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EXPERIMENTAL INVESTIGATIONS OF DRIVEN CAST-IN-SITU PILES

by

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A thesis submitted to the College of Engineering & Informatics, National University of Ireland, Galway, in partial fulfilment of the requirements for the Degree of Doctor of Philosophy

August 2014

Academic Supervisor: Dr. Bryan McCabe
Professor of Civil Engineering: Prof. Padraic O’Donoghue
DECLARATION

I hereby declare that this thesis has not been submitted in whole or in part to any other University as an exercise for a degree. I further declare that, except where reference is given in the text, it is entirely my own work.

This thesis may be lent or copied by the library.

Signed: ________________________ Date: _____________

Kevin N. Flynn
In memory of my late grandfather Noel
ACKNOWLEDGEMENTS

First and foremost, I would like thank Dr. Bryan McCabe for not only being a great supervisor over the past 4 years, but also a great friend. Bryan always had time to chat to me about all-things geotechnical, or otherwise, when I came knocking at his door and has provided endless feedback and encouragement for both my research (in particular when writing this thesis) and my accent impersonations. I’m also extremely grateful to him for taking time out of his busy schedule to help out with the installation of the test piles at Ryton in October 2013. Thanks for everything Bryan!

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ABSTRACT

The prediction of displacement pile capacity in sand is hampered by the extreme changes in stress which occur in the immediate vicinity of the pile during installation (Randolph 2003). High quality instrumented tests on steel model piles, such as those reported by Lehane (1992) and Chow (1997), have helped identify other factors that have a bearing on preformed displacement pile behaviour in sand. These factors include the extent of soil displacement during installation and loading, the reduction in shaft friction due to increasing load cycles during installation (referred to as friction fatigue), increases in radial stresses due to dilation at the pile-soil interface, differences in shaft resistance with loading direction (i.e. compressive and tensile loading) and increases in shaft capacity with time, i.e. pile ageing. The majority of these phenomena have now been incorporated in four new cone penetration test (CPT)-based design methods which give superior estimates of preformed displacement pile capacity in sand in comparison to traditional methods.

The knowledge gained from the high-quality studies of displacement pile behaviour is now being applied to other pile types such as partial displacement piles (e.g. open-ended piles) and replacement piles (e.g. bored and screw piles). One category of pile which has received sparse attention is the driven cast-in-situ (DCIS) pile which is typically classified as a large displacement pile, despite sharing certain aspects of its construction with replacement pile types. Furthermore, there are relatively few case histories of load tests on DCIS piles in the literature to verify the assumption that they behave as full displacement piles.

The behaviour of DCIS piles during installation, curing and maintained load testing was therefore investigated by constructing a total of seven instrumented DCIS piles in layered soils and sand at sites in the United Kingdom. The resistance of the steel installation tube during driving was derived using instrumentation fitted to the DCIS piling rigs. The variation in temperature and strain after casting was monitored continuously in three of the test piles to examine the development of residual loads during curing. After developing sufficient concrete strength, the test piles were subjected to maintained compression load tests to failure (i.e. a displacement in excess of 10% of the pile diameter), with the shaft and base resistance during loading derived from strain measured within the test piles using vibrating wire strain gauges.
The installation resistance derived by the rig instrumentation shows good agreement with the base resistance profile derived by the University of Western Australia UWA-05 method using the Dutch averaging technique. Residual loads developed during curing of the DCIS piles installed in layered soils due to consolidation settlement of soft soil layers as pore pressures induced by the driving process dissipated. On the other hand, residual loads for DCIS piles in uniform sand were negligible.

The instrumented DCIS piles in sand exhibited a clear reduction in normalised local shear stresses and radial effective stresses at failure with distance from the pile base, i.e. friction fatigue, which is a well-known characteristic of preformed displacement piles and its existence for DCIS piles implies that radial stresses during driven installation of the steel tube are not erased upon concreting and tube withdrawal. The normalised base resistance at failure showed excellent agreement with the UWA-05 design method for driven closed-ended displacement piles, with the onset of degradation in base stiffness occurring at large base displacements. Design correlations have subsequently been developed based on the results of the instrumented DCIS pile tests.

The main implication of the experimental data for DCIS pile design in sand is that the shaft, base and total capacities of a DCIS pile are similar to a preformed closed-ended displacement pile of equivalent dimensions. In keeping with this finding, an examination of the predictive performance of seven CPT-based displacement pile design methods using a database of 26 DCIS pile load tests with adjacent CPT $q_c$ profiles demonstrated that the recent methods provide improved estimates of DCIS shaft, base and total capacity in comparison to traditional simplified methods. However, a statistical study of DCIS pile load-displacement behaviour in sand using a database of 105 pile load tests showed that the total resistance of a DCIS pile tends to mobilise at a slower rate in comparison to preformed driven displacement piles, implying that DCIS piles may exhibit greater levels of displacement for a given applied load.
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NOMENCLATURE

Abbreviations:

ACIP        Auger Cast-In-Place
AOD         Above Ordnance Datum
CAPWAP      CAse Pile Wave Analysis Programme
CFA         Continuous Flight Auger
COV         Coefficient of Variation
CPT         Cone Penetration Test
CTE         Coefficient of Thermal Expansion
CTRL        Channel Tunnel Rail Link
DCIS        Driven Cast-In-Situ
DVL         Design Verification Load
EOD         End of Driving
FOS         Factor of Safety
GF          Gauge factor
ICE         Institution of Civil Engineers
ICP         Imperial College Pile
LPDT        Linear Potentiometer Displacement Transducer
PDA         Pile Driving Analyser
PIV         Particle Image Velocimetry
SCPT        Seismic Cone Penetration Test
SPERW       Specification for Piling and Embedded Retaining Walls
SPT         Standard Penetration Test
SST         Surface Stress Transducer
SWL         Specified Working Load
VWSG        Vibrating Wire Strain Gauge

Symbols:

\( a \)        Normalised hyperbolic curve-fitting parameter (mm)
\( a_{rig} \)  Distance between bottom hammer velocity sensor and point of impact (mm)
\( a' \)       Hyperbolic curve-fitting parameter
\( \overline{a} \) Sample mean for normalised hyperbolic curve-fitting parameter \( a \) (mm)
\( b \)        Normalised hyperbolic curve-fitting parameter (mm)
\( b' \)       Hyperbolic curve-fitting parameter
\( \overline{b} \) Sample mean for normalised hyperbolic curve-fitting parameter \( b \) (mm)
\( c_e \)      Tube elastic compression (mm)
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
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</thead>
<tbody>
<tr>
<td>$c_{rig}$</td>
<td>Distance between hammer velocity sensors (= 50 mm)</td>
</tr>
<tr>
<td>$\frac{d\varepsilon}{dt}$</td>
<td>Strain rate (%)</td>
</tr>
<tr>
<td>$\frac{d\varepsilon_{ref}}{dt}$</td>
<td>Reference strain rate (%)</td>
</tr>
<tr>
<td>$e$</td>
<td>Void ratio</td>
</tr>
<tr>
<td>$e_{\text{transfer}}$</td>
<td>Energy transfer efficiency</td>
</tr>
<tr>
<td>$f$</td>
<td>Empirical parameter for concrete stiffness</td>
</tr>
<tr>
<td>$f$</td>
<td>Stiffness curve-fitting parameter</td>
</tr>
<tr>
<td>$f_c$</td>
<td>Compression loading coefficient</td>
</tr>
<tr>
<td>$f_{ck}$</td>
<td>Concrete cylinder compression strength (MPa)</td>
</tr>
<tr>
<td>$f_{cu}$</td>
<td>Concrete cube compressive strength (MPa)</td>
</tr>
<tr>
<td>$f_{\text{current}}$</td>
<td>Current frequency (Hz)</td>
</tr>
<tr>
<td>$f_{\text{initial}}$</td>
<td>Initial frequency (Hz)</td>
</tr>
<tr>
<td>$f_t$</td>
<td>Tension loading coefficient</td>
</tr>
<tr>
<td>$g$</td>
<td>Acceleration due to gravity (= 9.81 m/s$^2$)</td>
</tr>
<tr>
<td>$g$</td>
<td>Stiffness curve-fitting parameter</td>
</tr>
<tr>
<td>$h$</td>
<td>Distance from base (mm)</td>
</tr>
<tr>
<td>$h_{\text{hammer}}$</td>
<td>Hammer drop height (mm)</td>
</tr>
<tr>
<td>$k$</td>
<td>Initial base stiffness (MPa)</td>
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<td>$k_n$</td>
<td>Equivalent normal stiffness (MPa)</td>
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<tr>
<td>$m$</td>
<td>Hammer mass (kg)</td>
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<tr>
<td>$m$</td>
<td>Sample mean</td>
</tr>
<tr>
<td>$n$</td>
<td>Sample size</td>
</tr>
<tr>
<td>$n$</td>
<td>Parabolic base stress-displacement response coefficient</td>
</tr>
<tr>
<td>$p_{\text{ref}}$</td>
<td>Atmospheric stress (= 100 kPa)</td>
</tr>
<tr>
<td>$p'$</td>
<td>Mean effective stress (kPa)</td>
</tr>
<tr>
<td>$p'_0$</td>
<td>Initial mean effective stress (kPa)</td>
</tr>
<tr>
<td>$p'_f$</td>
<td>Mean effective stress at failure (kPa)</td>
</tr>
<tr>
<td>$q_b$</td>
<td>Base resistance (MPa)</td>
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<tr>
<td>$q_{b,\text{lim}}$</td>
<td>Limiting base resistance (MPa)</td>
</tr>
<tr>
<td>$q_{b,\text{res}}$</td>
<td>Residual base resistance (MPa)</td>
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<td>$q_{b,s}$</td>
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<td>$q_{b,\text{ult}}$</td>
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<td>Cone resistance (MPa)</td>
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<td>Cone resistance at base (MPa)</td>
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<td>$q_{c1N}$</td>
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Hammer velocity sensor
Hammer impact location
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Cone penetrometer diameter (mm)
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Relative density at base (%)  
Relative density along shaft (%)  
Shaft diameter (mm)
Mean particle size (mm)
Linear equivalent secant base stiffness (MPa)
Concrete stiffness (MPa)
Hammer energy (kJ)
Initial stiffness (MPa)
Pile stiffness (MPa)
$E_{\text{steel}}$  Steel tube stiffness (MPa)
$E_0$  Small-strain stiffness (MPa)
$F_{\text{Dr}}$  Relative density coefficient
$F_{\text{load}}$  Load direction coefficient
$F_{\text{mat}}$  Material coefficient
$F_{r}$  Normalised friction ratio
$F_{\text{sig}}$  Overburden stress coefficient
$F_{\text{tip}}$  Base coefficient
$G$  Shear stiffness (MPa)
$G_0$  Small-strain shear stiffness (MPa)
$G_{0,\text{SCPT}}$  Small-strain shear stiffness derived from seismic CPT (MPa)
$H$  Distance from penetrometer tip to layer interface (m)
$I_r$  Rigidity index
$K_f$  Coefficient of lateral earth pressure at failure
$L$  Pile length (m)
$L_i$  Pile section length (m)
$N$  Number of cycles during installation and/or loading
$N_{k}$  Undrained shear strength cone factor
$N_q$  Bearing capacity factor
$N_{q,0.1Db}$  Bearing capacity factor at displacement equivalent to 10% $D_b$
$N_{\text{SPT}}$  SPT blowcount (blows/300 mm)
$P_i$  Load at section $i$ (kN)
$Q$  Applied load (kN)
$Q_b$  Base load (kN)
$Q_{\text{b,c}}$  Calculated base capacity (kN)
$Q_{\text{b,m}}$  Measured base capacity (kN)
$Q_c$  Calculated pile capacity (kN)
$Q_{\text{head}}$  Load at pile head (kN)
$Q_m$  Measured total capacity (kN)
$Q_s$  Shaft capacity (kN)
$Q_{\text{s,i}}$  Shaft resistance at section $i$ (kN)
$Q_{\text{st}}$  Slope tangent capacity (kN)
$Q_{\text{s,c}}$  Calculated shaft capacity (kN)
$Q_{\text{s,m}}$  Measured shaft capacity (kN)
$Q_{\text{s,total}}$  Overall shaft capacity (kN)
$Q_{t}$  Normalised cone resistance
$Q_{t}$  Total capacity (kN)
$Q_{t,0.1Db}$  Total capacity at displacement equivalent to 10% $D_b$ (kN)
$R$  Pile radius (m)
$R^2$  Coefficient of determination
$R_a$  Average roughness (μm)
$R_f$  Friction ratio (%)
\( R_n \) Normalised roughness
\( R_{n,crit} \) Critical normalised roughness
\( T_0 \) Temperature at time of casting (hours)
\( T_1 \) Current hydration temperature after casting (\(^\circ\)C)
\( T_{50\%} \) Time from peak temperature to 50% of excess temperature (hours)
\( T_{peak} \) Time to peak temperature after casting (hours)
\( V_s \) Shear wave velocity (m/s)
\( X_1 \) Randomly-generated parameter for \( a \)
\( X_2 \) Randomly-generated parameter for \( b \)
\( Z_a \) Standard normal random variable for parameter \( a \)
\( Z_b \) Standard normal random variable for parameter \( b \)
\( Z_s \) Pile embedment in strong layer (m)
\( Z_w \) Pile embedment in weak layer (m)
\( \alpha \) Coefficient of thermal expansion (\( \mu e/\circ\)C)
\( \alpha_b \) Base resistance coefficient
\( \alpha_s \) Shaft resistance coefficient
\( \gamma_b \) Bulk unit weight (kN/m\(^3\))
\( \delta \) Interface friction angle (\(^\circ\))
\( \delta_{cv} \) Constant-volume interface friction angle (\(^\circ\))
\( \delta_{f} \) Interface friction angle at failure (\(^\circ\))
\( \delta_{peak} \) Peak interface friction angle (\(^\circ\))
\( \Delta Q_s \) Change in shaft resistance (kN)
\( \Delta T \) Change in temperature during hydration (\(^\circ\)C)
\( \Delta \mu \) Change in pore pressure (kPa)
\( \Delta y \) Radial displacement due to dilation/contraction (mm)
\( \Delta \mu e \) Change in strain
\( \Delta \mu e_{residual} \) Change in strain due to residual load
\( \Delta \sigma'_r \) Change in radial effective stress (kPa)
\( \Delta \sigma'_{rd} \) Change in radial effective stress due to dilation (kPa)
\( \varepsilon \) Strain
\( \varepsilon_b \) Base resistance scaling coefficient
\( \varepsilon_{elastic} \) Creep-corrected elastic strain
\( \varepsilon_{ref} \) Reference strain
\( \varepsilon_{thermal} \) Thermal strain during hydration
\( \eta_{min} \) Ratio of steady-state penetration resistance in weak and strong layers
\( \lambda \) Base resistance reduction factor for soil relaxation
\( \lambda_a \) Equivalent standard deviation for lognormally-distributed parameter \( a \)
\( \lambda_b \) Equivalent standard deviation for lognormally-distributed parameter \( b \)
\( \mu \) Interface friction coefficient (= tan\( \delta \))
\( \mu \) Population mean
\( \xi_a \) Equivalent mean for lognormally-distributed parameter \( a \)
\( \xi_b \) Equivalent mean for lognormally-distributed parameter \( b \)
\(\xi\) Installation and shaft roughness coefficient

\(\rho\) Soil density (kg/m³)

\(\rho_{a,b}\) Pearson correlation coefficient for hyperbolic parameters \(a, b\)

\(\rho_{XaXb}\) Pearson correlation coefficient for parameters \(X_a, X_b\)

\(\sigma\) Population standard deviation

\(\sigma\) Stress (kPa)

\(\sigma'_{h0}\) Initial horizontal effective stress (kPa)

\(\sigma'_{hp}\) Peak horizontal effective stress (kPa)

\(\sigma'_{hs}\) Stationary horizontal effective stress (kPa)

\(\sigma'_{r}\) Radial effective stress (kPa)

\(\sigma'_{rc}\) Radial effective stress after installation and equalisation (kPa)

\(\sigma'_{rf}\) Radial effective stress at failure (kPa)

\(\sigma'_{rp}\) Peak radial effective stress (kPa)

\(\sigma'_{rs}\) Stationary radial effective stress (kPa)

\(\sigma'_{v0}\) Initial vertical stress (kPa)

\(\sigma'_{v0}\) Initial vertical effective stress (kPa)

\(\tau_s\) Shear stress (kPa)

\(\tau_{sf}\) Local shear at failure (kPa)

\(\tau_{s,avg}\) Average shear stress (kPa)

\(\tau_{s,lim}\) Limiting shear stress (kPa)

\(\tau_{s,loc}\) Local shear stress (kPa)

\(\tau_{s,max}\) Maximum local shear stress (kPa)

\(\tau_{s,min}\) Minimum local shear stress (kPa)

\(\tau_{s,res}\) Residual shear stress (kPa)

\(\nu\) Poissons' ratio

\(\phi'\) Soil friction angle (°)

\(\phi'_{cv}\) Constant-volume soil friction angle (°)
CHAPTER 1  INTRODUCTION
1.1 BACKGROUND

Piled foundations are typically used to transfer large structural loads into underlying competent strata. The use of displacement piles has evolved considerably over the past 4000 years or so, with simple timber stakes being used in pre-historic times to support settlements in flood plain deposits adjacent to lakes and rivers, before eventually being superseded by preformed concrete and steel displacement piles in the late 19th century which were dynamically driven into the soil by mechanical means. Replacement piles have proven increasingly popular in modern times, particularly with the advent of stringent environmental legislation preventing the use of driven displacement piles in urban areas due to excessive noise levels, although the costs associated with the disposal of spoil generated by replacement piling techniques may curtail such use, particularly at sites where soil contamination is an issue. The rapid growth of the renewable energy industry has also led to the use of large-diameter open-ended steel monopiles for offshore wind turbines.

The design of piled foundations remains highly empirical however, with the prediction of displacement pile capacity in siliceous sand arguably the most uncertain area of foundation design due to the extreme levels of soil distortion during driven installation (Randolph, 2003). Despite such uncertainties, major advances in the knowledge of the mechanisms governing preformed displacement pile behaviour in sand have been achieved over the past 25 years or so through a series of high-quality field-scale tests using heavily-instrumented closed-ended steel piles such as those reported by Lehane (1992) and Chow (1997). These tests have helped identify factors such as the extent of soil displacement during installation and loading, the reduction in shaft friction due to increasing load cycles during installation (referred to as friction fatigue), increases in radial stresses due to dilation at the pile-soil interface, differences in shaft resistance with loading direction (i.e. compressive and tensile loading) and increases in shaft capacity with time, i.e. pile ageing. The majority of these phenomena have now been incorporated in recent CPT-based design methods which have been shown to provide superior estimates of preformed displacement pile capacity in comparison to traditional design approaches based on earth pressure theory.

The knowledge gained from the high-quality instrumented tests on preformed displacement piles has inevitably led to increased focus on the behaviour of cast-in-situ replacement piles. The absence of high-quality studies involving cast-in-situ piles is also attributed to processes synonymous with concrete such as in-situ curing, shrinkage, creep, cracking and strain-dependent stiffness; these processes may have a significant influence on the performance of
pile instrumentation and the subsequent interpretation of pile behaviour. It is therefore unsurprising that design methods for cast-in-situ piles have tended to remain rather simplistic (Lehane, 2008). Driven cast-in-situ (DCIS) piles, on the other hand, have received relatively little attention in the literature, despite having the ability to readily adjust pile lengths to suit the depth of penetration required in comparison to preformed piles. While aspects of the DCIS pile construction process are shared with replacement pile types (i.e. casting of concrete in-situ and the resulting rough shaft interface), the use of driving to install DCIS piles has led to their classification as large displacement piles (BSI, 1986) and practitioners have consequently tended to resort to traditional displacement pile design methods when estimating the shaft and base capacity of DCIS piles in sand. Given the limited number case histories of axial load tests on instrumented DCIS piles however, the assumption that DCIS piles behave in a similar manner to preformed displacement piles (e.g. precast concrete and steel piles) remains to be verified.

