS-5KA sensors for the shaft and PS-5KB pressure sensors for the base. These sensors were primarily chosen due to their ultra thin structure, as the 0.6 mm thickness could be accommodated in the wall of the outer tube with minimal fabrication and without compromising the overall D/t ratio. The pressure transducers were installed into the pile so that the sensing face was flush with the pile circumference. Protrusion of the sensing face beyond the pile surface could lead to stress concentrations forming around the sensor and thus inaccurate results. The small diameter of the sensing face (6 mm) minimizes the distortion of the shaft geometry caused by the curvature of the tube. The sensors have a capacity of up to 500 kPa, which is sufficient for measuring shaft stresses in medium dense sand while maintaining the required sensitivity for measuring stresses in soft clay. The rated output is ≈ 1 mV/V with output sensitivity for the shaft sensors of 0.910 mV/V while the base sensors have an output sensitivity of 0.823 mV/V. The transducers operate under a temperature range of -20° C to 70° C with a thermal effect on output of $\pm 0.2\% \text{RO}/^\circ\text{C}$. Installation of the transducers required bonding the underside of the sensor to the pre-polished surface machined in the pile wall with a specialist RC19 adhesive. Additional details in relation to the sensor specifications and installation process can be found in Gallagher (2006). In order to measure pore pressures accurately a saturated porous disc was required to separate the soil stress and water pressure, thus allowing pore pressure only to act on the transducers sensing face. The choice of saturation fluid was

determined primarily based on its mechanical properties as the fluid should have a viscosity high enough to prevent diffusion of gas into the reservoir and low enough to allow saturation of the ceramic disc (Bond et al., 1991). In accordance with these requirements Glycerin BP saturation fluid was determined to be suitable.

A light weight disc attached to a graduated wire was inserted through the pile cap. The disc, which was lifted into contact with the underside of the pile cap during an installation jacking stroke, was subsequently lowered onto the top of the plug to allow measurement of the pile plug development during the installation process. An external load cell at the pile head measured the total load during installation and load testing, and two Linear Variable Displacement Transducers (LVDT's) mounted on the pile cap were used to measure pile head displacement during all load tests.

Construction Details

Lateral Stress Sensor Arrangement

To accommodate the total stress sensors which are housed directly in the wall of the outer tube, a 6.5 mm diameter, 1 mm deep slot was machined perpendicular to the shaft surface, with a 0.5 mm diameter channel drilled through the corner of the slot to allow the transducer lead wires pass through the outer tube and travel along the inner surface of this tube.

The additional depth (wall thickness) required to accommodate the pore pressure measurement sensing arrangement resulted in separate cut-out units being manufactured (See Figs. 3 and 4). A 48 mm diameter section was removed from a steel circular hollow section with an outside diameter of 168 mm and wall thickness of 6 mm. The pore pressure sensor was fitted into this unit as shown in Fig. 4, with the transducer recessed from the external circumference. The porous ceramic was placed in front of the transducer sensing face mounted on a ledge 0.5 mm above the transducer face. A reservoir was provided between the back of the ceramic disc and the transducer face providing direct contact between the surface of the saturated ceramic and the transducer. This allowed pore pressure to act immediately on the sensing face. The reservoir was cylindrical with a diameter of 7.6 mm and a height of 0.5 -1 mm. The screw and channel arrangement link the external shaft to the underside of the ceramic disc, which allowed the saturation fluid pass through to the reservoir and out through the ceramic disc, removing air and saturating the disc until it reaches the surface. Saturation is conducted by injecting fluid (using a syringe) into the saturation channel and ensuring the screw tightens against the O-ring seal which forces the fluid through the porous disc resulting in complete saturation, which was observed at the ceramic face as the disc begins to 'sweat'. The fired porous ceramic chosen is tough and durable and unlike porous metals that may oxidize or leak, porous ceramics are generally inert and can be produced with very consistent and uniform pore structures. Following installation the ceramic was abraded until it formed a smooth continuous curved profile with the pile shaft, ensuring no physical unconformities. Following manufacture of the pore pressure units they were welded into pre-machined holes in the wall of the instrumented pile.

Base Unit

The stainless steel annular ring welded to the inner tube is shown in Fig. 5. The KyowaTM PS-5KB sensors employed for this application had a 90° cable exit structure that was more suitable to the mounting conditions at the pile annulus. The transducers fit into preformed holes using the same installation procedure as the shaft sensors. However, in this instance, the cabling is attached to the inner tube, and exits at a different location in the pile cap. The same ceramic pore pressure measurement procedure was adopted as in the shaft. However, the ceramic disc is elliptic in order to fit within the geometric confines of the annular ring.

Top Cap Unit

The pile top cap units were machined from stainless steel and are joined together by four M8 hex-head bolts (See Fig. 6). The overall thickness of the centre portion of both caps is limited to 15 mm to reduce the piles self-weight. A 24 mm stainless steel nut was welded below the surface of the inner tube cap which provided the connection point between driving rods and the pile. Cable exit points are illustrated schematically in Fig. 6. The cables for the external tube exited through two slots at the perimeter of the pile cap. Cables from the inner tube were brought through an angled channel before exiting through the two holes provided in the pile caps. The cap could be removed and be connected to the top of the spacer sections.

Intermediate Connection Section

To allow additional extension pieces to be added to increase the pile length, an intermediate connection section was required. The connection piece consisted of a solid aluminum spacer, threaded into the instrumented section and an adaptive collar with holes machined to allow cable passage through the pile. The adaptive collar was machined from the same steel tube as the outer tube of the instrumented section, and welded to a steel plate 15mm thick to spread the load through to the spacer section. Additional threaded holes were drilled horizontally into the adaptive collar to allow connection of the interlocking extension system. A thin rolled steel sleeve then slid over the collar as far as the instrumented unit providing protection for the cable connection points. This resulted in a relatively uniform shaft profile.