1.2 DRIVEN CAST-IN-SITU PILES
The installation process of a temporary-cased DCIS pile, illustrated in Figure 1.1, involves top-driving a hollow steel tube using a pile driving hammer, with a sacrificial circular steel plate placed at the base of the tube in order to prevent ingress of soil and water during driving. When the required depth of penetration is reached, the reinforcement cage is inserted into the tube which is then filled with high-slump concrete using skipping or pumped methods. The hammer is then reattached and the tube is extracted. During removal, a number of blows are applied to the tube in order to compact the concrete. The pile is then left to cure in-situ for a number of days, with the steel plate remaining at the base.

Figure 1.1 - DCIS pile construction process (courtesy of Keller Foundations)
1.3 AIMS AND OBJECTIVES

The primary aim of this thesis is to investigate the behaviour of DCIS piles during installation, curing and maintained load testing in layered soil and uniform sand. In order to achieve this aim, the following objectives have been identified:

1. Examine the relationship between the installation resistance derived from the DCIS piling rig instrumentation and the base resistance profile predicted by several CPT-based design methods using CPT $q_c$ averaging techniques.
2. Investigate the development of residual stresses during curing.
3. Determine the behaviour of DCIS piles, in terms of shaft and base resistance, during maintained compression loading.
4. Investigate the ability of current CPT-based design methods for estimating the shaft, base and total capacity of DCIS piles in sand.
5. Examine and compare the load-displacement behaviour of DCIS piles in sand with traditional preformed closed-ended displacement piles by statistical means.

1.4 METHODOLOGY

In order to investigate the axial load behaviour of DCIS piles, the installation, curing and maintained load test phases were studied on a total of 7 instrumented DCIS piles founded in layered and uniform sand deposits at a number of sites in the United Kingdom. The location and details of each instrumented DCIS pile test is outlined in Table 1.1. It is important to note that, with the exception of Ryton-on-Dunsmore, the tests were performed at sites where contract piling works were undertaken by Keller Foundations and were therefore subjected to some constraints regarding the location and duration of testing.

The installation data for each pile was measured using instrumentation on the piling rig. The development of residual loads during curing was investigated using vibrating wire strain gauges installed in the test piles immediately after casting (continuous measurements of strain and temperature during the curing period were obtained for three of the instrumented test piles using a datalogger). After developing sufficient concrete strength, the test piles were subjected to maintained compression load tests to failure (i.e. a total pile displacement in excess of 10 % of the pile diameter), with the shaft and base resistance during loading derived from strain measurements provided by the strain gauges. Cone penetration tests (CPT)
conducted adjacent to the test pile locations enabled comparisons of normalised shaft and base behaviour with the results of high-quality tests on preformed displacement piles reported in the literature. The measured shaft, base and total DCIS pile capacities were also compared with the capacities predicted by several CPT-based displacement design methods. Finally, a database of over 100 maintained load tests was compiled to enable a comparison of the load-displacement behaviour of DCIS piles in sand with those reported for preformed driven displacement piles using a simple statistical simulation technique.

### Table 1.1 - Instrumented DCIS pile test programme

<table>
<thead>
<tr>
<th>Test location</th>
<th>Soil type</th>
<th>Date</th>
<th>Pile ref.</th>
<th>Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pontarddulais, Wales</td>
<td>Soft CLAY/loose SAND</td>
<td>Nov. 2010</td>
<td>P1</td>
<td>5.2</td>
</tr>
<tr>
<td>Shotton, Wales</td>
<td>Medium dense to dense SAND</td>
<td>May 2011</td>
<td>S1</td>
<td>6.2</td>
</tr>
<tr>
<td>Dagenham, England</td>
<td>Medium dense SAND, soft CLAY/PEAT, very dense sandy GRAVEL</td>
<td>Sep. 2011</td>
<td>D1</td>
<td>5.3</td>
</tr>
<tr>
<td>Erith, England</td>
<td>Soft CLAY/PEAT, very dense sandy GRAVEL</td>
<td>Nov. 2012</td>
<td>E3</td>
<td>5.4</td>
</tr>
<tr>
<td>Ryton-on-Dunsmore, England</td>
<td>Medium dense SAND</td>
<td>Oct.-Nov. 2013</td>
<td>R1, R2, R3</td>
<td>6.3</td>
</tr>
</tbody>
</table>

### 1.5 OUTLINE OF THESIS

A detailed review of closed-ended displacement pile behaviour in sand, in particular the results of a series of tests conducted by Imperial College London using heavily-instrumented closed-ended steel model piles, is presented in Chapter 2. The behaviour of the shaft and base of displacement piles are reviewed separately, with facets of behaviour relevant to driven cast-in-situ piles highlighted throughout. A summary of the various theoretical and empirically-based displacement pile design methods in sand is subsequently provided and their reliability in predicting pile capacity, as established in previous studies, is discussed. Finally, a detailed review of residual load development in cast-in-situ piles is presented.

The various experimental methods and procedures performed during the instrumented DCIS pile test programme in Table 1.1 are described in Chapter 3. The first section of the chapter describes the equipment and instrumentation used during DCIS pile installation, the curing phase and maintained compression load testing, as well as the various strain interpretation methods for assessing residual loads during curing of cast-in-situ piles, followed by the
procedures used to interpret the measured strains within the instrumented DCIS test piles during maintained compression load testing.

The ground conditions at the three layered sites (Pontarddulais, Dagenham and Erith) and two sand sites (Shotton and Ryton-on-Dunsmore) in the United Kingdom in which the instrumented DCIS pile tests were performed are summarised in Chapter 4. The location and geological history of each site is initially presented, followed by the results of in-situ and laboratory tests (where applicable). Parameters relevant to pile behaviour are also discussed.

In Chapter 5, the results of a series of tests conducted on instrumented DCIS piles at Pontarddulais, Dagenham and Erith in the United Kingdom are presented. These piles are grouped together as all three rely heavily upon a socket in granular soil for their capacity. General details of each test pile are summarised, followed by the results of the installation, curing and compressive maintained load test phases, and consideration of shaft and base behaviour.

The results of a series of tests on instrumented DCIS piles installed in uniform sand sites at Shotton and Ryton-on-Dunsmore in the United Kingdom are described in Chapter 6. Both sites provided an ideal opportunity to assess the behaviour of DCIS piles in sand, in particular the variation in local shear stress with normalised distance from the pile tip, i.e. friction fatigue.

Chapter 7 provides a discussion of the overall behaviour of DCIS piles in sand based on the results of the instrumented tests presented in Chapters 5 and 6, with reference to other relevant data from the literature. Firstly, the pile installation results are analysed, followed by an assessment of the curing temperatures and strains, based on the review of residual loads in cast-in-situ piles presented in Chapter 3. The behaviour of the instrumented DCIS test piles during maintained compression loading is then discussed, including aspects of shaft behaviour such as friction fatigue and correlations with cone resistance, and the variation in base resistance and stiffness during loading. The performance of seven CPT-based displacement pile design methods for estimating total, shaft and base capacity of DCIS piles in sand is
subsequently examined. Finally, a simple statistical simulation technique is used to compare the load-displacement behaviour of DCIS piles in sand to that of preformed displacement piles using a database of 105 DCIS pile load tests.

Finally, the conclusions of the research are summarised in Chapter 8. The findings demonstrate that DCIS piles behave in a similar manner to traditional preformed displacement pile types, in terms of both shaft and base resistance, and provide the practitioner with increased confidence in estimating DCIS pile capacity using current CPT-based displacement pile design methods. Recommendations for future research are also suggested.

The following additional information is provided in the Appendices:

**Appendix A**: Derivation of the installation resistance using DCIS piling rig instrumentation

**Appendix B**: Dynamic CAPWAP tests during installation

**Appendix C**: Load test schedules for instrumented DCIS piles

**Appendix D**: Load-displacement curves for DCIS test pile database

Finally, the following conference articles were published by the author during the course of this research:

CHAPTER 2  REVIEW OF DISPLACEMENT PILE BEHAVIOUR IN SAND
2.1 INTRODUCTION

Driven cast-in-situ (DCIS) piles are classified as large displacement piles according to the British Standard Code of Practice for Foundations (BSI, 1986), the classification chart for which is illustrated in Figure 2.1. As mentioned previously in Section 1.1 however, there are relatively few case histories of axial load tests on instrumented DCIS piles to verify the assumption that DCIS piles behave in a similar manner to traditional large displacement piles (i.e. closed-ended steel and precast piles). It is therefore necessary to conduct a review of published studies on the behaviour of traditional large displacement piles in sand, the findings of which will subsequently be compared with the results of the instrumented DCIS pile tests in both layered soil and uniform sand deposits in Chapters 5 and 6 respectively.

The prediction of displacement pile capacity in siliceous sand is arguably the most uncertain area in the design of piled foundations, predominantly due to the effects of densification and residual loads during installation (Randolph, 2003). Such uncertainties are highlighted by the relatively poor reliability of traditional pile design methods in these soils (Briaud and Tucker, 1988; Schneider et al., 2008; Toolan et al., 1990). Despite such difficulties, major advances in understanding of the mechanisms which govern displacement pile behaviour in sand have been achieved over the past 25 years or so through extensive high-quality laboratory and full-scale tests using instrumented piles.

This chapter presents a detailed review of closed-ended displacement pile behaviour in sand, in particular the results of a series of tests conducted by Imperial College London using heavily-instrumented closed-ended steel model piles. The behaviour of the shaft and base of displacement piles are reviewed separately, with facets of behaviour relevant to driven cast-in-situ piles highlighted throughout. A summary of the various theoretical and empirically-based displacement pile design methods in sand is provided and their reliability in predicting pile capacity is then discussed. Finally, a detailed review of residual load development in cast-in-situ piles is presented.
Types of Pile

Large displacement

Preformed: solid or hollow closed at the bottom end, driven into the ground and left in position

- Solid
- Precast concrete
  - Formed to required length
- Timber
  - Steel tubes or box piles
- Cast-in-situ: formed in-situ by driving a closed tubular section to form a void, and then filling the void with concrete whilst withdrawing the section
  - Hollow (closed at bottom end and filled or unfilled after driving)
  - Concrete tubes
  - Formed as units with mechanical joints and special driving shoes

Small displacement

Steel sections, includes H-piles, open-ended tubes and box piles

- Screw piles
  - By casing
  - By drilling mud
  - By soil on a continuous flight auger

Replacement

A void formed by boring or excavation and filled with concrete. The sides of the void are:

- Supported
- Unsupported
  - Permanently-cased
  - Temporarily

Figure 2.1 - Pile classification according to British Standards BS8004 Code of Practice for Foundations (BSI, 1986)
2.2 FUNDAMENTALS OF DISPLACEMENT PILE BEHAVIOUR IN SAND

The behaviour of a displacement pile in sand is intrinsically linked to the complex stress-strain history of the soil in which the pile is founded (Randolph, 2003; White, 2005) and the mechanisms which govern such behaviour can be classified in accordance with following phases:

1. Installation
2. Equalisation
3. Loading

White (2005) provides a conceptual model for the stress history of an idealised element of sand during each phase as it transits from an initial location at some distance beneath the base of the pile to a final position adjacent to the pile shaft. The model, illustrated in Figure 2.2, is summarised as follows:

1. Prior to installation, the initial stresses within the element correspond to in-situ horizontal and vertical effective stresses \( \sigma'_{h0} \) and \( \sigma'_{v0} \) respectively.
2. Both stresses within the soil begin to increase during installation, eventually reaching failure when the base of the pile comes into contact with the element. The mean effective stress \( p' \) within the element of soil at this point is comparable with the base resistance \( q_b \) of the pile.
3. The element subsequently passes around the base, inducing a maximum shear stress \( \tau_{s,max} \) along the shaft of the pile in the immediate vicinity of the base. This process is accompanied by a reduction in vertical and horizontal stress, predominantly due to principal stress rotation.
4. The element is then subjected to shearing cycles along the shaft as pile penetration increases (the number of cycles depending on the mode of installation i.e. driven or jacked), leading to reductions in radial stress \( \sigma'_r \) (and hence local shear stress \( \tau_{s,loc} \)) at the soil-pile interface due to contraction of the soil.
5. The pile installation phase is then followed by a period of equalisation, where changes in stress within the element may occur due to pore pressure dissipation, residual loads and age-related effects, resulting in an equalised radial stress \( \sigma'_{rc} \) prior to loading.
6. The loading phase may induce additional increases in radial stress \( \Delta \sigma'_{rd} \) on the shaft due to interface dilation, with shaft failure occurring at peak radial stress \( \sigma'_{rf} \).
The key to advancing the knowledge of displacement pile behaviour in sand, in particular the shaft resistance, lies in the ability to measure the complex changes in stress during the processes highlighted above using heavily-instrumented test piles in the field. As will be shown later, such observations have only been achieved relatively recently, but have led to significant improvements in design methods for displacement piles in sand.

The total capacity $Q_t$ of a circular closed-ended displacement pile during maintained compression loading is defined as the sum of the shaft capacity $Q_s$ and base capacity $Q_b$:

$$Q_t = Q_s + Q_b = \pi D_s \int \tau_{sf} dz + q_b \left( \frac{\pi D_b^2}{4} \right)$$

where $D_s$ and $D_b$ are the pile shaft and base diameter respectively; $\tau_{sf}$ is the local shear stress at failure; $z$ is the pile embedment and $q_b$ is the base resistance.

There is no consensus in the literature on the definition of total pile capacity $Q_t$; various criteria have been proposed, including mathematical models, graphical techniques and settlement–based definitions. However, for this thesis, $Q_t$ is defined as the applied head load corresponding to a head displacement equivalent to 10% of the pile base diameter $D_b$ (as used by McCabe and Lehane (2006) and others) and will be referred to as $Q_{t,0.1D_b}$. 
During the initial stages of loading, the majority of total capacity is provided by shaft friction, with peak values typically mobilised at a shaft displacement $w_s$ equivalent to 2% of the pile shaft diameter $D_s$ (Fleming et al., 2008). The displacement required to fully mobilise the end-bearing resistance is considerably larger in comparison to the shaft resistance, typically occurring at normalised base displacements $w_b/D_b$ of 10% for driven piles and up to 30% for replacement piles according to Kulhawy (1984). Although Equation 2.1 treats the shaft and base resistances separately, interaction effects are likely to occur near the pile tip (Randolph et al., 1994).

A key aspect of pile behaviour which is somewhat unique to preformed displacement piles is that of installation-related residual loads. Rebounding of the pile occurs as the compressive force on the pile head is removed after each load cycle during either driven or jacked installation. This rebounding process, illustrated in Figure 2.3, induces a downward force on the pile due to reversal of shear stresses along the shaft of the pile which must be counteracted by a compressive load at the base. Such loads acting on the pile are commonly referred to as residual loads and may be equivalent to as much as 40% of the total pile capacity according to Alawneh et al. (2001).

![Figure 2.3 - Development of residual load in driven displacement piles (Alawneh et al., 2001)](image-url)
While residual loads are often presumed to have a minimal influence on total pile capacity, effects on the distribution of load within the pile can be profound; ignoring such loads results in over-estimation of shaft resistance and under-estimation of base resistance (Fellenius, 2002). It is therefore crucial that residual loads are accounted for when interpreting the results of tests on instrumented preformed displacement piles. Unfortunately, residual loads in driven piles are notoriously difficult to measure in practice as the large dynamic stresses and accelerations generated within the pile during driving often result in erroneous readings from the pile instrumentation. As a result, residual loads that have developed upon installation have been ignored in many studies of driven piles in sand, thereby leading to erroneous interpretations of shaft and base resistance (Fellenius and Altaee, 1995). Recent high-quality investigations of displacement pile behaviour have used jacked installation methods as an alternative to driving, as the absence of dynamic-related installation effects synonymous with driving are largely avoided, enabling residual loads to be measured with a high degree of accuracy. However, it is important to note that there are several important differences between the behaviour of jacked and driven displacement piles in sand; these variations are highlighted in the forthcoming review.

The applicability of the classic driving-related mechanism of residual load development described above to DCIS piles is somewhat questionable. While elastic-rebound of the DCIS installation tube will inevitably occur during driving, the subsequent extraction process eliminates the presence of shear stresses at the pile-soil interface immediately after concreting. However, there are other processes which occur after DCIS pile installation which may lead to the development of residual loads; processes such as downdrag due to consolidation-related settlement which results from the dissipation of excess pore pressures generated within cohesive soils by driving and/or surcharging-related effects caused by the placement of fill at the surface for piling operations. The interaction of concrete with the surrounding soil as it cures after casting has also been postulated as a potential source of residual load for cast-in-situ piles (Pennington, 1995; Fellenius et al., 2009; Siegel and McGillivray, 2009). Existing research on these processes is reviewed in detail in Chapter 3 when discussing strain gauge interpretation procedures and is subsequently revisited during the instrumented DCIS pile test interpretation described in Chapters 5 and 6.
The following sections present a review of the mechanisms governing displacement pile behaviour in sand; Section 2.3 examines aspects of shaft behaviour, while an overview of the base resistance is presented in Section 2.4. In keeping with the framework of displacement pile behaviour by White (2005) described above, the review is structured in accordance with the three key phases which influence the stress-stain history of the soil in the immediate vicinity of the displacement pile, i.e. installation, equalisation and loading. Additional important factors which are not strictly related to these phases are subsequently discussed. The review primarily focuses on field studies using instrumented displacement piles, although laboratory-scale centrifuge and calibration chamber tests are also discussed. Each section concludes with a summary of the key mechanisms governing displacement pile behaviour and implications of such behaviour for DCIS piles.
2.3 AN OVERVIEW OF THE SHAFT RESISTANCE OF CLOSED-ENDED DISPLACEMENT PILES IN SAND

2.3.1 Introduction
This section presents a review of published literature on the shaft behaviour of closed-ended displacement piles in sand. It is worth noting at this stage that the majority of published studies on the shaft resistance of displacement piles in sand relate to steel piles; there are relatively few high-quality studies of instrumented concrete piles in which radial stresses on the pile shaft were measured during installation, equalisation and loading. Such a trend is hardly surprising given that major research projects on displacement pile behaviour have been largely funded by the offshore industry, where large-diameter open-ended steel piles are driven to depths in excess of 30 m in order to support structures such as platforms and wind turbines. The absence of high-quality studies using concrete piles can also be attributed to processes synonymous with concrete such as in-situ curing, shrinkage, creep, cracking and strain-dependent stiffness. As will be shown in Chapters 3, 5, and 6, these processes may have a significant influence on the performance of pile instrumentation and the subsequent interpretation of pile behaviour.

The local shear stress at failure $\tau_{sf}$ of a displacement pile in sand is expressed using the Coulomb friction equation:

$$\tau_{sf} = \sigma'_{rf} \tan \delta_f$$  \hspace{1cm} 2.2

where $\sigma'_{rf}$ is the radial effective stress at failure and $\delta_f$ is the interface friction angle at failure.