Sealing

Due to the sensitivity of the instrumentation, prevention of water ingress into the instrumented void was deemed a priority. Directly above the annular section, which was welded to the inner tube, a small gap existed between the annulus and the outer tube (See Fig. 7). This gap was filled with a flexible silicone based sealant to prevent water ingress into the void space during pile installation whilst simultaneously preventing load transfer between the outer and inner tubes. Additional sealing was provided to all strain gauges by initially coating them with a silicon based lacquer before covering this with a thin wax

layer. Sealing at the top of the pile was provided by an o-ring fitted into a groove between the two steel caps. The o-ring was pinched as the closing nuts were tightened to ensure a watertight connection. Cable exit points were protected by rubber glands and a secondary flowable silicon seal.

Calibration and Instrumentation Performance

Throughout installation and load testing of the UCD model pile, the following measurements were recorded:

- (i) Pile head load using a 250kN load cell
- (ii) Load distribution along the pile shaft using strain gauges
- (iii) Total Stress and Pore Pressure acting on the pile wall
- (iv) Displacements using Linear Vertical Displacement Transducers (LVDT's)

Calibration of all instrumentation was conducted before and after all site tests, to ensure the reliability of the data obtained and to assess if any zero drift had occurred.

Load Cell

The pile head load cell was calibrated using a 35 tonne concrete cube testing machine. The load cell demonstrated the expected linear behaviour and the required sensitivity for site testing.

Strain Gauges

The strain gauges were calibrated by mounting the pile in a steel frame and applying a vertical axial load to the pile through a hydraulic jack, as illustrated schematically in Fig. 8. The strain gauges were observed to obey Hooke's law, with the gauges placed on the internal wall exhibiting equal strain at all levels on the pile wall, while the outer tube exhibited no load transfer. This is because in the twin-walled load transfer mechanism, the gap between the annular base ring and the external pile wall precluded load transfer to the external wall. Confirmation of the consistency of the strain gauge calibration was obtained during all pile installations, as measurements of the pile head load cell directly corresponded to the local strain gauge output at each measurement level until the gauges at that level penetrated the soil surface.

Pressure Transducers

The pressure transducers used for the measurement of both radial total stress and pore pressure were calibrated immediately following manufacture and arrived with a pre-determined calibration factor. They were again recalibrated following insertion of the sensors into the pile wall. The arrangement designed to allow calibration of the sensors which utilised a head of water pressure measured against a mercury manometer is shown in Fig. 9. A moulded plastic cover was clamped to the pile shaft with a rubber gland cast in the recess between. Once tightened, the clamp formed a complete seal around the part

of the shaft containing the sensor and the water pressure was applied directly through a valve installed in the plastic cover.

The sensors were calibrated by both loading and unloading with no hysteretic behaviour observed, and typical linear responses provided a regression coefficient of approximately 0.99 (see Fig. 10). In order to observe the effect of temperature fluctuations and drift, the sensors were logged continuously for a one week period whilst the pile was in the laboratory. The maximum drift observed of 1 kPa was deemed acceptable for site testing. In addition the pressure sensors were recorded throughout the strain gauge calibration procedure described in the previous section. The application of relatively large axial loads caused no horizontal stress readings indicating that cross-sensitivity (change in horizontal stress due to axial loading) was not an issue.

Linear Variable Displacement Transducers (LVDT's)

Two LVDTs were used for each site test to measure the vertical pile head displacement under the applied loads. The LVDTs were mounted on an independent reference beam, allowing relative movements to be recorded. The LVDTs required a 10V excitation with a millivolt output. Calibrations were performed on-site against a digital micrometer accurate to 0.01mm prior to testing.

Experimental Results

Overview

The experimental results presented in this section are used to validate the instrument performance and highlight the importance of the specific instrumentation incorporated into the UCD-OE model pile. In particular the importance of load distribution, radial and annular stress measurements are demonstrated by comparison with existing approaches/assumptions commonly used to estimate design values for open-ended piles.

The field tests were performed in a specifically prepared loose sand test bed at Blessington, Wicklow and a medium-dense natural deposit at Donabate, Dublin. The loose sand test bed was formed at the Blessington test site, which is an over-consolidated ($q_c > 15$ MPa), fine sand with the water table at 8 m below ground level (bgl), see Gavin and Lehane 2007. The loose sand deposit was created by excavating a 10 m long, 2 m wide and 5.5 to 7 m deep trench in the dense sand deposit. The heavily over-consolidated nature of the deposit allowed the vertical trench to be maintained without support, whilst the excavated material was replaced using the raining deposition technique with a minimum drop-height of 1 m (Fig. 11). Four profiles of the CPT q_c and f_s resistance were taken at 2 m intervals along the trench seven days after formation of the loose sand deposit, with four additional CPT tests performed after a seven month interval. The average cone tip resistance and sleeve friction values are presented in Fig. 12. The traces show that the material is relatively uniform with a CPT q_c value of ≈ 0.8 to 1 MPa below a depth of 2 m. The elevated q_c values in the upper 2 m reflect the influence zone effected by regular trafficking of the CPT truck in this area. The in-situ shear modulus, G_0 ,

determined by Multi-channel Analysis of Surface Waves (MASW), varied from $\approx 20 - 40$ MPa. Also depicted on Fig.12 is the average q_c and f_s values for the Donabate test site. These show variable traces typical of natural deposits where the q_c values typically ranged from 5 to 8 MPa and the f_s values from 0.03 to 0.05 MPa. The low resistances recorded at approximately 2.8 m bgl. were associated with a thin peat layer.