The interface friction angle $\delta_f$ is easily measured in a laboratory using a shear box apparatus, with the majority of uncertainty in Equation 2.2 associated with prediction of the radial effective stress at failure $\sigma'_{rf}$. Traditional methods relate $\sigma'_{rf}$ to the in-situ vertical effective stress $\sigma'_v$ using conventional earth pressure theory:

$$\tau_{sf} = K_f \sigma'_v \tan \delta_f$$  \hspace{1cm} 2.3

where $K_f$ is the lateral earth pressure coefficient at failure and $\sigma'_v$ is the vertical effective stress.
The lateral earth pressure coefficient $K_f$ depends on several factors, including the level of soil displacement during installation and in-situ soil state. Recommended values of $K_f$ for design tend to be conflicting; Kraft (1990) for example suggests $K_f = 0.4$ and 1.6 for piles driven in loose and dense sands respectively, while previous editions of the American Petroleum Institute design standard for offshore pile design (API, 2007) specified a constant value of $K_f = 1.0$ for closed-ended displacement piles, regardless of density. Such values imply a linear relationship between local shaft friction and vertical effective stress, which is in contrast with observations of shaft resistance measured during instrumented pile tests in the field. Reviews of pile design methods over the years have also highlighted the relatively poor performance of earth pressure methods in predicting displacement pile capacity (Schneider et al., 2008).

The uncertainties surrounding traditional design methods prompted several high-quality investigations into the mechanisms governing the behaviour of shaft resistance of displacement piles in sand. Unlike previous studies on displacement piles behaviour in sand, these research programmes have successfully captured the changes in radial effective stress on the shaft of preformed displacement piles during installation, equalisation and loading in siliceous sand. The following sections review the results of such tests, in particular the extensive investigations conducted by Imperial College London.

2.3.2 Installation

As mentioned previously, reliable measurements of axial loads, radial stresses, pore pressures and residual loads during driven installation are notoriously difficult due to the influence of the large dynamic stresses generated by driving on the pile instrumentation. High-quality measurements of displacement pile behaviour, in particular radial stresses during installation, have been rather limited as a result. Such limitations have been overcome in recent years through the use of jacked installation methods (whereby the pile is pushed to the required depth using continuous jacking strokes) as an alternative to driving, and the majority of literature of displacement pile installation behaviour pertains to jacked piles as a result.

Undoubtedly the most extensive observations of displacement pile behaviour during installation in sand were obtained by researchers at Imperial College London over a 10 year period beginning in the late 1980s as part of a comprehensive research programme geared
towards offshore applications. Pile tests were conducted in a range of soil types at numerous locations in the UK and France using a heavily instrumented, 102 mm diameter, closed-ended steel model pile (referred to as the Imperial College Pile or ICP). The pile contained instrumentation clusters housed at three separate levels within a 3 m long section near the base of the pile (an additional cluster level was added at a later date for testing in dense sand at Dunkirk), enabling near-continuous measurements of radial and local shear stresses (using a surface stress transducer or SST), axial loads and pore pressures during installation, equalisation and loading. A schematic of the ICP is shown in Figure 2.4(a); cluster levels are identified by the distance $h$ from the base, normalised by the pile diameter $D$. Extension pieces attached to the instrumented section enabled a total pile length in excess of 6 m. The CPT $q_c$ profiles at the two ICP test sites (Labenne and Dunkirk) are illustrated in Figure 2.4(b).

![Figure 2.4 - (a) Schematic of ICP test pile and (b) CPT $q_c$ profiles at Labenne and Dunkirk](image-url)
The results of two ICP tests in loose to medium dense sand at Labenne, southwest France, were reported by Lehane (1992). The piles were incrementally jacked using ≈ 200 mm strokes to a final depth of 6 m below ground level. The following observations of shaft behaviour during installation were reported:

- Pore pressures remained near-hydrostatic during jacking, indicating that the pile installation process was essentially drained.
- Stationary radial effective stress $\sigma'_{rs}$ and local shear stress $\tau_{s,loc}$ profiles at each sensor level between jacking strokes (Figure 2.5) closely resembled the measured cone resistance $q_c$ profile at the test location, indicating that shaft behaviour during installation was strongly controlled by in-situ soil state.
- As evident in Figure 2.5, clear reductions in $\sigma'_{rs}$ and $\tau_{s,loc}$ at a given depth occurred with increasing pile embedment.
- The average shear stress $\tau_{s,avg} = Q_{s,total}/\pi D_z$ along the shaft of the pile reached quasi-constant values when pile embedment exceeded 2 m, corresponding to a normalised value $z/D \approx 20$.
- The interface friction angles $\delta_f$ inferred from radial and shear stresses during jacking were in good agreement with the critical state ($\delta_{cv}$) values measured in laboratory interface shear tests on steel surfaces with similar roughness.

An additional series of ICP tests were performed by Chow (1997) in a dense sand deposit at Dunkirk, France to enable a comparison with the results obtained in the loose to medium dense sand at Labenne. The testing programme comprised three jacked installations to a maximum depth of 7.4 m using 225 mm strokes. Radial and local shear stresses, pore pressures and axial loads were once again measured simultaneously during installation, equalisation and loading. Despite stationary radial effective stresses being almost four times greater than those observed in the loose to medium-dense sand at Labenne, the pattern of installation behaviour of the ICP in dense sand was similar to that reported by Lehane (1992) in medium dense sand at Labenne, with degradation in radial and shear stresses occurring with increasing penetration (see Figure 2.6), pore pressures remaining hydrostatic and average shear stresses becoming relatively constant for a normalised embedment $z/D > 30$. 
Figure 2.5 - Variation in (a) stationary radial effective stress and (b) local shear stress during ICP installation at Labenne (Lehane 1992)

Figure 2.6 - Variation in (a) stationary radial effective stress and (b) local shear stress during ICP installation at Dunkirk (Chow 1997)
The systematic reduction in stationary radial effective stress acting on the pile shaft with increasing pile embedment at a given distance $h$ from the base evident at Labenne and Dunkirk is known as ‘friction fatigue’ (Heerema, 1980). Despite contrasting density levels at the ICP sites, the reductions in normalised radial stress $\sigma'_{rs}/q_c$ with increasing $h/R$ (where $R$ is the pile radius) during installation were remarkably similar (Figure 2.7). A relatively unique relationship between $\sigma'_{rs}/q_c$ and $h/R$ may be inferred from this observation, but it is important to note that the piles at both sites experienced comparable levels of cyclic shearing during installation.

![Figure 2.7 - Comparison of normalised radial effective stresses during ICP installation at Labenne and Dunkirk (Chow, 1997; Lehane, 1992)](image)

It is widely accepted that the primary mechanism governing friction fatigue in sand is the gradual densification of the thin interface zone immediately adjacent to the shaft under the cyclic shearing action of driven or jacked installation (Randolph, 2003). This mechanism is illustrated in Figure 2.8, where the thin shear band at the pile interface, denoted Zone B, is surrounded by a mass of soil (Zone A) with high lateral stiffness that undergoes minimal deformation during shearing. As evident in Figure 2.8, contraction of the interface shear band during cycling shearing leads to unloading of the adjacent soil mass, with reductions in radial effective stress $\sigma'_r$ occurring as a result. Additional reductions in $\sigma'_r$ may also be attributed to particle-breakage within the interface shear band, as observed by White and Bolton (2004) during installation of model displacement piles in plane-strain calibration chamber tests using particle image velocimetry (PIV).
The above analogy suggests that the magnitude of friction fatigue is proportional to the number of shearing cycles experienced during installation; observations of local shear stress on displacement piles in the field support this concept. For example, Figure 2.9 shows a comparison of the shaft friction profile observed during continuous jacking of a cone penetrometer (DeJong and Frost, 2002) at a glacial sand site in Vermont, USA with that reported by Kolk et al. (2005b) during driving of a highly-instrumented 762 mm outer diameter steel open-ended pile in very dense sand ($q_c \approx 70$ MPa) at Eemshaven, The Netherlands. DeJong and Frost (2002) modified the penetrometer with multiple friction sleeves, enabling measurements of sleeve friction at $h/D$ values of between 3 and 35. Large jacking strokes ($\approx 1$ m) permitted the number of shearing cycles to be minimised during installation and, as evident in Figure 2.9(a), degradation in sleeve friction with increasing $h/D$ was practically non-existent as a result (minor reductions were attributed to variations in diameter of the friction sleeves). In contrast, the open-ended pipe pile at Eemshaven was subjected to over 3000 shearing cycles during hard driving in very dense sand. Maintained compression load tests performed at penetration depths of 30.5 m, 38.7 m and 46.9 m indicated high rates of degradation in local shear stress as pile embedment increased (Figure 2.9b). These case-histories undoubtedly represent the extrema in cyclic shearing during installation but reinforce the argument that the number of shearing cycles (and hence mode of installation) plays a major role in the distribution of radial and shear stresses on displacement piles in sand.
Figure 2.9 - Shaft friction profiles of (a) a multi-sleeved cone penetrometer (De Jong and Frost, 2002) and (b) a 762 mm outer diameter open-ended pile (Kolk et al., 2005b)

White and Lehane (2004) conducted a series of centrifuge tests to examine the effect of installation mode on the shaft resistance of steel model piles in fine silica sand. The piles had a nominal width $B = 9$ mm and were instrumented with pressure cells at several positions along the shaft to enable measurement of horizontal stress. Three modes of installation were investigated:

a) **Monotonic** installation, whereby the pile was pushed to a depth of 120 mm using two continuous 60 mm jacking strokes (to minimise cyclic shearing).

b) **Jacked** installation, whereby the pile experienced a set number of one-way shearing cycles comprising a downward displacement of $\approx 2$ mm, followed by unloading to 0 kN.

c) **Pseudo-dynamic** installation, whereby the pile was subjected to a set number of two-way cycles (to simulate the process of elastic rebound synonymous with driven piles) by applying a downward displacement of $\approx 2$ mm, followed by an upward displacement ($\approx 1.5$ mm) and then unloading to 0 kN.

A total of 18 pile installations were performed to a final penetration depth of 120 mm. Measurements of peak horizontal effective stress during pile movement, designated $\sigma'_{hp}$, were obtainable for monotonic installation only, as the stroke lengths for jacked and pseudo-
dynamic installation were insufficient to mobilise peak values. Normalised horizontal stresses \( \sigma'_{hp}/q_c \) remained relatively constant during monotonic installations, regardless of distance \( h \) from the pile tip; such behaviour is comparable with the observations by DeJong and Frost (2002) during continuous jacking of the multi-sleeved cone penetrometer as mentioned previously. The following trends in stationary horizontal effective stress \( \sigma'_{hs} \) during monotonic, jacked and pseudo-dynamic installation were observed:

- The largest values of \( \sigma'_{hs} \) were present near the base of the piles, regardless of installation mode, and reduced with increasing distance \( h \) from the pile tip.
- Measurements of \( \sigma'_{hs} \) along the upper portion of the pseudo-dynamically installed piles were typically only 10% of the stationary values measured between jacking strokes during monotonic installation.

Figure 2.10(a) shows the average variation in normalised stationary horizontal effective stress \( \sigma'_{hs}/q_c \) with normalised distance \( h/B \) from the pile tip for jacked and pseudo-dynamic installations; unlike the observations of friction fatigue during the ICP tests at Labenne and Dunkirk (see Figure 2.7), no unique relationship between \( \sigma'_{hs}/q_c \) and \( h/B \) was apparent from the centrifuge tests.

![Figure 2.10](image)

Figure 2.10 - Variation in normalised stationary radial stress during installation with (a) distance from the pile tip and (b) number of loading cycles (White and Lehane, 2004)
The variable rates of friction fatigue observed in Figure 2.10(a) could be explained by the fact that the jacked piles experienced considerably fewer shearing cycles in comparison to pseudo-dynamic installations. As illustrated in Figure 2.10(b), the degradation in $\sigma'_{\text{hs}}/q_c$ during installation is more intrinsically linked to the number of shearing cycles $N$ experienced at a given location on the pile shaft, rather than the absolute distance $h$ from the pile base.

Uncertainties surrounding the extrapolation of laboratory-based studies of pile behaviour to full-scale conditions prompted an investigation of friction fatigue during installation of a 73 mm diameter closed-ended steel pile in dense sand by Gavin and O’Kelly (2007). The pile was instrumented with total earth pressure cells and pore pressure transducers at shaft locations corresponding to $h/D$ values of 1.5, 5.5 and 10 in order to measure radial effective stresses during installation. Unlike the centrifuge study by White and Lehane (2004), stroke lengths were sufficiently large ($> 25$ mm) to mobilise peak radial stresses $\sigma'_{\text{rp}}$ during both monotonic and cyclic installations. While stationary radial stresses during installation were in agreement with the centrifuge results by White and Lehane (2004), the following observations of peak radial stress were noteworthy:

- Dilation-related increases in radial stress during monotonic and cyclic installation strokes resulted in peak radial stresses $\sigma'_{\text{rp}}$ being three to four times greater than stationary values.
- Peak radial stresses were greatest near the base of the monotonically-installed pile and reduced with increasing normalised distance $h/D$ from the base, regardless of installation mode.
- Profiles of $\sigma'_{\text{rp}}$ at $h/D \geq 5.5$ during monotonic and cyclic installation were essentially identical, which suggested that minimal shearing cycles ($N \approx 2$ for monotonic installation) were necessary to cause large reductions in $\sigma'_{\text{rp}}$ remote from the base.

The study provided field-scale verification of the relationship between friction fatigue and the number of load cycles during installation. However, the degradation in peak radial stress with increasing $h/D$ during monotonic installation was in stark contrast to the findings by White and Lehane (2004) which suggested that peak horizontal stresses on monotonically-installed piles were relatively independent of the distance from the pile base. The larger radial stresses observed near the base of the monotonically-installed piles were partly attributed to the presence of residual stresses during installation (Gavin and O’Kelly 2007).
While the rate of degradation in radial stress is clearly dependent on the number of shearing cycles $N$ during installation (as highlighted in the studies described previously), existing design approaches tend to model friction fatigue as a function of normalised distance from the base $h/D$ (or $h/R$). Lehane (1992), for example, derived the following expression for the reduction in stationary radial stress $\sigma'_{rs}$ with increasing $h/R$, based on the ICP installation tests at Labenne:

$$\sigma'_{rs} = 0.024q_c \left(\frac{h}{R}\right)^{-0.33}$$  \hspace{1cm} 2.4

As shown in Figure 2.7, Equation 2.4 provides a reasonable estimate of the level of friction fatigue during ICP installation at Dunkirk. However, as mentioned previously, the degradation rates were similar as both piles experienced near-identical levels of shearing during jacked installation. A review of over 70 high-quality load tests on instrumented driven piles in sand by Lehane et al. (2005b; 2007) showed considerably greater rates of friction fatigue in comparison to jacked piles, with statistical analyses yielding the following best-fit expression for the degradation in stationary radial stress for driven closed-ended piles:

$$\sigma'_{rs} = 0.03q_c \left(\frac{h}{D}\right)^{-0.5}$$  \hspace{1cm} 2.5

The applicability of friction fatigue to DCIS piles is conceivable, although unverified, despite the fact that the steel installation tube is subsequently extracted and replaced with fresh concrete. The soil surrounding the pile is subjected to extreme loading cycles as the tube is driven into the soil; two-way cyclic shearing is likely to occur along the interface of the shaft of the tube due to elastic rebounding of the tube after each hammer blow. Therefore, friction fatigue is considered a potential characteristic of DCIS pile behaviour and is subsequently investigated as part of the instrumented DCIS pile tests described in Chapters 5 and 6.

### 2.3.3 Equalisation

The equalisation periods ($\approx 15$ hours) of the ICP tests at Labenne and Dunkirk were characterised by instantaneous stabilisation of radial and shear stresses after completion of the final installation stroke and negligible changes thereafter (modest increases in radial stress at Dunkirk were attributed to creep-related effects in the sand). Such observations led Lehane (1992) to conclude that long-term equalised radial stresses $\sigma'_{rc}$ acting on the pile shaft were essentially identical to stationary values observed between installation strokes and could subsequently be expressed using Equation 2.4.
2.3.4 Loading
A comprehensive series of static and cyclic load tests was performed on the ICP piles at Labenne and Dunkirk to investigate the factors affecting pile behaviour during loading in sand. A unique aspect of the tests was the ability to simultaneously measure changes in local shear stresses and radial effective stresses using surface stress transducers (SST), enabling effective stress paths and interface friction angles during loading to be determined. This section reviews the results of first-time compression static load tests only; tensile load tests are covered separately as part of loading direction effects in Section 2.3.8. The shaft behaviour of the ICP during compressive loading at both sites was essentially identical, with the following observations noteworthy:

- Pore pressures remained hydrostatic throughout the loading process, indicating drained pile behaviour.
- Radial stresses at failure $\sigma'_rf$ were up to 50 % greater than equalised values $\sigma'_rc$ prior to loading.
- In a similar manner to installation, profiles of radial and local shear stress at failure closely resembled the corresponding CPT $q_c$ profiles at each site and reduced with distance from the base.

Figure 2.11 shows the changes in radial effective stress $\sigma'_r$ with local shear stress $\tau_{s,loc}$ at the three instrument clusters of the ICP during first-time compressive loading at Labenne. The initial stages of loading were characterised by minor reductions in radial stress at all levels which Lehane (1992) attributed to principal stress rotation and soil anisotropy. Such reductions were temporary however, with significant increases in radial stress occurring due to interface dilation (discussed further in Section 2.3.6) as the pile approached failure.

The interface friction angles at failure $\delta_t$ inferred from measurements of radial and shear stress on the SSTs were in close agreement with the constant volume friction angle $\delta_{cv}$ measured during direct shear interface tests using steel surfaces of similar roughness to the SSTs; this outcome prompted Lehane (1992) to conclude that ultimate shaft friction is controlled by $\delta_{cv}$, rather than the peak interface angle $\delta_{peak}$ as was previously assumed.
Figure 2.11 - Effective stress path during first-time ICP compression loading at Labenne (Lehane 1992)

The clearest trend to emerge from the ICP test programmes at Labenne and Dunkirk was the verification that the local shear stress at failure $\tau_{sf}$ for preformed displacement piles in sand is governed by the Coulomb failure criterion:

$$\tau_{sf} = \sigma'_{rf} \tan\delta_f = (\sigma'_{rc} + \Delta\sigma'_{rd}) \tan\delta_{cv}$$  \hspace{1cm} 2.6$$

where $\sigma'_{rf}$ is the radial effective stress at failure, $\sigma'_{rc}$ is the radial stress after installation and equalisation ($= \sigma'_{re}$), $\Delta\sigma'_{rd}$ is the increase in radial stress during loading due to interface dilation and $\delta_f$ is the interface friction angle at failure ($= \delta_{cv}$).

The ICP tests demonstrated the major influence of radial stresses on the mechanisms governing the shaft behaviour of displacement piles in sand. However, several other factors which influence displacement behaviour in sand are now discussed.
2.3.5 Interface friction

The local shear stress of a displacement pile in sand depends not only on the complex changes in radial effective stress which occur during installation, equalisation and loading, but also on the pile-soil interface friction angle $\delta$, as evident from the Coulomb criterion in Equation 2.6. This parameter is affected by factors such as interface roughness, particle size and shape, as well as soil density and confining stress. For example, interface shear tests by Potyondy (1961) showed that higher peak friction angles $\delta_{\text{peak}}$ for sand were obtained when sheared against rough surfaces compared to smoother surfaces.

The effect of particle size on interface shearing behaviour is illustrated in Figure 2.12 (Uesugi and Kishida, 1986) where it is apparent that, for a given surface roughness, the interface friction angle reduces with increasing mean particle size $D_{50}$; results of steel interface shear tests using a range of particle sizes by Jardine et al. (1993) verify this trend (see Figure 2.13a).

![Figure 2.12 - Effect of particle size on interface shearing behaviour (Uesugi & Kishida 1986)](image)

A relatively unique relationship between interface roughness, mean particle size and interface friction angle is obtained using the normalised roughness parameter $R_n = R_a / D_{50}$ proposed by Uesugi and Kishida (1986); results of interface shear test results by Jardine et al. (1993), Dietz (2000) and Frost et al. (2002) show increases in friction coefficient $\mu = \tan \delta_{\text{cv}}$ with increasing normalised roughness $R_n$. However, above a critical roughness $R_n, \text{crit} \approx 0.08 - 0.2$, shear failure occurs within the adjacent soil mass and becomes independent of the interface properties (i.e. $\delta = \phi'_{\text{cv}}$), as illustrated in Figure 2.13(b) by Lehane et al. (2007a).
As the normalised roughness of the DCIS pile shaft is likely to be greater than $R_{n,\text{crit}}$ (due to the rough concrete surface created by in-situ placement), the interface friction angle at failure $\delta_{cv}$ will be equivalent to the constant volume friction angle $\phi'_{cv}$ of the surrounding sand; this parameter is an intrinsic soil property and a review of the strength and dilatancy characteristics of sands by Bolton (1986) suggests $\phi'_{cv} = 33^\circ \pm 1^\circ$ for quartz sands. As the majority of the sites described in Chapter 4 had insufficient interface shear test data, $\delta_{cv} = 33^\circ$ was therefore assumed when deriving radial effective stresses acting on the instrumented DCIS test piles from measured profiles of local shear stress at failure in the sand layers using Equation 2.6.

### 2.3.6 Interface dilation

Changes in radial stress during loading are generally attributed to constrained dilation of the sand particles at the pile-soil interface under the action of shearing. Such changes can be predicted using the following expression proposed by Boulon and Foray (1986) based on cavity expansion theory:

$$\Delta\sigma'_{rd} = \frac{4G\Delta y}{D} = k_n \Delta y \quad 2.7$$

where $\Delta\sigma'_{rd}$ is the increase in radial stress due to interface dilation, $G$ is the shear stiffness of the sand mass constraining dilation, $\Delta y$ is radial displacement of shear band, $D$ is the pile diameter and $k_n$ is the equivalent normal stiffness.
Equation 2.7 implies that interface dilation varies inversely with pile diameter, thus providing an explanation for the higher values of shaft friction typically observed on model scale piles in comparison to field-scale piles. For example, tension tests on centrifuge model piles in sand by Lehane et al. (2005a) showed large variations in average shear stress $\tau_{\text{avg}}$ with minor changes in model pile diameter (Figure 2.14a), while Gavin et al. (2009) reported near-identical profiles of average shear stress (Figure 2.14b) mobilised on two instrumented CFA piles with nominal diameters of 450 mm and 800 mm respectively during static loading in sand, suggesting interface dilation was negligible. Therefore, significant uncertainties exist regarding the extrapolation the results of model-scale pile tests to full-scale conditions.

The magnitude of $\Delta \sigma'_{pd}$ also depends on the level of displacement $\Delta y$ during dilation, which in turn varies with average interface roughness $R_a$, mean particle size $D_{50}$ and stress level. A review of interface shear test data by Schneider (2007) show substantial increases in $\Delta y$ when values of $R_a$ exceed $\approx 0.02$ mm (see Figure 2.15a and b); such observations suggest that the rough shaft surface of cast-in-situ piles (including DCIS piles) may lead to somewhat enhanced levels of interface dilation in comparison to steel piles. However, such increases may be minor in comparison to the effects of diameter on $\Delta \sigma'_{pd}$ at full-scale conditions, as evident from the results of the CFA pile tests by Gavin et al. (2009) mentioned previously.
2.3.7 Pile embedment

The results of a series of static load tests on instrumented closed-ended steel piles of variable length in silty sand by Vesic (1970) showed that the average shear stress $\tau_{s,avg}$ reached quasi-constant values beyond a ‘critical’ depth of embedment (see Figure 2.16a) which Vesic (1970) attributed to the development of limiting values of local shear stress near the pile base. The ‘critical depth’ concept has proved controversial over the years (Kulhawy, 1984; Fellenius and Altee, 1997), predominantly due to the fact that Vesic (1970) failed to account for the presence of residual loads during driven installation.

![Figure 2.15 - Variation in dilation-related displacement with (a) average interface roughness and (b) normalised roughness (Lehane et al., 2007a; Schneider, 2007)](image1)

![Figure 2.16 - Variation in average shear stress profiles with pile embedment at (a) Ogeechee River (Vesic 1970) and (b) Labenne and Dunkirk (Lehane 1992; Chow 1997)](image2)
Constant values of average shear stress were also observed during ICP installations at Labenne and Dunkirk (Figure 2.16b); Lehanee (1992) concluded that the convergence of $\tau_{s,avg}$ to a limiting value was a direct result of degradation in local shear stress at sections remote from the pile base with increasing pile embedment during installation and not due to limiting values of local shear stress near the base as suggested by Vesic (1970).

### 2.3.8 Direction of loading

Laboratory- and field-scale studies of displacement pile behaviour in sand in the 1960s and 1970s suggested that the tensile shaft resistance was less than that observed during compressive loading. The magnitudes of reduction reported in the literature varies however, with differences of up to 65 % reported by Gregersen et al. (1973) during compressive and tensile load tests on instrumented precast concrete piles, while Briaud and Tucker (1984) suggested typical reductions of between 5 % and 30 %, based on a database of 11 displacement pile tests in which installation-related residual loads were measured with a reasonable degree of accuracy.

The variation in shaft resistance with loading direction was investigated as part of the ICP test programmes at Labenne and Dunkirk. Comparison with the first-time compressive load test results previously described in Section 2.3.4 showed that:

- Reductions in radial stress were considerably greater during the initial stages of tensile loading.
- Peak local shear stresses $\tau_{sf}$ and radial effective stresses at failure $\sigma'_{rf}$ were typically 20 % less than values observed during compressive loading.
- Interface friction angles at failure $\delta_f$ were relatively independent of loading direction.

A parametric study of displacement pile behaviour in sand by De Nicola and Randolph (1994) showed that differences in shaft capacity with loading direction were primarily due to principal stress rotation, soil anisotropy and increases in cross-sectional area of the pile during compressive loading, termed the ‘Poisson effect’. Reductions in local shear stress during tensile loading are typically accounted for in modern pile design methods by applying a single reduction factor $f_t$ to the Coulomb equation:

$$
\tau_{sf} = f_t \sigma'_{rf} \tan \delta_{cv}
$$

2.8
2.4 AN OVERVIEW OF THE BASE RESISTANCE OF CLOSED-ENDED DISPLACEMENT PILES IN SAND

2.4.1 Introduction

This section provides a review of the base resistance of closed-ended displacement piles during installation, equalisation and loading in sand. Additional factors influencing base behaviour such as the level of soil displacement, stiffness, soil layering, partial embedment and correlations with CPT cone resistance $q_c$ (including averaging techniques) are subsequently discussed.

The base resistance $q_b$ of a pile foundation in sand is traditionally described using bearing capacity theory:

$$ q_b = N_q \sigma'_{v0} \tag{2.9} $$

where $N_q$ is a dimensionless bearing capacity factor and $\sigma'_{v0}$ is the vertical effective stress at the pile base. Given the excessive pile displacements typically required to achieve plunging failure in sand, the failure criterion for base resistance is routinely defined as the stress corresponding to a base displacement $w_b$ equivalent to 10% of the pile base diameter $D_b$ and will be referred to as $q_{b,0.1D_b}$ (Randolph, 2003; Gavin and Lehane, 2007).

Several theories regarding the mechanism of bearing failure at the base of a displacement pile in sand currently exist, e.g. Berezantsev et al. (1961), Brinch Hansen (1961) and Vesic (1972), the majority of which assume the bearing capacity factor $N_q$ varies logarithmically with the friction angle $\phi'$ of the soil at the base of the pile (Figure 2.17). However, additional factors such as the reduction in $\phi'$ (Cheng, 2004) and soil rigidity $I_r = G/p'_{0}$ (Gupta, 2002) with increasing confining stress (or depth) are not accounted for. Such simplistic bearing capacity theories therefore tend to be in poor agreement with observations of base resistance measured in the field (see Coyle & Castello 1981).

As described previously in Section 2.2, White’s (2005) conceptual model of the idealised stress-strain history of the soil during displacement pile installation and loading advocates that the base resistance is more intrinsically related to the mean effective stress at failure $p'_f$ of the soil beneath the pile tip, rather than the initial vertical stress $\sigma'_{v0}$ prior to loading implied by
Equation 2.9. Furthermore, the similar penetration processes of a displacement pile and a CPT cone penetrometer suggests that the steady-state base resistance at failure $q_{b,ult}$ is directly comparable with the cone resistance $q_c$ and is therefore discussed at length in forthcoming sections.

![Figure 2.17 - Relationships between bearing capacity factor $N_q$ and sand friction angle $\phi'$ (Coyle & Castello 1981)](image)

### 2.4.2 Installation and equalisation

Although the Imperial College Pile test programmes primarily investigated the mechanisms governing shaft behaviour of displacement piles in sand, the axial load cell at the tip of the ICP enabled continuous measurements of base resistance during installation at Labenne and Dunkirk. The following observations of base behaviour at both sites were apparent:

- The base resistances during installation were in reasonable agreement with their corresponding CPT $q_c$ profiles (Figure 2.18); the smoother profiles of base resistance during installation compared to the $q_c$ profiles were attributed to differences in diameter between the pile and the cone penetrometer.
- Pore pressures measured at the sensor closest to the base ($h/D = 4$) remained hydrostatic throughout installation and the subsequent equalisation periods.
- The higher jacking loads required to install the ICP in dense sand at Dunkirk resulted in residual base loads being nearly four times greater than values observed after ICP installation in the loose to medium dense sand at Labenne.

The comparable profiles of end-bearing resistance $q_b$ and CPT $q_c$ during ICP installation in sand is hardly surprising given their analogous modes of penetration. Similar profiles of $q_b$
and \( q_c \) have been reported during jacked installation of laboratory-scale displacement piles in centrifuge tests (e.g. Bruno and Randolph, 1999; Klotz and Coop, 2001; Deeks and White 2007) and calibration chamber tests (Lehane and Gavin, 2001; Gavin and Lehane, 2003). However, the majority of these studies were performed in relatively homogeneous strata where steady-state values of \( q_b \) and \( q_c \) were mobilised after a shallow embedment and remained quasi-constant with increasing depth. Layered deposits on the other hand tend to highlight significant scale effects due to the differences in diameter between a displacement pile and a cone penetrometer. A suitable averaging method must therefore be applied to the CPT \( q_c \) profile to account for such scale effects prior to assessing correlations with \( q_{b,0.1Db} \). Assessments of the various averaging techniques are discussed in detail in Section 2.4.5.

![Figure 2.18](image_url)

**Figure 2.18 - Comparison of base and cone resistance profiles during ICP installation at Labenne and Dunkirk (after Lehane 1992 and Chow 1997)**
2.4.3 Loading

The base behaviour of a displacement pile during loading in sand depends on factors such as the level of soil displacement and residual loads during installation, soil stiffness and stress history. Several of these factors were investigated by Gavin and Lehané (2007) through a series of static load tests on a 111 mm outer diameter open-ended pile, a 73 mm diameter closed-ended pile and a 100 mm diameter bored pile in dense sand. Strain gauges placed at the base of all piles enabled accurate measurement of base resistance during loading. The variation in base resistance $q_b$ with normalised base displacement $w_b/D_b$ for the three pile types during loading is shown in Figure 2.19(a) where it is apparent that:

- No base displacement occurred for the displacement piles until the applied stress exceeded the residual stress present at the base after installation.
- Stiff linear base stress-displacement responses were evident for all piles during the initial stages of loading.
- The highest base stresses were mobilised on the closed-ended pile, followed by the open-ended pile which remained fully plugged during loading. The bored pile had a significantly reduced base response in comparison to the displacement piles.

![Figure 2.19 - Variation in (a) base resistance response and (b) equivalent linear base stiffness with normalised base displacement during maintained compression load tests (Gavin and Lehané, 2007)](image-url)

The equivalent linear secant base stiffness $E_{b,eq}$ of the piles were back-figured from the measured base stress-displacement responses using the following equation:

$$E_{b,eq} = \frac{q_b D_b (1 - u^2) \pi}{w_b}$$  

2.10
where Poisson’s ratio $\nu \approx 0.2$ for drained loading in sand (Mitchell and Soga, 2005). Figure 2.19(b) illustrates the variation in $E_{b, eq}$ with normalised displacement $w_b/D_b$ during loading of the test piles. The following behaviour was noted:

- Values of $E_{b, eq}$ remained relatively constant during the initial stages of loading and were comparable with in-situ measurements of small-strain stiffness $E_0$ prior to installation.
- Degradation in $E_{b, eq}$ began at a normalised yield displacement $w_{by}/D_b$ which varied with pile type, being lowest for the bored pile ($w_{by}/D_b \approx 0.8\%$) and greatest for the closed-ended pile ($w_{by}/D_b \approx 2\%$).

Gavin and Lehane (2007) attributed the prolonged stiff linear response of the displacement piles during the initial stages of loading to the high levels of prestress induced at the base of the piles after jacked installation. The effects of prestress were further examined by performing several unload-reload cycles on the bored pile during loading. While cycling had no effect on the initial stiffness response, the formation of residual loads at the base after unloading led to an increase in the value of $w_{by}/D_b$ during subsequent reloading.

The findings of Gavin and Lehane (2007) suggest that the base stress-displacement response of a pile in sand can be modelled if the small-strain stiffness $E_0$ and normalised yield displacement $w_{by}/D_b$ are known; $E_0$ is typically measured in-situ using a Seismic CPT or predicted using empirical correlations such as those proposed by Mayne (2006) and Robertson (2009). The following expression for estimating $w_{by}/D_b$ was derived by Gavin and Lehane (2007) based on the field tests described above and laboratory-scale experiments by Gavin (1998):

$$\frac{w_{by}}{D_b} = \frac{q_{b,ult} \left(1-\nu^2\right)\pi/4}{E_0} \leq 1.5\% \quad 2.11$$

where $q_{b,ult}$ is the base stress at plunging failure ($\approx q_c$ for closed-ended displacement piles).

The following idealised base stress-displacement response, illustrated in Figure 2.20, was proposed by Gavin and Lehane (2007), comprising of the following phases:

1. An initial phase where no base displacement occurs until the base stress exceeds the residual stress $q_{b, res}$ present at the base prior to loading.
2. A linear phase where the stiffness of the base stress-displacement response corresponds to the in-situ small-strain stiffness $E_0$.

3. A non-linear phase, the beginning of which coincides with the onset of stiffness degradation at $w_{by}/D_b$, resulting in a parabolic stress-displacement response up until $w_{by}/D_b = 10\%$.

![Idealised base resistance-displacement response](image)

**Figure 2.20 - Idealised base resistance-displacement response (Gavin and Lehane, 2007)**

The linear phase can be expressed as follows:

$$q_b = k \left( \frac{w_b}{D_b} \right) + q_{b,\text{res}} \quad \text{for } w_{by}/D_b < w_{by}/D_b$$

where $k = [(4/\pi)E_0/(1 - \nu^2)]$ and $q_{b,\text{res}}$ is the residual stress present at base prior to loading. The base stress-displacement response during the non-linear phase is modelled using Equation 2.13:

$$q_b = k \left( \frac{w_{by}}{D_b} \right)^{1-n} \left( \frac{w_b}{D_b} \right)^n + q_{b,\text{res}} \quad \text{for } 0.1 > w_{by}/D_b > w_{by}/D_b$$

where $n$ is the parabolic base stress-displacement response coefficient.

The implications of the work of Gavin and Lehane (2007) for DCIS piles is that the onset of degradation in base stiffness during loading should not occur until a normalised displacement $w_{by}/D_b \approx 1.5\%$ due to the high levels of prestress induced in the soil underlying the base by driving of the closed-ended steel installation tube; these aspects of base behaviour are investigated as part of the instrumented DCIS pile tests described in Chapters 5 and 6.
2.4.4 Soil layering

As mentioned previously in Section 2.4.2, the differences in diameter between a displacement pile and cone penetrometer result in variable zones of influence during penetration. This scale effect implies that the base resistance of the displacement pile will react more slowly to stratigraphic changes in comparison to the smaller CPT cone resistance $q_c$ measurement, particularly during partial embedment in layered deposits. For example, Figure 2.21 shows the $q_c$ profiles of two cone penetrometers with nominal diameters of 36 mm and 250 mm respectively during penetration across a soft clay/dense sand interface at Kallo, Belgium (De Beer et al., 1979); the values of $q_c$ for the larger penetrometer are significantly reduced in comparison to the standard 36 mm penetrometer, suggesting that a greater depth of embedment is required to achieve a steady-state resistance. Therefore, appropriate averaging must be applied to a CPT $q_c$ profile to account for partial embedment of displacement piles in layered soils if accurate correlations between the base resistance at failure $q_{b,0.1D_b}$ and $q_c$ are to be achieved.

![Figure 2.21 - Measured CPT $q_c$ profiles for 36 mm and 250 mm diameter cone penetrometers at Kallo, Belgium (De Beer et al. 1979)](image)

Several expressions have been proposed for predicting the reduction in base resistance of a displacement pile due to partial embedment in a layered deposit. Meyerhof (1983), for example, suggested that full mobilisation of the steady-state base resistance in a strong soil layer underlying a weak stratum occurs only once the depth of embedment in the strong layer $Z_e$ exceeds $10D_b$. However, elastic analyses of penetrometer penetration by Vreugdhenhil et al.
(1994) show that $Z_s$ increases from $10D_b$ up to a maximum of $40D_b$ as the steady-state penetration resistance ratio $q_{b,w}/q_{b,s}$ of the weak and strong layers reduces from 0.5 to 0.05. The general consensus regarding penetration from strong to weak layers is that an embedment depth of only $2D_b$ is necessary to achieve steady-state resistance in the weak layer (Ahmadi and Robertson, 2005; Meyerhof, 1983; Vreugdenhil et al., 1994).

Numerical analyses of pile behaviour in layered soils by Xu and Lehane (2008) revealed that the resistance of a penetrometer partially embedded in a strong layer depends on the ratio of steady-state penetration resistance in both the strong and weak layers i.e. $\eta_{\text{min}} = q_{b,w}/q_{b,s}$, but penetration in a weak layer remains relatively independent of an overlying strong layer when the embedment $Z_w$ exceeds $2D_b$. These are in agreement with the findings by others mentioned previously. Xu and Lehane (2008) therefore proposed the following non-linear equation for predicting the variation in resistance ratio $\eta = q_b/q_{b,s}$ with distance $H$ from the penetrometer tip to the layer interface during penetration from a weak to strong layer:

$$\eta = \frac{q_b}{q_{b,s}} = \eta_{\text{min}} + (1 - \eta_{\text{min}}) \exp \left[ -\exp \left( A_1 + A_2 \frac{H}{D} \right) \right] \quad \text{for} \ 0.01 > \eta_{\text{min}} > 0.9 \quad 2.14$$

where

$$A_1 = -0.22 \ln \eta_{\text{min}} + 0.11 \leq 1.5$$

$$A_2 = -0.11 \ln \eta_{\text{min}} - 0.79 \leq -0.2$$

The validity of Equation 2.14 was further examined by Xu (2007) by performing a series of centrifuge-scale displacement pile installation tests in two- and three-layered soils. Profiles of base resistance during installation showed excellent agreement with those predicted by Equation 2.14, particularly during penetration in a strong layer overlying a weak layer.
2.4.5 CPT averaging techniques

Numerous CPT $q_c$ averaging methods have been proposed over the years, with the most popular CPT-based design approaches for estimating displacement pile base resistance in sand typically using one of the following three techniques:

1. Arithmetic average over a distance corresponding to $1.5D_b$ above and $1.5D_b$ below the base.
2. Geometric average over a distance of $8D_b$ above and $2D_b$ to $4D_b$ below the base depending on the variations in soil stratigraphy.
3. The complex Dutch averaging method (Schmertmann, 1978; Xu, 2007), summarised in Figure 2.22, where a weighted average of the minimum path $q_c$ values between $0.7D_b$ and $4D_b$ below the pile base and $8D_b$ above is obtained.