Laboratory testing was used at both test sites to supplement the in-situ site investigation and establish basic soil properties. Particle size distributions conducted on the Blessington sand classified the material as fine to medium grained, with a mean particle size varying from 0.1-0.15mm. The sand at Donabate had a mean particle size in the range 0.212 to 0.3mm. The moisture content measured in the Blessington trench increased with depth from 8% at ground surface to approximately 13% at 4.5m depth. Shear box tests on the Blessington sand yielded a constant volume friction angle of 37° degrees, however tests which utilised a stainless steel interface resulted in a lower interface friction angle of 28 degrees.

Similar installation procedures were adopted at Blessington and Donabate with the UCD open-ended pile installed from the base of a starter hole using the hydraulic rams of a CPT truck at a penetration rate of 20 mm/s. The starter holes were 1.9 m and 0.6 m deep for the Blessington and Donabate tests respectively. Both test piles were installed using 100 mm jacking strokes to push the piles to a final depth of 5.9 m bgl. at Blessington, and 2.6 m bgl. at Donabate. Following pile installation, displacement transducers were set up at the pile head and a series of load tests were performed. Two aspects of the pile

behaviour are discussed herein; specifically the distribution of base and shaft load and scale effects related to dilation at the pile-soil interface.

Separation of Load Components

One of the key aspects of the pile design was to allow accurate measurement of the load distribution, by separating the shaft and base load using the twin walled design. The field measurements of both the internal and external strain gauges were monitored throughout testing at the two sites and are illustrated in Fig. 13. These measurements represent the average of three gauges (at each level), and allow a consistent profile of shaft and base load to be derived. The reliability of the measurements attained at both sites validated the procedures used for installing the strain gauges. At both test sites, the base load was seen to rise during each installation jacking stroke before returning to a positive compressive value upon unloading, reflecting the build up of the residual base stress. In response the shaft was also seen to develop an equal residual negative (tensile) load component. In both tests the residual stress returned to zero following the final tension load tests performed on the piles. This provides confidence in the strain gauge measurements, confirming that no drift occurred during the test period and allowed the residual loads to be accurately quantified. Further separation of the annular and plug load components is possible using the local strain gauge measurements recorded along the pile shaft.

The separation of the annulus and plug load using conventional single-walled strain gauged piles requires extrapolation of the load from the lowermost strain gauge to the pile annulus. A number of techniques have been used to achieve this. Paik et al., 2003,

use a linear extrapolation while Lehane and Gavin, 2001, noting the large increase in internal shear stress, τ_{si} , near the pile tip propose an exponential extrapolation as follows:

$$[4a] \quad \tau_{si} = \beta q_{plug} \exp (-2 \beta h/R_i)$$

$$[4b] \quad \beta = K \tan \delta$$

Where K is the ratio of radial to vertical effective stress within the pile plug and h/R_i is the distance from the tip of pile normalized by the pile internal radius. Fig. 14(a) presents the load breakdown along the inner tube for a jacking stroke at 3m depth during installation and compares the measured annular resistance (from the annular pressure sensor) with that determined from the strain gauges using the extrapolation methods adopted by Paik et al., 2003 and Lehane and Gavin, 2001. The exponential extrapolation adopted by Lehane and Gavin 2001 (Eqn. [4]) is shown using recommended lower- and upper-bound β values of 0.25 and 0.45 respectively. The annular load is determined by subtracting the calculated shear force below the lowermost strain gauge from the gauge value. Fig. 14(b) shows the extrapolated and measured annular resistance throughout installation. It is evident that both the linear and exponential extrapolation approaches do not provide an accurate assessment of the annular resistance and that a large amount of load transfer occurs over a very small length at the bottom of the pile plug (0.3D). However, assuming $q_{ann} = q_c$ as suggested by Gavin and Lehane, 2003, provides an excellent prediction throughout installation. The authors suggest that if an annular stress sensor is not incorporated in the pile design, the q_{ann} should be taken as equal to the CPT

q_c rather than extrapolating from strain gauges. This would provide a more accurate estimate of both the annular and plug load .

Scale Effects

The horizontal effective stresses acting on the pile wall during installation of the UCD-OE pile at Blessington are shown in Fig. 15a, for sensors located at $h/D = 1.5$ (black lines) and 5.5 (grey lines). The peak values (solid line) represent the radial effective stress at failure, σ'_{rf} , while the stationary values (dashed line) represent the equalised radial effective stresses, σ'_{re} . The peak radial effective stresses are approximately two to three times the stationary values throughout installation. As a result approximately 50–70 % of the shaft capacity is provided by dilation which would not be present in larger diameter full scale piles. These measurements of radial stress are a unique feature of the UCD-OE twin-walled pile, with previous research using strain gauges in isolation to measure the load break down (eg. Paik et al, 2003 and Gavin and Lehane 2003). The results of these tests using piles instrumented exclusively with strain gauges, provided a noted contribution to the understanding of pile behaviour. However, because of the contribution from dilation to the shaft resistance mobilised, it is difficult to extrapolate this data to full-scale pile geometries. For reliable application to offshore design, the dilation component must be measured using radial stress transducers.

Fig. 15b shows a comparison between the measured interface dilation and that predicted by Eqn. [3], using the measured in-situ shear stiffness, G_0 from the MASW data and a

surface roughness values, R_a , of $1\mu\text{m}$ chosen to match the stainless steel pile roughness . The predicted $\Delta\sigma'_{rd}$ values significantly underestimated the measured values, despite the use of a high shear modulus, equal to the small-strain stiffness (G_0). A similar underestimate of the $\Delta\sigma'_{rd}$ values measured at the sensor at $h/D = 1.5$ on the pile tested in the medium-dense sand at Donabate is noted in Figure 16. These results highlight the importance of measuring the radial effective stresses on model piles, as attempts to estimate the effect of interface dilation using methods such as Eqn. [3] could result in an over-prediction of the shaft capacity developed by full-scale piles in similar ground conditions.