The sensitivity of the $q_c$ averaging methods was investigated by Xu and Lehane (2005) using a database of CPT profiles at 13 separate sites in which reliable measurements of displacement pile base resistance during loading were reported. Method performance, shown in Figure 2.23, was assessed by plotting the average cone resistance $q_{c,avg}$ against the corresponding local value at the base $q_{c,tip}$. While negligible differences between $q_{c,tip}$ and $q_{c,avg}$ were apparent for the majority of sites where the soil profiles were relatively homogeneous, regardless of averaging method, the highly-layered $q_c$ profile at Kallo (De Beer et al. 1979) resulted in $q_{c,avg} \leq q_{c,tip}$ for the geometric and Dutch methods. Therefore, the ability of CPT-based methods to estimate the base resistance of displacement piles in layered soils will be strongly influenced by the choice of averaging technique.

![Figure 2.22 - Dutch CPT cone resistance averaging method (Schmertmann 1978, Xu 2007)](image-url)
Figure 2.23 - Comparison of cone resistance at pile base with average cone resistance using (a) arithmetic, (b) geometric and (c) Dutch averaging methods (after Xu & Lehane 2005)
Xu and Lehane (2005) also examined the ability of the three $q_c$ averaging methods to estimate the base resistance profile of a 350 mm wide precast concrete pile reported by Lehane et al. (2003) during jacked installation in soft alluvial deposits in Perth, Australia. The pile was instrumented with strain gauges at the base to derive accurate measurements of base resistance during jacked installation. As illustrated in Figure 2.24, the Dutch method provided a superior estimate of the $q_b$ profile in comparison to the arithmetic and geometric averaging methods.

The ability of the three $q_c$ averaging methods to predict the base resistance of a 1 m diameter closed-ended pile during penetration across the interface of a two-layer soil deposit was examined by Xu (2007). The base resistance was derived using Equation 2.14 for the theoretical profile, with the upper weak layer having a constant resistance of 1 MPa and the underlying strong layer having a resistance of 10 MPa (i.e. $\eta_{\min} = 0.1$). As illustrated in Figure 2.25, the Dutch method once again provided the best estimate of base resistance in the layered deposit given by Equation 2.14.

The above studies demonstrate the capability of the Dutch $q_c$ averaging method in accounting for scale effects during penetration in layered deposits in comparison to the arithmetic and geometric methods. Given the highly-layered stratigraphy at several of the sites described in
Chapter 4 in which instrumented DCIS piles were installed and load tested to failure, a unique opportunity is available to further analyse the performance of the three $q_c$ averaging methods, in particular the Dutch method, in predicting the base resistance of DCIS piles. Further details are given in Chapters 5 and 6.

![Diagram of CPT averaging techniques with idealised base resistance profile during penetration in a two-layered soil deposit (after Xu 2007)](image)

**Figure 2.25 - Comparison of CPT averaging techniques with idealised base resistance profile during penetration in a two-layered soil deposit (after Xu 2007)**

### 2.4.6 Correlations with cone resistance

The relationship between the base resistance of a closed-ended displacement pile in sand at failure $q_{b,0.1D_b}$ and the arithmetic average cone resistance $q_{c,avg}$ over a zone of $1.5D_b$ above and below the pile base was examined by Chow (1997) using a database of 28 high-quality load tests on instrumented closed-ended steel and concrete piles in which residual base loads were measured with a reasonable degree of accuracy (although this has subsequently been disputed by others as described below). Interestingly, the sub-database of concrete piles contains 6 no. expanded-base DCIS piles (known as Franki piles) which were driven and load tested in the two-layered soil deposit at Kallo by De Beer et al. (1979). As shown in Figure 2.26, a systematic reduction in normalised base resistance $q_{b,0.1D_b}/q_{c,avg}$ with increasing pile diameter was evident which could be expressed as follows:
where $q_{c,avg}$ is the average cone resistance over a zone of $1.5D_b$ above and below the pile base and $D_{CPT}$ is the diameter of the cone penetrometer.

\[
\frac{q_{b,0.1D_b}}{q_{c,avg}} = 1 - 0.5\log\left(\frac{D_b}{D_{CPT}}\right) \geq 0.3
\]  

Equation 2.15 was subsequently incorporated into the Imperial College ICP-05 CPT-based design method (Jardine et al., 2005) for closed-ended displacement piles in sand (described in Section 2.5.2). The suggestion that normalised base resistance $q_{b,0.1D_b}/q_{c,avg}$ is diameter-dependent has proven controversial however. Randolph (2003) noted that Chow’s (1997) database contained several small-scale jacked piles ($D_b \leq 0.2$ m) which tend to mobilise higher $q_{b,0.1D_b}/q_{c,avg}$ values in comparison to driven piles due to the large jacking strokes and higher residual stresses present after installation. A re-examination of Chow’s (1997) database by White and Bolton (2005) revealed that the results of the driven pile tests at Drammen (Gregersen et al., 1973), Hoogzand (Beringen et al., 1979), Hunter’s Point (Briaud et al., 1989b) and Seattle (Gurtowski and Wu, 1984) had residual stresses in excess of 50% of the base resistance at failure which was considered unrealistic. Furthermore, when these effects are corrected, both Randolph (2003) and White and Bolton (2005) advocated that $q_{b,0.1D_b}/q_{c,avg}$ was independent of pile diameter.

Figure 2.26 - Variation in normalised base resistance (using the arithmetic averaging method) with pile base diameter (Chow 1997)
The findings by Randolph (2003) and White and Bolton (2005) prompted Xu and Lehane (2005) and Xu (2007) to re-assess Chow’s (1997) database using the Dutch averaging method for \( q_{c,\text{avg}} \) due to its superior ability for estimating the base resistance of displacement piles in layered soils as discussed previously. Unlike Chow’s (1997) study however, driven and jacked piles were considered separately, primarily due to the higher magnitude of residual base stresses typically observed after jacked installation, as noted by Randolph (2003). The findings, illustrated in Figure 2.27, show that \( q_{b,0.1D_b}/q_{c,\text{avg}} \) for the driven piles was relatively constant (≈ 0.6) and independent of pile diameter; the only exceptions to this trend were the test piles at Baghdad (Altaee et al., 1992) and Ogeechee River (Vesic 1970) where erroneous interpretations of residual loads were suspected. A definitive assessment of \( q_{b,0.1D_b}/q_{c,\text{avg}} \) for jacked piles was not pursued due to the limited number of tests in the dataset.

![Figure 2.27 - Variation in normalised base resistance (using the Dutch averaging method) with pile base diameter (Xu 2007)](image)

Therefore, Lehane et al. (2005c) recommend the use of Equation 2.16 for closed-ended driven piles as part of the University of Western Australia UWA-05 design method for displacement piles in siliceous sand (described in more detail in Section 2.5.2):

\[
\frac{q_{b,0.1D_b}}{q_{c,\text{avg}}} = 0.6
\]

where \( q_{c,\text{avg}} \) is the average cone resistance derived using the Dutch \( q_c \) averaging method.
As DCIS piles are constructed by driving a closed-ended steel tube (which remains closed-ended after concreting due to the sacrificial driving plate), it might be expected that the normalised base resistance at failure $q_{b,0.1D_b}/q_{c,avg}$ for a DCIS pile would be similar to that of a traditional preformed displacement pile. The fact that Franki piles were included in the ICP-05 and UWA-05 base resistance databases by Chow (1997) and Xu (2007) respectively is also encouraging. However, as differences in construction method exist between a Franki pile and the temporary-cased non-expanded base DCIS pile investigated in this thesis, it is necessary to examine the relationship between $q_{b,0.1D_b}$ and $q_{c,avg}$ for temporary-cased non-expanded base DCIS piles during the instrumented pile tests in both layered and homogenous sand deposits described in Chapters 5 and 6 respectively.
2.5 DISPLACEMENT PILE DESIGN METHODS IN SAND

2.5.1 Introduction
The strong relationship between the CPT cone resistance $q_c$ and the behaviour of a closed-ended displacement pile (in terms of both shaft and base resistance) has led to the development of numerous CPT-based design methods for estimating displacement pile capacity in sand. This section reviews the most popular CPT-based design approaches, which are categorised into (i) simplified ‘alpha’ methods and (ii) recent methods. Finally, the performance of these methods in predicting pile capacity is discussed.

2.5.2 Simplified ‘alpha’ CPT methods
CPT-based design methods in which single, empirically-derived, alpha coefficients are used to relate cone resistance $q_c$ to local shear stress at failure $\tau_{sf}$ and base resistance $q_b$ of a displacement pile in sand are known as ‘alpha’ methods (Schneider et al. 2008). The following simplified alpha methods are summarised in this section:

- LCPC-82 (Bustamante and Gianeselli, 1982)
- EF-97 (Eslami and Fellenius, 1997)
- VI-86 (Van Impe, 1986)

Design equations and parameters specified by each method for calculating the shaft and base resistance are summarised in Tables 2.1 and 2.2 respectively.

The local shear stress at failure is expressed in general form as:

$$\tau_{sf} = \frac{q_c}{\alpha_s} \leq \tau_{s,lim}$$

2.17

where $\alpha_s$ is the shaft friction coefficient and $\tau_{s,lim}$ is the limiting value of shear stress. The LCPC-82 and VI-86 methods specify values of $\alpha_s$ which are a function of cone resistance and pile type. Interestingly, the LCPC-82 method suggests values of $\alpha_s$ for DCIS piles which are double those stipulated for preformed concrete piles; this implies that, for a given $q_c$ value, the shaft resistance of a DCIS pile in sand is 50% less than an equivalent preformed concrete pile. The EF-97 method uses the effective cone resistance $q_E = q_t - u_2$, where $q_t$ is the cone resistance corrected for end effects and $u_2$ is the measured pore pressure response behind the cone tip during penetration) to derive values of $\alpha_s$ and does not consider pile type. Limiting
values of shear stress $\tau_{s,\text{lim}}$, which also depend on the magnitude of $q_c$ (see Table 2.1), are specified for the LCPC-82 method only.

The base resistance is derived using the following equation:

$$q_b = \alpha_b q_{c,\text{avg}}$$  \hspace{2cm} (2.18)

where $\alpha_b$ is the base resistance coefficient. The average cone resistances at the pile base $q_{c,\text{avg}}$ for the LCPC-82 and EF-97 methods are obtained using the arithmetic $q_c$ and geometric $q_E$ averaging techniques respectively (described in detail in Section 2.4.5). On the other hand, the VI-86 method adopts a highly cumbersome analytical procedure, described in detail by De Beer (1971a; 1971b; 1971c), to derive the representative $q_{c,\text{avg}}$ from the measured $q_c$ profile. The base coefficient $\alpha_b$ varies with $q_c$ for the LCPC-82 method, but a constant value $\alpha_b = 1$ is assumed for the EF-97 and VI-86 methods.

<table>
<thead>
<tr>
<th>Methods</th>
<th>Design equations</th>
<th>$q_c$ range (MPa)</th>
<th>Shaft coefficient $\alpha_s$</th>
<th>Limiting shaft friction $q_{s,\text{lim}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>VI-86</td>
<td>$\tau_{sf} = \frac{q_c}{\alpha_s}$</td>
<td>$&lt; 10$</td>
<td>150</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>$\zeta_f = 1.0$ for concrete closed-ended piles</td>
<td>$10 - 20$</td>
<td>$100 + 5q_c$</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$&gt; 20$</td>
<td>200</td>
<td>N/A</td>
</tr>
<tr>
<td>EF-97</td>
<td>$\tau_{sf} = \frac{q_E}{\alpha_s}$</td>
<td>Silty sand</td>
<td>100</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Sand</td>
<td>250</td>
<td>N/A</td>
</tr>
<tr>
<td>LCPC-82</td>
<td>$\tau_{sf} = \frac{q_c}{\alpha_s}$</td>
<td>Driven cast-in-situ piles (category 1B)</td>
<td>$&lt; 5$</td>
<td>150</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$5 - 12$</td>
<td>200</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$&gt; 12$</td>
<td>300</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Driven precast concrete piles (category 2A)</td>
<td>$&lt; 5$</td>
<td>60</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$5 - 12$</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$&gt; 12$</td>
<td>150</td>
</tr>
</tbody>
</table>

Note: $\zeta_f$ = factor for installation method and shaft roughness; Effective cone resistance $q_E = q_t - u_2$
Table 2.2 - Base resistance equations for simplified alpha CPT methods (adapted from Schneider et al. 2008)

<table>
<thead>
<tr>
<th>Methods</th>
<th>Design equations</th>
<th>$q_b$ range (MPa)</th>
<th>Base coefficient $\alpha_b$</th>
</tr>
</thead>
<tbody>
<tr>
<td>LCPC-82</td>
<td>$q_b = \alpha_b q_{c,avg}$</td>
<td>$&lt; 5$</td>
<td>0.5</td>
</tr>
<tr>
<td></td>
<td>Driven cast-in-situ &amp; driven precast concrete piles (Category 2)</td>
<td>$5 - 12$</td>
<td>0.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$&gt; 12$</td>
<td>0.4</td>
</tr>
<tr>
<td>EF-97</td>
<td>$q_b = \alpha_b q_{E,geomean}$</td>
<td>Silty sand &amp; Sand</td>
<td>1.0</td>
</tr>
<tr>
<td>VI-86</td>
<td>$q_b = \lambda \alpha_b \epsilon_b q_{c,avg}$</td>
<td>Sand</td>
<td>1.0</td>
</tr>
</tbody>
</table>

$\lambda$ = 1 for DCIS piles
$\epsilon_b$ = 1 for in sand

Note: $\lambda$ = reduction factor for soil relaxation; $\epsilon_b$ = scaling coefficient

2.5.3 Recent methods

The ability of the simplified alpha methods to capture the several complex phenomena influencing displacement pile behaviour in sand using a single parameter is highly questionable (Lehane, 2008). The results of various high-quality tests on displacement piles in sand, e.g. the ICP tests at Labenne and Dunkirk, have therefore led to the development of several modern CPT-based design methods for predominantly offshore applications. These methods are a significant improvement on the simplified alpha methods described above, as they have the potential to account for factors such the degree of soil displacement and friction fatigue during installation, interface dilation at the pile shaft during loading, the reduction in shaft resistance during tensile loading and the plugging behaviour of open-ended piles. The updated methods, which have been incorporated into the latest version of the American Petroleum Institute’s (API) recommended practice for fixed offshore structures (API, 2007), include:

- Fugro-05 (Kolk et al., 2005a)
- ICP-05 (Jardine et al., 2005)
- NGI-05 (Clausen et al., 2005)
- UWA-05 (Lehane et al., 2005c; 2007b)

A summary of the shaft capacity design equations for each method is provided in Table 2.3. Key aspects regarding the shaft capacity calculations for the four methods include:
The Fugro-05, ICP-05 and UWA-05 methods express the local shear stress at failure \( \tau_{sf} \) in general form using Equation 2.6. A relative density factor \( F_{Dr} \), derived from \( q_c \), is used to estimate \( \tau_{sf} \) for the NGI-05 method.

Dilation-related increases in radial stress during loading \( \Delta \sigma'_{rd} \) are ignored by the Fugro-05 method, presumably due to the large-diameter pile database from which the method is formulated. The ICP-05 and UWA-05 derive \( \Delta \sigma'_{rd} \) using Equation 2.7, with \( \Delta y = 0.02 \text{ mm} \), based on twice the typical average roughness of a steel pile.

In the ICP-05 and UWA-05 methods, the interface friction angle \( \delta_{cv} \) varies with mean effective diameter \( D_{50} \) (as shown in Figure 2.13a), while a constant value \( \delta_{cv} = 29^\circ \) is assumed for the Fugro-05 method, irrespective of particle diameter and interface roughness.

Friction fatigue is modelled in the NGI-05 method using a triangular shear stress distribution, while the other methods use normalised distance from the base \( h/R \) (or \( h/D \)) raised to a negative power which controls the rate of degradation in \( \sigma'_{rc} \).

All methods apply a reduction factor to \( \tau_{sf} \) for tensile loading.

The base resistance equations for the four methods are summarised in Table 2.4. Key differences in method formulation include:

- The base resistance at failure \( q_{b,0.1Db} \) is defined at a base displacement corresponding to 10% of the pile base diameter \( D_b \) for the Fugro-05 and UWA-05 methods, while the ICP-05 and NGI-05 methods define \( q_{b,0.1Db} \) at a head displacement equivalent to 10% \( D_b \). Significant differences in the definition of capacity may therefore occur for piles where elastic compression is substantial (Xu et al., 2008).

- The Fugro-05 and ICP-05 methods use the arithmetic average to derive \( q_{c,avg} \), while the UWA-05 method uses the Dutch method (these averaging techniques were previously described in detail in Section 2.4.5). The NGI-05 method uses the local cone resistance at the pile base \( q_{c,tip} \).

- As discussed in Section 2.4.6, the ICP-05 method advocates that normalised base resistance \( q_{b,0.1Db}/q_{c,avg} \) varies with pile diameter in accordance with Equation 2.15, while a constant \( q_{b,0.1Db}/q_{c,avg} = 0.6 \) for driven piles is assumed for the UWA-05 method. In the Fugro-05 method, \( q_{b,0.1Db}/q_{c,avg} \) is a function of the square root of \( q_c \), resulting in ratios greater than unity when \( q_{c,avg} < 7.2 \text{ MPa} \) (Xu et al., 2008). The NGI-05 method assumes \( q_{b,0.1Db}/q_{c,tip} \) reduces with increasing relative density \( D_r \).
### Table 2.3 - Shaft resistance equations for recent CPT methods (adapted from Schneider et al. 2008)

<table>
<thead>
<tr>
<th>Methods</th>
<th>Design equations</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Fugro-05</strong></td>
<td>$\tau_{sf} = 0.08q_{c} \left( \frac{\sigma_{v0}}{p_{ref}} \right)^{0.05} \left( \frac{h}{R} \right)^{-0.90}$ (compression loading for $h/R \geq 4$)</td>
</tr>
<tr>
<td></td>
<td>$\tau_{sf} = 0.08q_{c} \left( \frac{\sigma_{v0}}{p_{ref}} \right)^{0.05} \left( \frac{h}{4R} \right)^{-0.90}$ (compression loading for $h/R \leq 4$)</td>
</tr>
<tr>
<td></td>
<td>$\tau_{sf} = 0.045q_{c} \left( \frac{\sigma_{v0}}{p_{ref}} \right)^{0.15} \left[ \max \left( \frac{h}{R} \right) \right]^{-0.85}$ (tension loading)</td>
</tr>
<tr>
<td><strong>ICP-05</strong></td>
<td>$\tau_{sf} = a \left[ 0.29bq_{c} \left( \frac{\sigma_{v0}}{p_{ref}} \right)^{0.11} \left[ \max \left( \frac{h}{D} \right) \right]^{-0.38} + \Delta \sigma_{rd} \right] \tan \delta_{t}$</td>
</tr>
<tr>
<td>$a = 1.0$ for closed-ended piles</td>
<td></td>
</tr>
<tr>
<td>$b = 0.8$ for piles in tension and 1.0 for piles in compression</td>
<td></td>
</tr>
<tr>
<td><strong>NGI-05</strong></td>
<td>$\tau_{sf} = \frac{1}{2} L p_{ref} F_{D_c} F_{f_{sig}} F_{u_{q}} F_{load} F_{mat} \geq \tau_{s,min}$</td>
</tr>
<tr>
<td>$F_{D_c} = 2.1 \left( D_{c} - 0.1 \right)^{0.7}$</td>
<td></td>
</tr>
<tr>
<td>$D_{c} = 0.4 \ln \left( q_{c1N} / 22 \right)$ (may be greater than 1.0)</td>
<td></td>
</tr>
<tr>
<td>$F_{f_{sig}} = \left( \sigma_{v0} / p_{ref} \right)^{0.25}$</td>
<td></td>
</tr>
<tr>
<td>$F_{u_{q}} = 1.6$ for driven closed-ended</td>
<td></td>
</tr>
<tr>
<td>$F_{load} = 1.0$ for tension and 1.3 for compression</td>
<td></td>
</tr>
<tr>
<td>$F_{mat} = 1.0$ for steel and 1.2 for concrete</td>
<td></td>
</tr>
<tr>
<td>$\tau_{s,min} = 0.1 \sigma_{v0}$</td>
<td></td>
</tr>
<tr>
<td><strong>UWA-05</strong></td>
<td>$\tau_{sf} = \frac{f_{l}}{f_{c}} \left[ 0.03 q_{c} \left[ \max \left( \frac{h}{D} \right) \right]^{-0.5} + \Delta \sigma_{rd} \right] \tan \delta_{t}$</td>
</tr>
<tr>
<td>$f_{l}/f_{c} = \text{ratio of tension to compression capacity (equal to 1 for compression and 0.75 in tension)}$</td>
<td></td>
</tr>
</tbody>
</table>

Note: $\tau_{sf} =$ local shear stress at failure; $\delta_{t} =$ interface friction angle at failure; $p_{ref} =$ 100 kPa; $L =$ pile length; $z =$ element depth; $h =$ height above pile tip; $R =$ pile radius = $D/2$; $\Delta \sigma_{rd} =$ change in radial displacement during loading; $G =$ 185$q_{c1N}^{-0.17}$ = operational level of shear modulus; $\Delta y =$ 2$R_{y} =$ 0.02 mm = radial displacement during pile loading; and $R_{d} =$ average roughness of the pile.