Conclusions

This paper described the design, construction and calibration of a model instrumented open-ended pile which was developed to allow for multiple installations in clay and loose to medium-dense sand deposits. The model pile has a slenderness ratio (L/D) and ratio of pile diameter to wall thickness (D/t) typical of full-scale piles used offshore. The importance of using a twin-walled form of construction, providing annular base stress sensors and measuring horizontal effective stresses at multiple locations on the pile shaft were discussed. Details of the choice of instrumentation and the construction issues which arose due to incorporation of sensitive instruments into the model are presented. Laboratory calibration procedures which ensured reliable and repeatable results from the instrumentation were described.

A detailed field calibration exercise was discussed for a loose sand deposit at Blessington and medium-dense sand at Donabate. Both tests demonstrated accurate separation of the shaft and base loads using the twin walled design. The tests highlighted the need for a pressure sensor located at the pile annulus to specifically measure the annular resistance in order to accurately separate the shaft and base load components. The importance of measuring effective horizontal stresses was also illustrated through the difference in the dilatational stress measured at the shaft and that predicted using conventional formulae. Failure to measure the effective horizontal stresses would lead to a significant over-prediction of shaft capacity when extrapolating to larger diameter full scale piles.

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Figures:

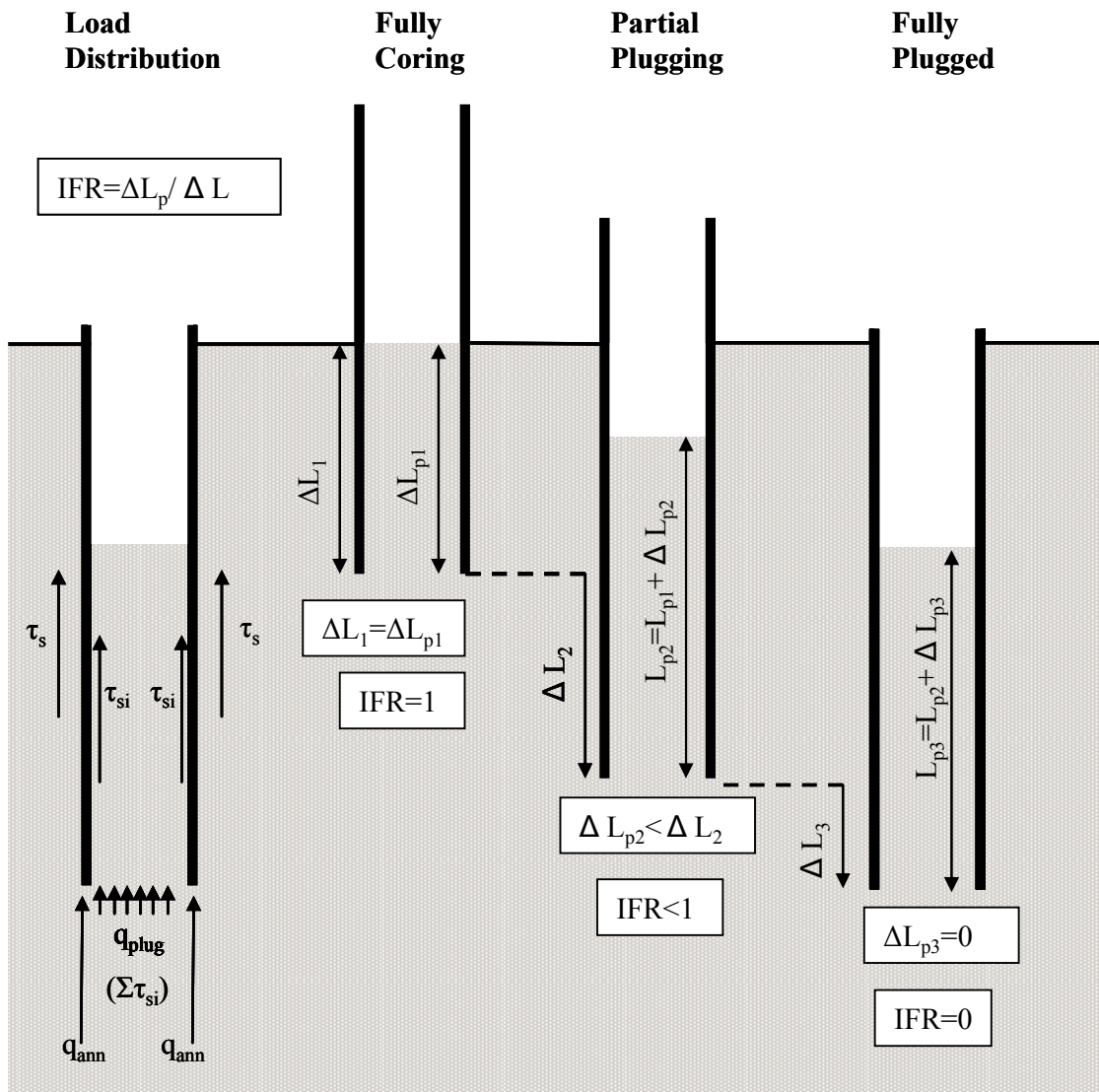


Fig.1: The Load distribution and modes of penetration of an Open-Ended Pile

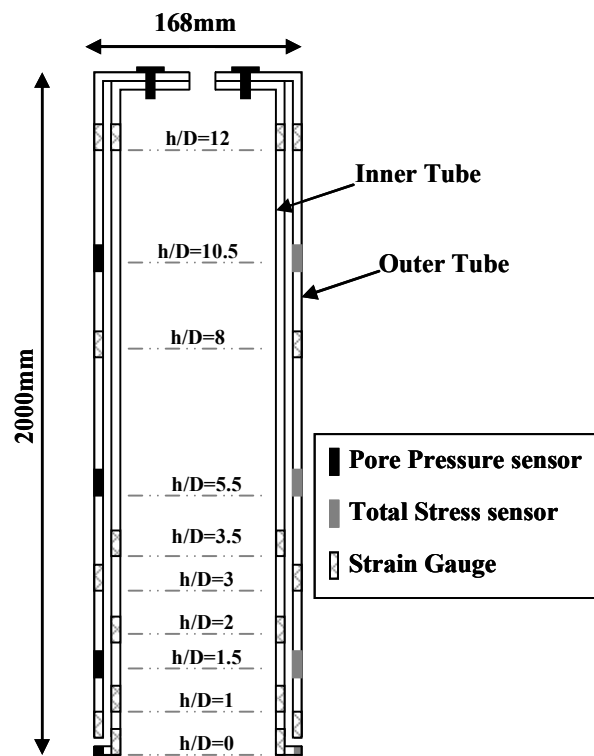


Fig. 2: Pile Construction and Instrumentation Layout (after Doherty et al. 2009)

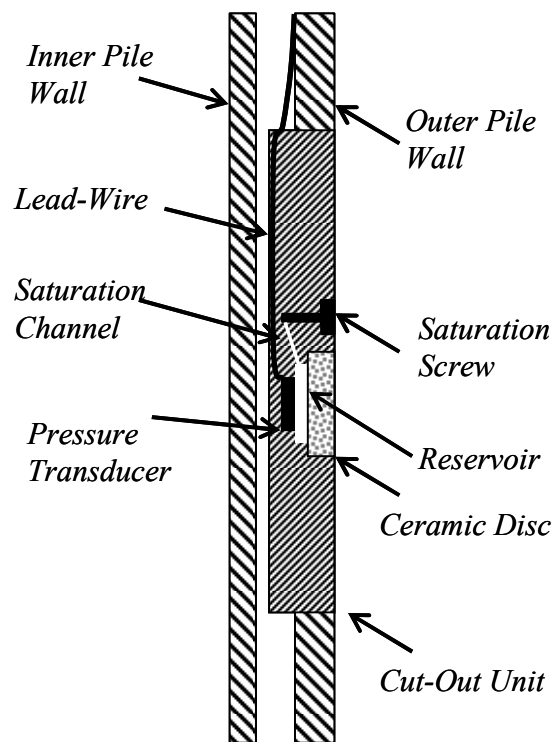


Fig. 3: Pore Pressure system adopted for the UCD-OE Pile

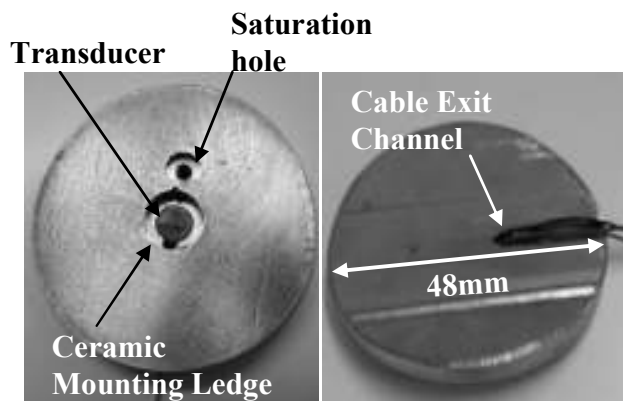


Fig. 4: Front and Back of Pore Pressure Cut out units

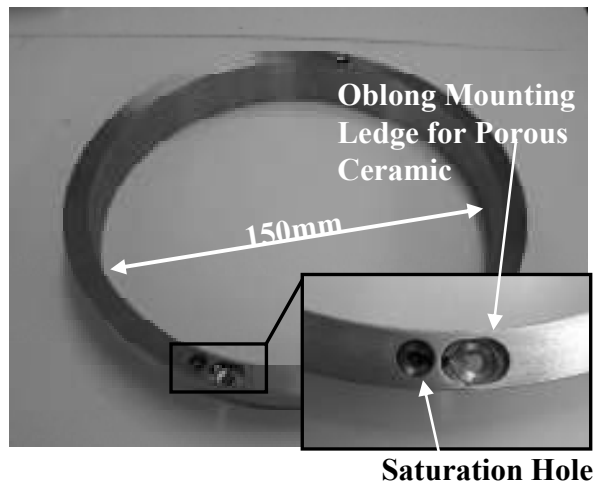


Fig. 5: Annular Base unit machined to accommodate total and pore pressure sensors

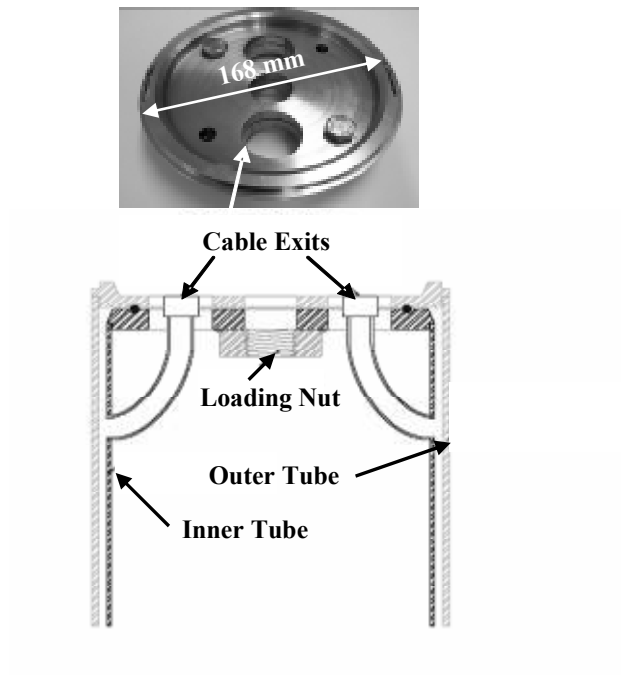


Fig. 6: Detail of Pile Cap Construction

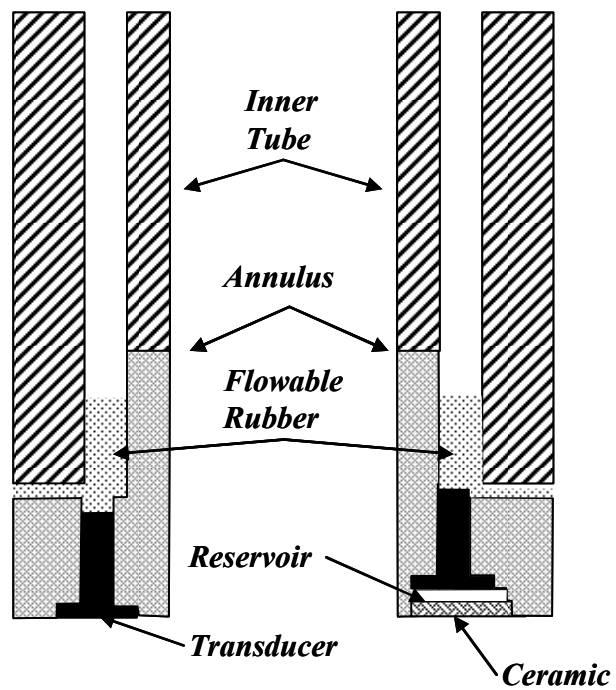


Fig. 7: Pile Base Detail including Sensor Detail and Moisture Seal

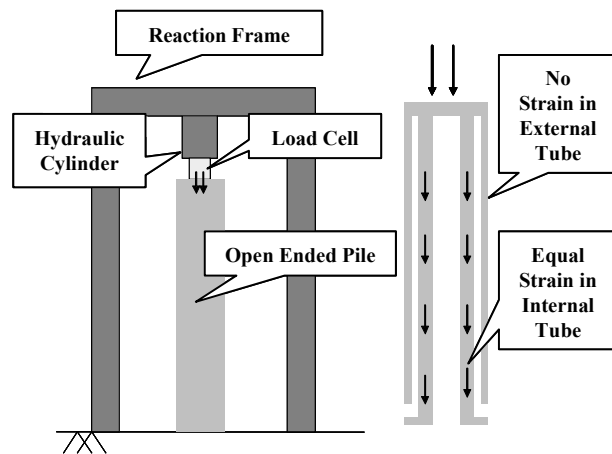


Fig. 8: Checking Performance of Strain Gauges and Validation of twin wall design

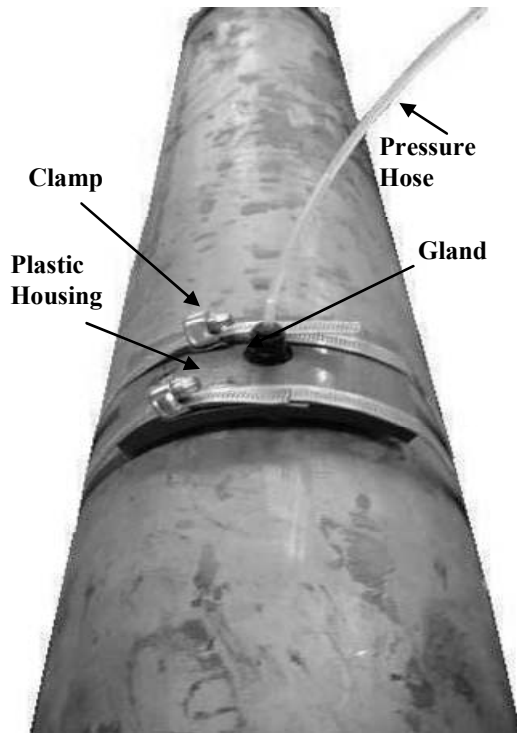


Fig. 9: Pressure Sensor Calibration System

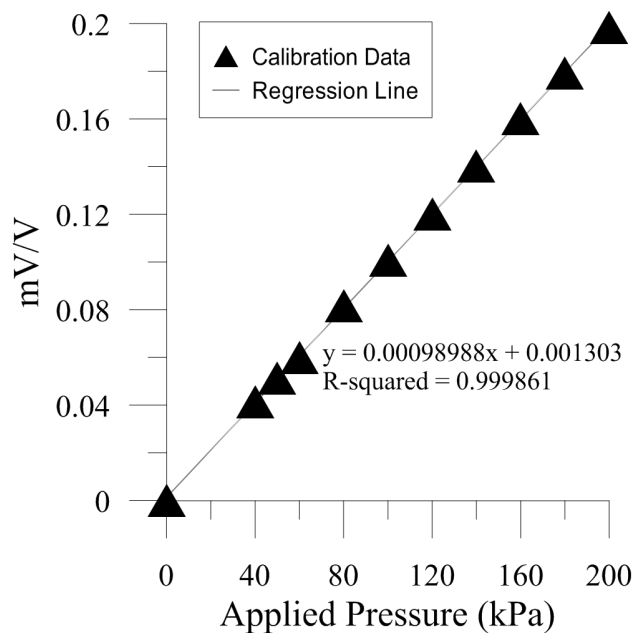