### Table 2.4 - Base resistance equations for recent CPT methods (adapted from Xu et al. 2008)

<table>
<thead>
<tr>
<th>Methods</th>
<th>End condition</th>
<th>Design equations</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Fugro-05</strong></td>
<td>Closed-ended</td>
<td>$q_{b,0.1 Db} = 8.5 \left( \frac{p_{ref}}{q_{c,avg}} \right)^{0.5}$</td>
</tr>
<tr>
<td><strong>ICP-05</strong></td>
<td>Closed-ended</td>
<td>$q_{b,0.1 Db} / q_{c,avg} = \max \left[ 1 - 0.5 \log \left( \frac{D_{b}}{D_{CPT}} \right) \right]$</td>
</tr>
<tr>
<td><strong>NGI-05</strong></td>
<td>Closed-ended</td>
<td>$q_{b,0.1 Db} / q_{c,tip} = 0.8 \frac{1}{1 + D_{t}^{2}}$</td>
</tr>
<tr>
<td><strong>UWA-05</strong></td>
<td>Closed-ended</td>
<td>$q_{b,0.1 Db} / q_{c,Dutch} = 0.6$</td>
</tr>
</tbody>
</table>

Notes: $D_{b} =$ pile base diameter; $D_{CPT} =$ cone diameter; $q_{c,avg} =$ $q_{c}$ averaged $\pm 1.5D_{b}$ over pile tip level; $q_{c,Dutch} =$ $q_{c}$ averaged using Dutch averaging technique; $D_{t} = 0.4 \ln (q_{c,tip} / 22(\sigma_{v0} p_{ref}))$ expressed as a decimal;

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2.5.4 Prediction performance of CPT-based design methods

Despite the popularity of CPT-based design methods, assessments of their ability to predict the total capacity of displacement piles in sand have been rather limited. Briaud and Tucker (1988) used a database of 24 driven piles in sand to assess the prediction performance of thirteen separate displacement pile design methods (including six CPT-based methods). The results of the study showed that the LCPC-82 method (Bustamante and Gianeselli 1982) provided significantly improved estimates of displacement pile capacity in sand in comparison to traditional design methods.

The advances in knowledge of the mechanisms governing displacement pile behaviour in sand over the past 25 years have led to the development of the four new CPT-based design methods discussed in Section 2.5.3. An assessment of the base capacity prediction performance of the four methods was undertaken by Xu et al. (2008) using the database of closed-ended displacement piles from which the UWA-05 method was formulated. Method performance was evaluated using the average \( \mu \) and coefficient of variation \( \text{COV} = \sigma/\mu \) (where \( \sigma \) is standard deviation) of the ratio of calculated to measured base resistance \( Q_{b,c}/Q_{b,m} \). The results of the study, summarised in Table 2.5, show that the UWA-05 method provided the best estimate of base resistance for closed-ended displacement piles in sand, while the Fugro-05 method over-predicted \( q_{b,0.1D_b} \) by 42 % on average.

The performance of the four recent CPT-based methods and two simplified alpha methods (EF-97 and LCPC-82) in predicting total compressive displacement pile capacity \( Q_t \) was assessed by Schneider et al. (2008) using the UWA database containing 32 compressive load tests on concrete and steel closed-ended displacement piles in siliceous sand. Table 2.6 summarises the predictive performance of the six methods; while the COVs for the methods are relatively similar, the NGI-05 method provided the best average estimate of total capacity \( (Q_t/Q_m = 1.02) \). The performances of the ICP-05 and UWA-05 methods were almost identical, albeit slightly conservative \( (Q_t/Q_m \approx 0.90) \) while the simplified alpha and Fugro-05 methods over-predicted total capacity.

The prediction performance of the VI-86 method was not reported in the above studies and is therefore assessed in Chapter 7 using a database of DCIS test piles with adjacent \( q_c \) profiles.
Table 2.5 - Base capacity predictive performance of CPT methods for closed-ended displacement piles in sand (Xu et al. 2008)

<table>
<thead>
<tr>
<th>$Q_b/Q_{b,m}$</th>
<th>Fugro-05</th>
<th>ICP-05</th>
<th>NGI-05</th>
<th>UWA-05</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average $\mu$</td>
<td>1.42</td>
<td>1.02</td>
<td>1.10</td>
<td>1.00</td>
</tr>
<tr>
<td>COV</td>
<td>0.31</td>
<td>0.16</td>
<td>0.27</td>
<td>0.12</td>
</tr>
</tbody>
</table>

Table 2.6 - Total capacity predictive performance of CPT methods for closed-ended displacement piles in sand (Schneider et al. 2008)

<table>
<thead>
<tr>
<th>$Q/Q_m$</th>
<th>LCPC-82</th>
<th>EF-97</th>
<th>Fugro-05</th>
<th>ICP-05</th>
<th>NGI-05</th>
<th>UWA-05</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average $\mu$</td>
<td>1.08</td>
<td>1.24</td>
<td>1.16</td>
<td>0.92</td>
<td>1.02</td>
<td>0.90</td>
</tr>
<tr>
<td>COV</td>
<td>0.32</td>
<td>0.25</td>
<td>0.33</td>
<td>0.30</td>
<td>0.33</td>
<td>0.29</td>
</tr>
</tbody>
</table>
2.6 RESIDUAL LOAD DEVELOPMENT DURING CURING

2.6.1 Introduction

The load developed in a pile between its installation and a subsequent load test is often referred to as ‘residual load’, and there are a number of sources of residual loads depending on the pile type and installation process. As discussed in Section 2.2, the dominant cause of residual load for preformed piles (i.e. precast concrete piles and steel piles) is the reversal of shaft resistance along the pile as it attempts to rebound after hammer impact during driving. A large number of studies have examined this particular phenomenon and several methods for estimating the quantity of residual load present have been developed, e.g. Briaud and Tucker (1984), Poulos (1987) and Fellenius (2002). Unfortunately, an exclusive association has developed between residual load and the driving process in industry among those who do not appreciate that there are other sources of residual load; some of these sources are relevant to cast-in-place piles. Given that several facets of both driven and cast-in-situ pile behaviour are relevant to DCIS piles, it is worthwhile undertaking a review of residual load development in cast-in-situ piles, in particular the interpretation of such loads based on the strain and temperatures monitored during curing in several of the instrumented DCIS piles in Chapters 5 and 6.

Cast-in-situ piles must cure in the ground (unlike preformed piles) and will therefore undergo changes in volume as the concrete sets and cures. In addition, the installation of a pile, regardless of type, will result in disturbance to the surrounding soil (from either lateral or vertical soil movements) and the generation of excess pore pressures in cohesive soils which will induce downdrag on the pile as they dissipate (Fellenius, 2002). Therefore, the assumption of a load-free pile as the initial condition at the time of a load test is questionable. A limited number of studies have been conducted in recent years into the processes which occur within the concrete and soil after installation of a cast-in-situ pile to gain a better understanding of the development of residual loads. The level of detail of these studies varies from basic two-point concrete strain measurements (i.e. immediately post-installation and just before the load test) to more detailed monitoring of strain and temperature over part or all of this period. This section aims to review these studies which pertain to different cast-in-situ pile types in a systematic way, along with current methods of interpreting residual load present in a pile prior to conducting a load test from measured strains.
2.6.2 Processes leading to residual load development in cast-in-situ piles

Despite the assumption that cast-in-situ piles are not prone to residual load effects, a limited number of studies have investigated the strain and temperature behaviour within such piles. A summary of the scope of these tests is provided in Table 2.7, which includes details of pile type, pile dimensions, and duration of curing, as well as highlighting the various processes for which measurements were made within the concrete and soil. This section aims to describe each process in terms of strain and temperature (where relevant) as these can be measured with relative ease using instrumentation incorporated in the pile prior to casting (reference can be made to Table 2.7 throughout).

The most popular type of instrumentation for measuring strain within cast-in-situ piles are vibrating wire strain gauges due to their ease of installation and long-term reliability (Hayes and Simmonds, 2002) and these were used by the majority of the studies reviewed in this section. The gauges measure total strains which comprise of mechanical strains in the pile and thermal strains due to temperature variations. As the variation in temperature during initial set and hydration can be large, a number of studies have applied a temperature correction to the measured total strains in order to eliminate any thermal-related effects (these studies are highlighted in Table 2.7 under the ‘thermal strains’ column). Depending on the choice of coefficient of thermal expansion (CTE), the derived thermal strains can be significantly large. As a consequence of this, the correction for temperature effects can result in a complete reversal in strain behaviour (i.e. changing from total tensile strain to a compressive mechanical strain, and vice versa). This has led to contradicting early-age strain behaviour profiles in the literature, as several studies report total (i.e. uncorrected) strain profiles while others present temperature-corrected mechanical profiles.
Table 2.7 - Summary of published data on strain behaviour in cast-in-situ piles

<table>
<thead>
<tr>
<th>Reference</th>
<th>Location</th>
<th>Pile type</th>
<th>Length (m)</th>
<th>Diameter (m)</th>
<th>Temperature</th>
<th>Initial set</th>
<th>Curing strains</th>
<th>Thermal strains</th>
<th>Curing period (days)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pennington (1995)</td>
<td>Bangkok, Thailand</td>
<td>Bored cast-in-situ</td>
<td>36, 51</td>
<td>1.5</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>&gt; 28</td>
</tr>
<tr>
<td>Walter et al. (1997)</td>
<td>Bayfield, Canada</td>
<td>Drilled shaft</td>
<td>22</td>
<td>0.91</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>10</td>
</tr>
<tr>
<td>Kim et al. (2004)</td>
<td>Texas, USA</td>
<td>Auger cast-in-place</td>
<td>19</td>
<td>0.46</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>18</td>
</tr>
<tr>
<td>Vipulanandan et al. (2007)</td>
<td>Texas, USA</td>
<td>Auger cast-in-place</td>
<td>10</td>
<td>0.76</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>7</td>
</tr>
<tr>
<td>Farrell &amp; Lawler (2008)</td>
<td>Dublin, Ireland</td>
<td>Continuous flight auger</td>
<td>12.3</td>
<td>0.45</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>3</td>
</tr>
<tr>
<td>Fellenius et al. (2009) / Kim et al. (2011)</td>
<td>Shinho, South Korea</td>
<td>Post-grouted concrete cylinder pile</td>
<td>56</td>
<td>0.6</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>&gt; 40</td>
</tr>
<tr>
<td></td>
<td>Myeongji, South Korea</td>
<td>Post-grouted concrete cylinder pile</td>
<td>31</td>
<td>0.6</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>&gt; 40</td>
</tr>
<tr>
<td></td>
<td>Vancouver, Canada</td>
<td>Bored cast-in-situ</td>
<td>74.5</td>
<td>2.6</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>&gt; 35</td>
</tr>
<tr>
<td>Siegel &amp; McGillivray (2009)</td>
<td>Rincon, USA</td>
<td>Auger cast-in-place</td>
<td>9.15</td>
<td>0.457</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>103</td>
</tr>
<tr>
<td>Lam &amp; Jefferies (2011)</td>
<td>Stratford, UK</td>
<td>Bored cast-in-situ</td>
<td>27</td>
<td>1.2</td>
<td>✓</td>
<td>✓</td>
<td></td>
<td></td>
<td>28</td>
</tr>
<tr>
<td>Lin &amp; Lim (2011)</td>
<td>Hawaii, USA</td>
<td>Drilled shaft</td>
<td>39.3</td>
<td>1.65</td>
<td>✓</td>
<td></td>
<td></td>
<td></td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Hawaii, USA</td>
<td>Drilled shaft</td>
<td>18.3</td>
<td>2.44</td>
<td>✓</td>
<td></td>
<td></td>
<td></td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Hawaii, USA</td>
<td>Drilled shaft</td>
<td>15.24</td>
<td>3.2</td>
<td>✓</td>
<td></td>
<td></td>
<td></td>
<td>-</td>
</tr>
</tbody>
</table>
2.6.2.1 Initial concrete set and hydration

Once a cast-in-situ pile is constructed, the mixture of the cement binder and water results in an exothermic chemical reaction. The level of heat generated during hydration varies with the properties of the binder, the pile width/diameter, the ambient temperature of the mix during casting and the ambient temperature of the soil in which the concrete is cast. As the reaction takes place, the hydration temperatures will continue to rise for several hours after casting, and the concrete will begin to harden or ‘set’ during this period. Initial set is assumed to be complete at peak temperature (Neville and Brooks, 1987).

Peak pile temperatures during initial set and hydration were reported by Pennington (1995), Walter et al. (1997), Fellenius et al. (2009), Kim et al. (2011) and Lin and Lim (2011). The highest temperatures were observed by Fellenius et al. (2009) in two 600 mm diameter precast post-grouted cylindrical piles with peaks in the region of 85°C while Pennington (1995) reported temperatures of up to 75 °C in 1.5 m diameter bored piles in Bangkok. The magnitude of peak temperature will inevitably depend on the concrete (or grout) mix. The reported temperature data from the literature was used to produce Figure 2.28 which shows that the duration of initial set (i.e. the time required to reach peak temperature) is a near-linear function of pile shaft diameter $D_s$ (over a wide range of $D_s$ values). As hydration temperatures were not reported in the studies by Farrell and Lawler (2008) and Siegel and McGillivray (2009), the duration between casting and peak temperature has been assumed to correspond to the peak strain during this period in Figure 2.28. This figure serves as a useful means of estimating the duration of initial set for cast-in-place piles.

![Figure 2.28 - Variation in time to peak temperature with shaft diameter](image)
Studies in which total strains during initial set were reported, and were uncorrected for effects of temperature, include Pennington (1995), Siegel and McGillivray (2009), Fellenius et al. (2009) and Lam and Jefferis (2011). All of these studies measured compressive total strains as the hydration temperatures increased. On closer examination of the strain and temperature profiles in a driven precast post-grouted cylindrical pile by Fellenius et al. (2009), the largest compressive strains (≈ 150 με) developed at sections of the pile where the hydration temperatures were greatest. Fellenius et al. (2009) concluded that the compressive strains were a result of the steel vibrating wire strain gauges attempting to elongate at a faster rate than the surrounding concrete, due to the difference in coefficient of thermal expansion (CTE) of the two materials (i.e. a net value of ≈ 2.2 με/°C according to Fellenius et al. 2009). Siegel and McGillivray (2009) observed a peak compressive strain of 100 με in a 457 mm diameter auger cast-in-place (ACIP) pile approximately 10 hours after casting. Unfortunately, hydration temperatures were not reported and thus the relationship between compressive strains and temperature was unknown.

The level of thermal strain $\varepsilon_{\text{thermal}}$ present in a pile at a particular instance after casting can be determined using Equation 2.19 (Pennington, 1995):

$$\varepsilon_{\text{thermal}} = \alpha \Delta T = \alpha (T_1 - T_0)$$  \hspace{1cm} 2.19

where $\alpha$ is the CTE of the material in question, $\Delta T$ is the difference in temperature, $T_0$ is the temperature at time of casting and $T_1$ is the current temperature. Despite the simplicity of Equation 2.19, inconsistency exists in the literature regarding the choice of $\alpha$ for correction. As vibrating wire strain gauges are traditionally composed of a steel wire enclosed in steel casing, the $\alpha$ value of the gauge is assumed to be between 10 με/°C and 12 με/°C (based on steel). Pennington (1995) and Farrell and Lawler (2008) presented strain profiles during initial set and hydration which were corrected for thermal strains. The study by Pennington (1995) calculated thermal strains using Equation 2.2, based on a CTE value of 10.8 με/°C. To obtain the ‘mechanical’ strain in the pile during this period, the derived thermal strains from Equation 2.2 were subtracted from the compressive total strains, resulting in a tensile strain response at peak temperature. A CTE value of 10.8 με/°C was also used by Farrell and Lawler (2008) to correct strains during initial set. However, the peak tensile strains were considerably less in comparison to those observed by Pennington (1995). Fellenius et al. (2009) back-calculated a net CTE value of 1.7 με/°C from strain and temperature data from concrete pile
specimens, which is broadly in agreement with the theoretical difference in CTE values of concrete and steel of 2.2 με/°C.

The influence of the correction for thermal strains on the strain profile during initial set and hydration is significant. If uncorrected, the strain response is compressive. However, the studies in which the correction was applied show a tensile mechanical response. Unfortunately, the dearth of detailed studies on these factors for cast-in-situ piles prevents any comprehensive conclusions to be made regarding the most appropriate CTE value to use.

2.6.2.2 Curing and strength development
After hydration temperatures have peaked, the pile will begin to cool and develop further strength. This is characterised by a gradual reduction in temperature with time and a change in volume (and hence strain) within the pile due to either shrinkage or swelling effects. The durations for full decay of hydration temperatures after casting were in the region of 8-10 days for the studies by Pennington (1995) and Fellenius et al. (2009). However, Fellenius et al. (2009) also reported the temperature profile within a 2.6 m diameter bored pile in Vancouver, Canada where a period of 30 days was required for pile temperatures to stabilise.

The author has compiled published data to show that the time taken for pile temperatures to reduce from peak temperature \( T_{\text{peak}} \) to \( T_0 + 0.5(T_{\text{peak}} - T_0) \) is once again a function of pile shaft diameter as shown in Figure 2.29. An attempt to correlate the time between \( T_{\text{peak}} \) and \( T_0 + 0.1(T_{\text{peak}} - T_0) \) was not as fruitful, as measurements were not always carried through to the latter temperature, and small discrepancies between \( T_0 \) measured before casting and after curing were influential. Hydration temperatures will have a prolonged effect on the strain profile (in terms of thermal strains) of large diameter piles during curing. Figure 2.28 and Figure 2.29 combined serve as a useful means of estimating the time to the end of curing, with a view to isolating the effect of other possible residual load processes, such as dragload.
As the pile cools, it will continue to absorb moisture for hydration. The process of promoting concrete hydration is referred to as curing (Neville and Brooks, 1987). For conventional concrete structures, curing takes place within steel or plywood formwork under carefully controlled conditions, with moisture provided by spraying or steam. However, for cast-in-situ piles, the surrounding soil must act as formwork and moisture is free to migrate across the soil-pile interface. If the pile is constructed in saturated soil, a continuous supply of moisture will be available for hydration which may cause the pile to elongate or swell. Pennington (1995), Kim et al. (2004), Siegel and McGillivray (2009), Fellenius et al. (2009) and Lam and Jefferis (2011) reported tensile total strains after hydration temperatures had stabilised. The magnitudes of elongation varied from study to study, although Fellenius et al. (2009) observed tensile strains as large as 320 με. In order to confirm whether the swelling was related to moisture absorption, Fellenius et al. (2009) conducted a comprehensive laboratory study on 2.0 m long concrete pile specimens which were initially cast and cured in a dry environment for 154 days. The strain response during this period was generally compressive, although relaxation of the specimens was observed for about 4 days after peak hydration temperature. After 154 days, the specimens were placed in water which resulted in a significant tensile strain response as the concrete began to swell. Thus, a tensile strain response in piles during curing may be a consequence of swelling due to moisture absorption. Another possible source of tensile strains is internal restraint in the concrete due to drying shrinkage (Hayes and Simmonds, 2002).