Fig. 10: Calibration data of Total Stress Transducer at h/D of 1.5



Fig. 11 Excavating and Backfilling the Trench at Blessington

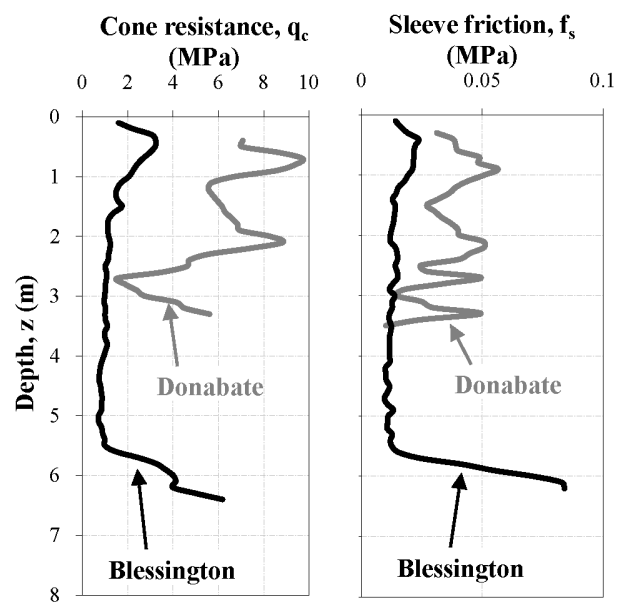


Fig. 12: CPT End Bearing q_c , and Sleeve Friction f_s Profiles from Blessington and Donnabate

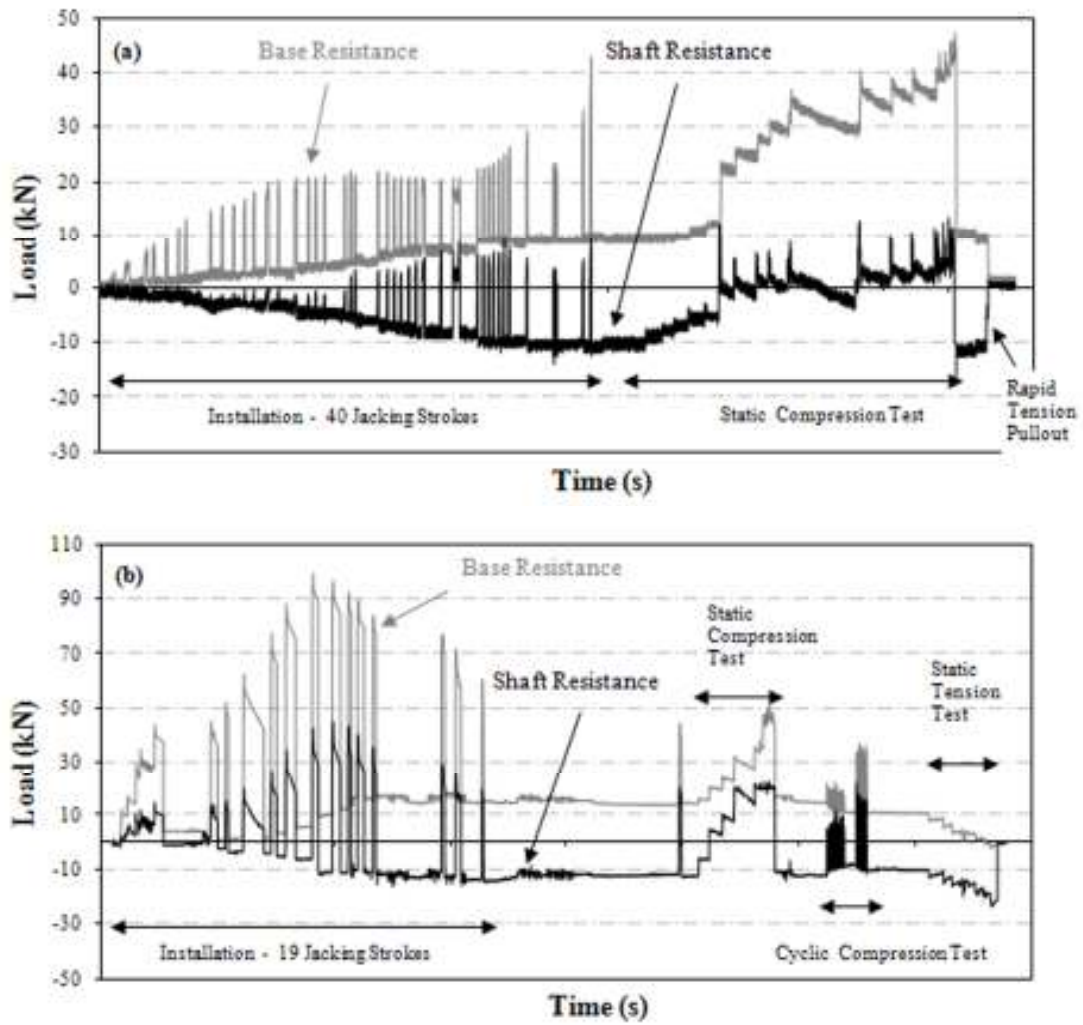


Figure 13: Time History showing separation of Shaft and Base Load during pile tests at
(a) Blessington and (b) Donnabate

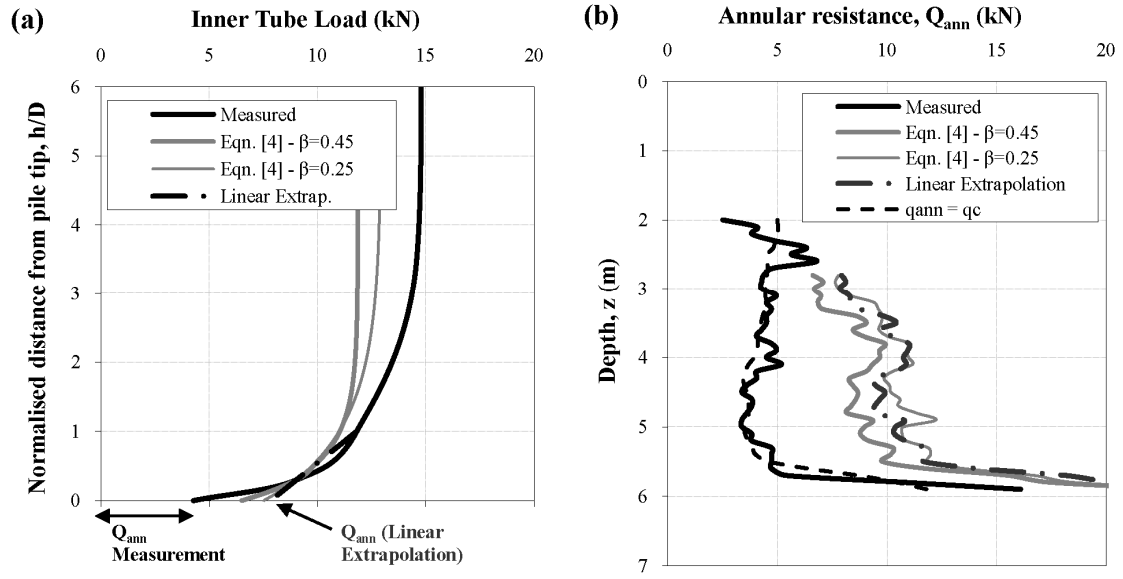


Fig. 14 Measured and Predicted Annular Load Extrapolation (a) for Jacking Stroke at 3m Depth and (b) during Entire Installation

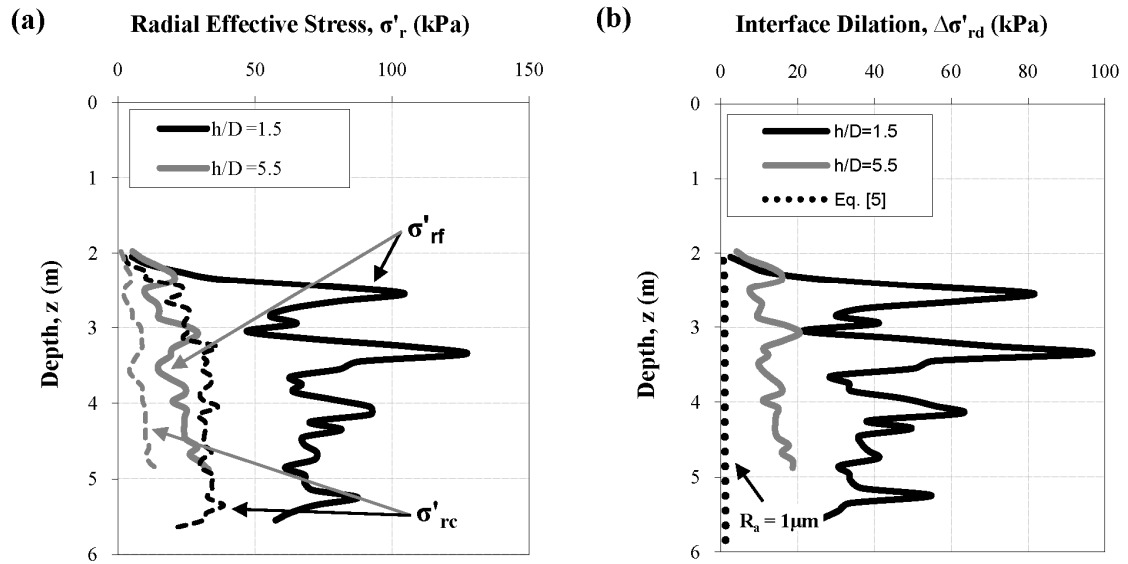


Fig. 15 (a) Radial Effective Stresses and (b) Dilation during Installation of Blessington Pile

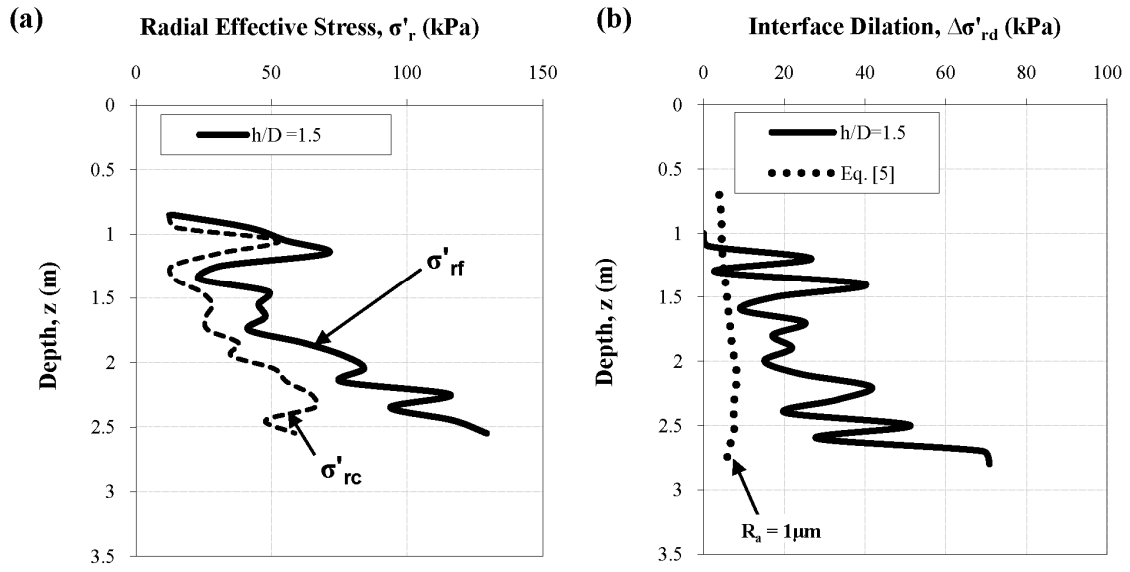


Fig. 16 (a) Radial Effective Stresses and (b) Dilation during Installation of Donnabate Pile