**Figure 2.29 - Variation in time between peak and 50 % of excess temperature with shaft diameter**
While the uncorrected total strain profiles in the literature show a trend of tensile strain development during curing, the temperature-corrected strain behaviour during this period is less clearcut. The correction for thermal effects results in strain profiles which may be either tensile or compressive, depending on the magnitude of the thermal strain derived using Equation 2.19. For example, Pennington (1995) presents the corrected strain profile during curing where, despite the elimination of thermal effects, the test pile remained in a tensile state. In contrast, the temperature-corrected strain profile within an continuous flight auger (CFA) pile in glacial till by Farrell and Lawler (2008) revealed a compressive state (≈ 100 με) approximately 3 days after casting, although temperatures were most likely still reducing and further reductions in compressive strain may have occurred had the pile not been tested so soon after installation. Walter et al. (1997) also reported net compressive strains in a drilled shaft after cooling. As hydration temperatures diminish with time during curing however, the temperature differential (ΔT) component in Equation 2.19 reduces. Therefore, the thermal strain correction during the latter stages of curing will tend to zero.

2.6.2.3 Pile installation and soil consolidation effects

The installation of a pile results in disturbance to the surrounding soil, irrespective of pile type (Fellenius, 2002), although this category is most relevant to displacement pile types. Preformed piles may develop residual loads due to elastic rebound during driving (as mentioned previously). In addition, the generation of excess pore pressures during pile construction in cohesive deposits may lead to consolidation of the soil and the resulting settlement will induce compressive loads in the pile over time (due to negative skin friction). Such behaviour will occur independently of and in parallel with the processes within the concrete as described above. The development of such loads is characterised by an increase in compressive strain with time at various sections of the pile. To highlight the long-term increase in residual loads due to consolidation settlement, Siegel and McGillivray (2009) continued to monitor the total strain behaviour within an ACIP pile in clay after the internal concrete processes had ceased. Approximately 200 hours after casting, the gauges in the lower half of the pile began to show a trend of increasing compressive strains due to the development of residual load as the clay layer began to settle. Similar findings were reported by Fellenius et al. (2009) in precast post-grouted concrete cylinder piles in soft marine clay after concrete-related effects had diminished. For both studies, the strains near the head of the pile remained relatively constant during this period, indicating the absence of residual load at this location as might be expected.
2.7 CONCLUSIONS

This chapter presented a detailed review of the axial behaviour of closed-ended displacement piles in sand. The review primarily focused on the results of a series of tests conducted by Imperial College London using heavily-instrumented closed-ended steel model piles. Mechanisms governing both the shaft and base resistance were discussed and possible implications for DCIS piles were highlighted throughout. Several CPT-based design approaches were subsequently discussed and their predictive performance reviewed. Finally, the processes leading to residual load development in cast-in-situ piles were assessed. The key mechanisms governing the shaft and base resistance of traditional closed-ended displacement piles (i.e. closed-ended steel and precast concrete piles) highlighted in the review are subsequently appraised against the results of the instrumented DCIS pile tests in Chapters 5 and 6 in order to verify that DCIS piles behave as large displacement piles.

Figure 2.30 illustrates the methodology used for each of the instrumented DCIS pile tests conducted in Chapters 5 and 6 to assess the behaviour of DCIS piles during installation, curing and maintained compression load testing in layered soil and uniform sand respectively. This process begins with the characterisation of the ground conditions at the test pile location, the results of which are presented in Chapter 4, followed by the design of the test pile, including dimensions, reinforcement and instrumentation details (i.e. gauge type and levels). The instrumented test pile is then constructed, followed by a suitable curing period to enable the concrete to develop sufficient strength, after which the pile is subjected to a maintained compression load test to displacements in excess of 10 % of the pile base diameter $D_b$. The experimental procedures used during these processes are detailed in Chapter 3 (including the strain interpretation process used to measured residual loads during curing, as well the shaft and base resistance during loading). The behaviour of the test pile during installation, curing and maintained compression load testing is subsequently examined in Chapter 7, with comparisons made throughout with the key mechanisms governing traditional closed-ended displacements in sand reviewed in Chapter 2.
Research objective:
Assess behaviour of driven cast-in-situ piles during installation, curing and maintained compression load testing in layered soil and uniform sand

Site characterisation (Chapter 4):
- Site location
- Geological history
- Previous site investigations
- Cone penetration tests (CPT) at test pile location to identify soil profile
- Laboratory tests for soil parameters to aid in test interpretation

Design of instrumented DCIS pile and test procedures
- Test pile dimensions – diameter, length, pile cap type (Chapter 3)
- Estimation of total, shaft and base capacity using CPT-based design methods (Chapter 2)
- Concrete strength, steel reinforcement (Chapter 3)
- Instrumentation type, levels based on soil layers (Chapter 3)
- Load test schedule, frame size, dimensions of anchor piles (Chapter 3)

Installation, curing and load testing of instrumented DCIS pile:
- Instrument reinforcement cage with strain gauges (Chapter 3)
- Pile installation, measurement of driving resistance using rig instrumentation, insertion of instrumented reinforcement cage (Chapter 3)
- Logging of strain and temperature data at 15 minute intervals for first 24 hours after casting, hourly intervals for remainder of curing period (Chapter 3)
- Maintained compression load test to large normalised displacements ( > 10 % of pile base diameter $D_b$) to mobilise shaft and base resistance (Chapter 3)
- Measurement of applied load, pile head displacement and strains during loading (Chapter 3)

Assessment of DCIS pile behaviour
- Installation
  - Comparison of driving resistance with base resistance profiles predicted by CPT-based methods (Chapters 2, 5, 6, 7)
- Curing
  - Temperature and strain behaviour during curing (Chapter 2, 5, 6, 7),
  - Assessment of residual load based on strain profile (Chapter 3, 5, 6, 7)
- Maintained load testing
  - Load test strain interpretation – raw strains, correction for creep, derivation of pile stiffness, load distribution (Chapter 3, 5, 6)
  - Shaft resistance – local and average shear stress mobilisation, variation in local shear stress and radial effective stress with distance from pile base and number of loading cycles during installation (Chapter 2, 3, 5, 6, 7)
  - Base resistance – base resistance at failure, comparison with cone resistance averaging techniques, stiffness degradation, bearing capacity factors (Chapter 2, 5, 6, 7)
  - Predictive performance of CPT-based design methods (Chapters 2, 7)
  - Comparison of load-displacement curve with traditional displacement piles (Chapter 7)

Figure 2.30 - Methodology flowchart for instrumented DCIS pile tests
CHAPTER 3 EXPERIMENTAL METHODS AND PROCEDURES
3.1 INTRODUCTION
This chapter provides a detailed description of the various experimental methods and procedures performed as part of the instrumented DCIS piles tests described in Chapters 5 and 6. The first section of the chapter describes the equipment and instrumentation used during DCIS pile installation, the curing phase and maintained compression load testing. A review of the various curing strain interpretation methods is then presented, followed by the procedures used to interpret the measured strains within the instrumented DCIS test piles during maintained compression load testing.

3.2 INSTALLATION

3.2.1 DCIS pile installation equipment
The instrumented DCIS test piles were installed using Junttan PM26 piling rigs fitted with a 20 mm thick open-ended steel installation tube with nominal outer diameters varying from 283 mm to 457 mm and lengths varying from 10 m to 21.5 m (depending on the shaft diameter and length of DCIS pile required). The head of the tube is tapered outwards to enable extraction from the soil after concreting using a steel retractable collar fitted to the leader of the rig. The installation tube is driven using a Junttan HHK 5ASS series hydraulic hammer which has a total ram mass of 5000 kg, a maximum drop height of 1.2 m and a maximum rated energy of 58.8 kJ. The hydraulic power accelerates the hammer during freefall, resulting in minimal losses in energy upon impact with the tube. The vertical position of the hammer along the 28.7 m long piling leader is controlled by the rig operator using a system of cables and pulleys which are connected to a motor within the rig. An illustration of the DCIS piling rig, together with the hydraulic hammer, installation tube and collar, is shown in Figure 3.1.

The installation tube is driven in a closed-ended manner using a sacrificial steel circular plate, fitted to the base of the tube prior to driving, which remains at the base of the freshly-cast pile after installation. Figure 3.2(a) shows a typical 380 mm diameter plate (albeit inverted) used during installation of the instrumented DCIS test piles in Chapters 5 and 6, while the attachment of the plate to the base of the installation tube before commencement of driving is illustrated in Figure 3.2(b).
Figure 3.1 - Junttan PM26 series DCIS piling rig

Figure 3.2 - (a) Inverted steel base plate and (b) with installation tube in position prior to driving
3.2.2 Pile construction

The driving plate is placed at the intended location of the pile prior to installation and the rig is subsequently manoeuvred into position with the tube placed on top of the plate as shown previously in Figure 3.2(b). The hydraulic hammer is then lowered into position at the top of the tube and driving commences, with the hammer drop height controlled by the rig operator. The rig instrumentation provides the operator with information regarding installation resistance, tube depth and total number of blows using an electronic screen within the rig cabin. When the tube reaches the required depth, driving is stopped and the hammer is retracted upward along the leader. The pile is subsequently constructed by introducing concrete into the top of the installation tube via a specialised skip which can be raised or lowered using the piling winch cable (see Figure 3.3a). Alternatively, the concrete can be pumped into the tube using a specialised connection fitted to the head of the tube. When concreting is complete, the hammer is reattached and the tube is extracted from the soil using the collar connected to the tapered section at the head. Several blows are applied to the tube during this process in order to eliminate any voids present in the concrete. Figure 3.3(b) shows the tube after extraction from the ground, together with excess concrete which is removed prior to pile cap construction.

![Figure 3.3 - (a) Skipping of concrete into top of installation tube and (b) tube withdrawal](image-url)
3.2.3 Reinforcement

The test piles were reinforced with four ribbed steel bars throughout the pile length, with diameters ranging from 20 mm to 40 mm, depending on the estimated structural capacity of the pile. A helical cage, typically 8 mm to 10 mm in diameter with a pitch spacing of between 200 mm and 300 mm, was welded to the outer section of the main bars in order to provide shear reinforcement. The reinforcement was typically fabricated at Keller Foundations head office in Ryton-on-Dunsmore and subsequently transported to site where it was placed on timber sleepers to enable attachment of the pile instrumentation prior to installation. Plastic spacers were placed on the helical reinforcement at locations between gauge levels in order to provide cover distances of between 40 mm and 75 mm, depending on ground conditions at each test site.

In order to minimise damage to the strain gauge instrumentation, the reinforcement cage (with the instrumentation attached) was installed after casting of the concrete and extraction of the installation tube. The cage was slowly lifted into a vertical position using the winch cable on the piling rig and pushed into the fresh concrete by site operatives (see Figure 3.4a). Care was taken throughout this process to ensure the cage remained vertical. As shown in Figure 3.4(b), the bucket of a JCB was typically used to push the cage during the latter stages of installation.

Figure 3.4 - (a) Cage installation and (b) additional force provided by JCB bucket
3.2.4 Pile cap

A concrete pile cap was constructed at the head of each test pile after installation in order to accommodate the instrumentation for the maintained load test (see Section 3.6.2). With the exception of the test piles installed at Ryton-on-Dunsmore, the caps for each test pile comprised 700 mm square, 900 mm deep concrete which were embedded ≈ 500 mm below ground level (bgl) after test pile installation.

Each cap was constructed by excavating the surrounding soil and placing a hollow wooden frame which was subsequently concreted with the top surface trowelled flat and the strain gauge wiring bundled out through a corner of the cap as shown in Figure 3.5(a). Due to a change in site operating procedures at Keller Foundations, the test pile caps at Ryton-on-Dunsmore were constructed using a 450 mm diameter, 700 mm long steel casing which was embedded ≈ 300 mm below the surface (Figure 3.5b). The strain gauge wiring was accommodated through a cavity in the side of the casing. Cap reinforcement was provided by the main bars and shear cage of the test piles which protruded to within 50 mm of the top of the cap.

Figure 3.5 - (a) Square pile cap with wooden casing and (b) circular pile cap with steel casing
3.3 RIG INSTRUMENTATION

The DCIS piling rigs at Keller Foundations are fitted with instrumentation which enables the installation resistance of the driving tube to be derived using the Danish pile driving formula (Chellis, 1961). The velocity of the hammer during freefall is measured using two sensors located at the top of the hammer casing. The sensors, shown in Figure 3.6(a), are mounted on a steel bracket and placed ≈ 50 mm vertically apart. During hammer freefall, the time taken for the hammer rod to transit this distance is measured by the sensors and converted to a velocity. The tube depth is measured by a laser which is mounted at the top of the piling leader (Figure 3.6b), with a target plate placed on top of the hammer carriage (Figure 3.6a). The derivation of the installation resistance $q_{cp}$, conducted as part of this thesis, is given in Appendix A.

![Figure 3.6 - (a) Hammer velocity sensors and (b) tube displacement laser](image)
3.4 PILE INSTRUMENTATION AND DATA ACQUISITION

3.4.1 Vibrating wire strain gauges

The DCIS test piles were instrumented with vibrating wire strain gauges (VWSG) at four separate levels in each pile, with four gauges placed at each level to capture bending effects, as well as for redundancy purposes. Vibrating wire strain gauges were chosen due to their robustness, ease of installation and protection from moisture. Each gauge contains a tensioned steel wire within a plastic casing which in turn is connected to steel bars at each end. A magnetic field generated by electromagnetic coils housed within the plastic casing causes the steel wire to oscillate at its resonant frequency. This process generates an alternating current, the frequency of which is processed by a datalogger connected to the gauges. A change in load within the pile at the gauge level results in a proportional change in length (and hence frequency) of the wire, enabling the change in strain $\Delta \mu \varepsilon$ to be derived using the following formula:

$$\Delta \mu \varepsilon = \left( \frac{f_{\text{initial}}^2}{1000} - \frac{f_{\text{current}}^2}{1000} \right) \text{GF}$$  \hspace{1cm} (3.1)

where $f_{\text{initial}}$ and $f_{\text{current}}$ are the initial and current frequency readings respectively and GF is the strain gauge factor obtained during calibration by Geosense (no independent calibration was conducted by the author); note that positive values of $\Delta \mu \varepsilon$ indicate compression and negative values tension. Each gauge is also fitted with a thermistor within the plastic casing, enabling measurement of temperature during curing and load testing.

Three variations of VWSGs were used during the series of pile tests described in Chapters 5 and 6. The first variant is the Geosense G100-40 sister-bar strain gauge, shown in Figure 3.7(a), which comprises the gauge casing with $\approx 500$ mm steel bars connected to each end. The second type is the reduced sister-bar gauge where the bar lengths are reduced to $\approx 200$ mm (to enable the gauges to be placed as close to the pile base as possible) and fitted with circular steel flanges at the end to enable a sufficient bond to develop within the concrete during loading (see Figure 3.7b). The third variant is the embedment vibrating-wire strain gauge (Geosense VWS2100) which was used in the instrumented test piles at Ryton-on-Dunsmore (see Section 6.3 of Chapter 6). The embedment gauge operates in an identical manner to the sister-bar type, but are considerably smaller, having a total length of $\approx 150$ mm (Figure 3.7c).
Figure 3.7 - (a) Sister-bar (b) reduced-length sister bar and (c) embedment vibrating wire strain gauges
3.4.2 Strain gauge installation

The strain gauges were procured by ESG on behalf of NUI Galway/Keller Foundations and packaged in cardboard prior to transport to site, with the plastic gauge housing covered in protective foam to prevent ingress of moisture prior to use. Once on site, the serial number and position of each gauge was noted, after which the gauges were securely attached in a vertical orientation to the inner portion of the helical reinforcement cage using cable ties (Figure 3.8 and Figure 3.9). The strain gauge wiring was carefully threaded along the main reinforcement bars and out through the pile head where it was bundled in order to prevent damage during installation. Given that each test pile contained 16 gauges, the attachment process typically took 2 hours to complete. The gauge level positions were chosen to optimise the measurement of shaft resistance across soil layers (particularly at the layered soil sites in Chapter 5). Each gauge was subsequently checked to see if it was in working order using a hand-held device which sends an electrical charge to excite the wire within the gauge, enabling the device to measure the wire frequency. Readings were also taken after the cage was inserted into the concrete in order to assess if either damage or large shifts in readings had occurred. The length of the embedment strain gauges (≈ 150 mm) used in the instrumented test piles at Ryton-on-Dunsmore prevented attachment to the shear reinforcement due to the pitch of the helical cage. To overcome this, the gauges were cable-tied to two steel mounting rings welded to the inside of the main reinforcement bars approximately 100 mm apart at each gauge level (Figure 3.10). The mounting rings were wrapped in PVC tape at the point of attachment to provide an additional level of grip for the gauges during cable-tying.

Figure 3.8 - Attachment of the sister-bar vibrating wire strain gauges
Figure 3.9 - Attachment of reduced-length sister-bar vibrating wire strain gauges

Figure 3.10 - Attachment of embedment vibrating wire strain gauges
3.4.3 Data acquisition

The variation in strain and temperature during curing of the test piles at Dagenham, Erith and Shotton (see Chapters 5 and 6 for more details) was measured using a Campbell Scientific AM416 multiplexer data acquisition unit, shown in Figure 3.11, which enabled up to 20 vibrating wire strain gauges to be monitored at any given time. Strain and temperature measurements were typically logged at 15 minute intervals for the first 24 hours or so after casting, followed by hourly intervals for the remainder of the curing period. The multiplexer was powered by a portable battery and placed in metal casing to protect it from damage on site during curing (see Figure 3.12). Strains were also logged continuously by the multiplexer during the subsequent maintained compression load test.

![Figure 3.11 - Campbell Scientific multiplexer](image1)

![Figure 3.12 - Multiplexer and battery in metal casing](image2)
3.5 RESIDUAL LOAD INTERPRETATION METHODS

The review of the literature in Section 2.6 highlighted the contribution of concrete curing and soil consolidation to the development of residual load in cast-in-situ piles. Given the observed strain profiles, three methods for interpreting the level of residual load present in a cast-in-situ pile have been proposed. This section aims to describe each interpretation method in detail; these will be appraised for the DCIS test series in Section 7.3.2.

3.5.1 Simplified method (Method 1)

A simplified method of interpretation, described by Kim et al. (2004), assumes that the absolute change in total strain during the period between casting and conducting a load test is entirely due to residual loads. This method, designated Method 1, is convenient as strain readings are only required immediately after casting and prior to conducting a load test. However, the variation in strain and temperature during this period remains unknown. Furthermore, several papers have reported significant tensile strains in existence after the effects of hydration had diminished (as described in Section 2.6.2.2) which would result in unrealistically large tensile loads throughout the pile. This method is therefore deemed to be the least accurate for determining residual loads in cast-in-situ piles.

3.5.2 Continuous methods (Methods 2 and 3)

The downfall of the simplified method lies in its inability to separate the change in strain between casting and load testing into the various processes described in Section 2.6.2. To overcome this, continuous or frequent measurements of strain and temperature can be obtained using a datalogger, enabling an accurate profile of strain and temperature variation in the pile during curing to be known. Two methods of interpretation of residual loads based on frequent measurements of strain and temperature have been proposed, with the main difference between the methods relating to the times at which residual load is assumed to develop.

Pennington (1995) proposed calculating the change in strain between peak temperature and load testing to represent the residual load in a cast-in-situ pile, on the basis that the pile is in a stress-free state at peak temperature. The temperature-corrected strain profile was used for the
interpretation, resulting in a change in strain which was compressive at each gauge level. This method is designated Method 2.

The alternative interpretation method for residual loads using continuous strain measurements was recommended by Siegel and McGillivray (2009) and Kim et al. (2011). The correction procedure, referred to as Method 3, is based on the assumption that residual loads are negligible at or near the head of the pile. By placing a set of strain gauges at this level, a strain profile which is independent of residual load can be obtained. This profile can then be compared to the profiles at the remaining sections of the pile. The instance at which a strain profile at a particular section begins to deviate from the top gauge level is deemed to be the time at which residual loads begin to develop, and the change in strain at each gauge level after this instance represents the strain due to residual load.

The measured strain profiles at two gauge levels (4 m and 49 m bgl) after casting of a 1.2 m diameter post-grouted cylinder pile reported by Fellenius et al. (2009) are shown in Figure 3.13 where compressive total (i.e. uncorrected) strains coincided with peak hydration temperature, followed by a tensile response as the pile cooled during curing. The strain profiles began to diverge however approximately 10 days after casting due to the development of residual loads within the pile. Figure 3.13 also shows the interpreted change in strain due to residual load $\Delta \mu e_{\text{residual}}$ given by each method described previously.

Figure 3.14 shows a comparison of the derived residual strain distributions at five separate gauge levels within the post-grouted concrete cylinder pile by Fellenius et al. (2009) approximately 39 days after casting where it is clear that the three interpretation methods give contrasting distributions of residual strain. Method 1 (Kim et al., 2004) was deemed to be the least accurate as the large tensile strains would have likely resulted in cracking of the concrete, while the compressive strains calculated using Method 2 (Pennington, 1995) appear excessive ($> 200 \mu e$). For both methods, the conversion of the residual strains to loads (using Equation 2.19) and the addition of these residual loads to the load measurements during the subsequent static load test yielded distributions of increasing load with depth at failure which seem highly unrealistic. Method 3 by Siegel and McGillivray (2009) appears to give the most realistic results, as the derived residual load distribution is similar to the dragload which was
anticipated to develop due to the influence of external processes in the soft clay layers in which the pile was driven (i.e. negative skin friction due to consolidation). Furthermore, this method resulted in a ‘true’ load distribution at failure which reduced with depth, as demonstrated by Kim et al. (2011). Method 3 is therefore used to interpret the residual loads within the instrumented DCIS piles in Chapters 5 and 6 in which curing strains and temperatures were continuously monitored during curing.

Figure 3.13 - Comparison of residual load interpretation methods

Figure 3.14 - Interpreted residual strain distributions
3.6 LOAD TESTING

3.6.1 Loading frames
Environmental Scientifics Group (ESG) Ltd. was contracted by Keller Foundations UK to perform the maintained compression load tests at all test sites. The majority of the load tests were performed using ESG’s System 300 loading frame which has a maximum working load of 3000 kN and is connected to four DCIS anchor piles using high-strength Dywidag steel anchor bars which were centrally cast into the anchor piles after installation. Connections are also available for two additional anchor piles if needed (as was used for the compression load test at Shotton). The tests at Ryton-on-Dunsmore were conducted using a 2200 kN capacity System 200 frame connected to three DCIS anchor piles; the setup arrangement for this frame is shown in Figure 3.15. The anchor piles were designed by the author based on the estimated test pile capacity derived using the CPT-based methods described in Section 2.5 with a geotechnical factor of safety FOS = 2.0.

Figure 3.15 - ESG System 200 loading frame
3.6.2 Instrumentation

The instrumentation for the maintained compression load tests comprised a load cell, hydraulic jack, displacement transducers and data acquisition unit; the general setup of the instrumentation on the pile cap is shown in Figure 3.16. The hydraulic jack, manufactured by Woodland Weighing Systems Ltd., has a spherical-seat platen to minimise eccentricities during loading. Calibration of the jack was performed by ESG prior to testing. The applied load during the maintained compression load tests was measured using a Penny & Giles Ltd. 3000 kN capacity load cell, calibrated by ESG prior to use, which was placed between the underside of the loading frame and the hydraulic jack. Four linear potentiometric displacement transducers (LPDT) rigidly mounted to an independent reference beam were used to measure the pile head displacements using flat surfaces attached to the side of the pile cap. The transducers have an accuracy of 0.1 mm and a resolution of 0.01 mm. The supports for the reference beam were placed a minimum of three pile diameters from the centre of the test pile. Data acquisition during loading was performed using a Campbell Scientific datalogger which was connected to the LPDTs, load cell and strain gauge instrumentation. A digital thermometer was also used to monitor the ambient temperature throughout the duration of the load test. The overall setup of the instrumentation at the test pile is shown in Figure 3.16.

![Figure 3.16 - General layout of load test instrumentation](image_url)
3.6.3 Loading procedure

The maintained compression load tests on the instrumented DCIS piles at Pontarddulais, Dagenham, Erith and Shotton were performed in accordance with the Institution of Civil Engineers Specification for Piling and Embedded Retaining Walls or SPERW (ICE, 2007) which is the standard specification for static load testing of piles in the United Kingdom. Loading is performed in several stages, with the level of increment based on the design verification load (DVL) of the pile which was calculated by dividing the estimated total capacity (determined by either earth pressure or CPT-based methods) by the geotechnical factor of safety. Cycling (i.e. a single unload-reload loop) is also performed at applied loads corresponding to 100 % DVL and 100 % DVL + 25% SWL, where SWL is the specified working load (note that SWL is equivalent to DVL for the test piles in Chapters 5 and 6). Table 3.1 summarises the minimum loading times specified by ICE SPERW for single- and multi-cyclic maintained compression load tests. The specification also requires that the applied load remain constant until one of the following settlement rates has been satisfied:

- 0.1 mm/hour when the pile head displacement is less than 10 mm
- 1 % × pile head displacement/hour for pile head displacements between 10 mm and 24 mm
- 0.24 mm/hour when the pile head displacement is greater than 24 mm

Single cycle tests were performed on the test piles at Dagenham and Erith due to difficulties associated with interpretation of the strains (see Chapters 5 and 6 for more details).

The test piles at Ryton-on-Dunsmore (see Section 6.3 of Chapter 6) were not performed in accordance with ICE SPERW as these tests were conducted purely for research purposes and were not part of a specific piling contract (which was the case at the other test pile sites). The piles were loaded in increments equivalent to 10 % of the predicted compressive capacity, reducing to 5 % when pile failure was imminent. The minimum and maximum duration of each load was 30 minutes and 6 hours respectively. The only exception to this criterion was Pile R1 which was held at 962 kN for 12 hours in order to accommodate a rest period for the load test technician. The next load increment was applied when the settlement rate had reduced to 0.2 mm/hour (or the maximum hold duration was reached), although this criterion became increasingly difficult to adhere to when creep displacements increased significantly as the pile approached failure. The specific loading schedules for each test pile are summarised in Appendix C.
Table 3.1 - Minimum loading times for a maintained compression load test (adapted from ICE SPERW 2007)

<table>
<thead>
<tr>
<th>Load</th>
<th>Minimum time of holding for a single-cyclic pile test</th>
<th>Minimum time of holding for a multi-cyclic pile test</th>
</tr>
</thead>
<tbody>
<tr>
<td>25% DVL</td>
<td>30 minutes</td>
<td>30 minutes</td>
</tr>
<tr>
<td>50% DVL</td>
<td>30 minutes</td>
<td>30 minutes</td>
</tr>
<tr>
<td>75% DVL</td>
<td>30 minutes</td>
<td>30 minutes</td>
</tr>
<tr>
<td>100% DVL</td>
<td>6 hours</td>
<td>6 hours</td>
</tr>
<tr>
<td>75% DVL</td>
<td>N/A</td>
<td>10 minutes</td>
</tr>
<tr>
<td>50% DVL</td>
<td>N/A</td>
<td>10 minutes</td>
</tr>
<tr>
<td>25% DVL</td>
<td>N/A</td>
<td>10 minutes</td>
</tr>
<tr>
<td>0</td>
<td>N/A</td>
<td>1 hour</td>
</tr>
<tr>
<td>100% DVL</td>
<td>N/A</td>
<td>1 hour</td>
</tr>
<tr>
<td>100% DVL + 25% SWL</td>
<td>1 hour</td>
<td>1 hour</td>
</tr>
<tr>
<td>100% DVL + 50% SWL</td>
<td>6 hours</td>
<td>6 hours</td>
</tr>
<tr>
<td>100% DVL + 25% SWL</td>
<td>10 minutes</td>
<td>10 minutes</td>
</tr>
<tr>
<td>100% DVL</td>
<td>10 minutes</td>
<td>10 minutes</td>
</tr>
<tr>
<td>75% DVL</td>
<td>10 minutes</td>
<td>10 minutes</td>
</tr>
<tr>
<td>50% DVL</td>
<td>10 minutes</td>
<td>10 minutes</td>
</tr>
<tr>
<td>25% DVL</td>
<td>10 minutes</td>
<td>10 minutes</td>
</tr>
<tr>
<td>0</td>
<td>1 hour</td>
<td>1 hour</td>
</tr>
</tbody>
</table>
3.7 INTERPRETATION OF LOAD TEST STRAINS

3.7.1 Introduction
This section describes the procedures used to interpret the strain gauge data measured during maintained compression load testing of the instrumented DCIS piles in Chapters 5 and 6. These procedures include correction for creep effects, derivation of pile stiffness and averaging methods.

3.7.2 Raw strains
The strain gauge frequency readings during both curing and load testing were converted to strain using Equation 3.1 and subsequently compiled in a Microsoft Excel file for analysis. The raw strains were initially plotted against both applied stress at the pile head and time during the load test to assess any potential abnormalities in gauge behaviour during the maintained load test. If no abnormalities existed (as was generally the case), the strains were subsequently corrected for creep effects as described in Section 3.7.3.

3.7.3 Creep effects
The increase in strain within concrete under the application of a constant stress is referred to as creep (Neville and Brooks, 1987) and the development of creep strains within a concrete pile can therefore be significant during a maintained compression load test (where applied loads may be held constant for several hours as described previously in Section 3.6.3). The magnitude of creep is inversely proportional to the strength of the concrete (Neville and Brooks, 1987) and will therefore be considerably larger within concrete piles which are load tested within a few days of casting (due to the applied load being a greater portion of the compressive concrete strength at the time of loading). Lehane et al. (2003), for example, observed large increases in displacement of a 350 mm instrumented precast concrete pile during a maintained compression load test approximately 7 days after casting which was attributed to creep deformation within the concrete. Such increases in strain during each load hold can have a strong influence on the interpreted pile stiffness and should therefore be accounted for accordingly (Lam and Jefferis, 2011).
For this thesis, elastic strains were assumed to be fully mobilised approximately 10 minutes after application of a new load (Lam and Jefferis, 2011), with increases in strain thereafter attributed to creep within the concrete. Figure 3.17 shows the corresponding variation in measured and elastic strain at a particular gauge level with time during a maintained compression load test on an instrumented DCIS pile; the elastic strains remain constant during each load hold, while creep within the pile leads to increases in the measured strain, resulting in a difference of ≈ 13% in this case at the maximum applied load. The resulting measured and elastic stress-strain responses of the data in Figure 3.17 are shown in Figure 3.18 where the responses are comparable during the initial stages of loading, the difference becoming considerably larger during the latter stages (as the applied stress becomes a greater portion of the concrete compressive strength). Creep strains must therefore be corrected for in order to obtain an accurate pile stiffness during loading.

Figure 3.17 - Variation in measured and elastic strain with time during a maintained load test

Figure 3.18 - Stress-strain response during loading
3.7.4 Strain averaging

After correction for creep effects, the four strain readings at each gauge level were averaged in order to account for the effect of pile bending during loading. Following the recommendations of Fellenius and Tan (2012), averaging was initially performed using diametrically-opposing strain gauges (Figure 3.19a) and then compared with the average obtained from the all four gauges at each level (Figure 3.19b). If a gauge exhibited irregularities in measured strain (due to either damage during installation or cracking during curing and/or loading), the diametrically-opposing gauge was subsequently ignored and the remaining two gauges at the level in question were used to determine the average strain; this was rarely required however as the majority of test piles had fully functioning gauges.

![Figure 3.19 - Variation in (a) individual strains and (b) average strain with applied stress](image)

3.7.5 Pile stiffness

The variation in pile stiffness $E_p$ during loading of an instrumented concrete pile must be derived in order to convert the measured strains to loads. While several methods for determining $E_p$ are available, the strain-dependent secant and tangent methods (Fellenius, 1989) remain the most popular (Lam and Jefferis, 2011). These methods, illustrated in Figure 3.20, derive $E_p$ from the slope of the stress-strain response at the uppermost gauge level, the only difference between them being the type of slope used, i.e. secant or tangent. Values of $E_p$ are calculated for each load increment and plotted against the corresponding elastic strain, after which an $n^{th}$-order polynomial is fitted to the data in order to obtain $E_p$ for a given strain level.
Figure 3.20 - Pile stress-strain relationship and modulus derivation methods

Figure 3.21 shows a comparison of the derived secant and tangent moduli for the stress-strain response in Figure 3.19(b) where it is apparent that the methods yield vastly different stiffness curves, with a somewhat erratic tangent modulus response evident during the latter stages of loading. This erratic response is generally attributed to the increased sensitivity to errors in stress and strain values during the differentiation process of the tangent method (Fellenius, 2012a; Lam and Jefferis, 2011). The stiffness response of the instrumented test piles in Chapters 5 and 6 were therefore determined using the secant modulus method.

Figure 3.21 - Comparison of secant and tangent modulus methods
Lehane et al. (2003) proposed an alternative method for determining the variation in pile stiffness with strain level, based on a series of compression tests on instrumented concrete cylinders tested between 4 days and 28 days after casting. Unlike the secant and tangent methods, no correction for creep strains is necessary, enabling the variation in load during each hold to be determined using the following expression:

\[
E_c = E_i \left[ 1 - f \left( \frac{\sigma}{f_{ck}} \right) \right] \left[ 1 + 0.08 \left( \frac{\sigma}{f_{ck}} \right)^{1.5} \ln \left[ \frac{d\varepsilon}{dt} \right] \left( \frac{d\varepsilon_{ref}/dt}{d\varepsilon_{ref}/dt} \right) \right]
\]

where \( E_i \) is the initial linear stiffness of the concrete, \( f_{ck} \) is the concrete cylinder compressive strength, \( f \) is an empirical parameter, \( d\varepsilon_{ref}/dt \) is the reference creep rate during loading and \( d\varepsilon/dt \) is the strain rate. Lehane et al. (2003) used a reference strain rate \( \varepsilon_{ref} = 3.5 \mu \varepsilon/s \) to derive the load distribution of an instrumented 350 mm wide precast concrete pile during maintained compression load testing in soft alluvial sand and clay. However, this method was not used to determine the moduli of the instrumented DCIS test piles in Chapters 5 and 6 as the required tests to determine the compressive strength \( f_{ck} \) of the concrete used in each test pile were not performed.

The majority of the instrumented DCIS test piles in Chapters 5 and 6 had uppermost strain gauge levels which were placed between 0.5 m and 2.5 m below the ground surface, with the shaft resistance mobilised across this distance during loading resulting in a lower stress level at the gauges. In order to determine the stiffness correctly, the magnitude of shaft resistance was estimated using empirical relationships with the cone resistance \( q_c \) profile measured at each pile location (see Chapter 4 for more details) proposed by Bustamante and Gianeselli (1982), described in Section 2.5.2, primarily due to the simplicity of the equations. The load-transfer curve between the surface and the uppermost gauge level was subsequently derived using the procedure by Niazi and Mayne (2010), summarised in Figure 3.22, which is based on the closed-form solutions for pile displacement during loading by Randolph and Wroth (1978). The small-strain shear modulus \( G_0 \) was derived using relationships with the CPT \( q_c \) profile as discussed in Chapter 4. However, given that the shear modulus reduces with strain level during loading, a reduction factor was applied to \( G_0 \) in order to calculate the operational shear modulus \( G \) using the expression by Fahey and Carter (1993):
where $\tau_{s,\text{loc}}/\tau_{sf}$ is the mobilised level of shear stress, and $f (=0.99)$ and $g (=0.3)$ are empirical curve fitting exponents. The resulting shaft load transfer curve, obtained using the procedure in Figure 3.22 and Equation 3.3, was subtracted from the load-displacement response measured at the pile head in order to derive the variation in stress $\sigma$ at the uppermost gauge level. The pile modulus was then estimated using the secant method described previously.

\[
G = G_0 \left[ 1 - f \left( \frac{\tau_{s,\text{loc}}}{\tau_{sf}} \right)^g \right] \tag{3.3}
\]

Once the strain-dependent pile modulus has been established, the load $P_i$ at a particular gauge level was obtained using the following relationship:

\[
P_i = \varepsilon_{\text{elastic}} E_p A_p \tag{3.4}
\]

where $\varepsilon_{\text{elastic}}$ is the creep-corrected elastic strain, $E_p$ is the secant pile stiffness and $A_p$ is the cross-sectional area of the pile at the gauge level in question. The resulting load distribution was then used to derive the local shear stress $\tau_{s,\text{loc}}$ and base resistance $q_b$ during loading, as described below.
3.7.6 Base resistance

The lowermost level of vibrating wire strain gauges were typically placed within ≈ 100 mm to 250 mm (depending on the type described in Section 3.4.1) of the base of each DCIS test pile in order to measure the base resistance $Q_b$ during loading. Given that additional shaft resistance was likely to be generated between the lowermost gauge level and the base, the resulting load distribution was linearly extrapolated to the base level in order to estimate the true base resistance of the pile; such a procedure is likely to give a slight over-prediction in $Q_b$ as the local shear stress of a displacement pile generally increases exponentially adjacent to the base.

3.7.7 Shaft resistance

The local shear stress $\tau_{s,loc}$ between two successive gauge levels is derived from the load distribution using the following expression:

$$\tau_{s,loc} = \frac{\Delta Q_s}{\pi D_s}$$  \hspace{1cm} 3.5$$

where $\Delta Q_s$ is the change in load between the two gauge levels in question and $D_s$ is the pile shaft diameter.

3.7.8 Elastic shortening

The applied load at the pile head inevitably leads to elastic shortening of the pile during loading. While several methods have been proposed for estimating the quantity of elastic shortening $w_{\text{elastic}}$ during loading, the following expression is used in this thesis, assuming a triangular distribution in local shear stress $\tau_{s,loc}$ between the pile head and base (Kyfor et al., 1992):

$$w_{\text{elastic}} = \left(\frac{Q_{\text{head}}}{2} - \frac{Q_{si}}{2}\right) \frac{4L_i}{\pi D_s^2 E_p}$$  \hspace{1cm} 3.6$$

where $Q_{\text{head}}$ is the applied load at the pile head, $Q_{si}$ and $L_i$ are the total shaft resistance and length between the pile head and the level in question respectively, $D_s$ is the shaft diameter and $E_p$ is the strain-dependent pile stiffness.
CHAPTER 4  GROUND CONDITIONS AT PILE TEST SITES
4.1 INTRODUCTION

This chapter describes the ground conditions at the three layered sites (Pontarddulais, Dagenham and Erith) and two sand sites (Shotton and Ryton-on-Dunsmore) in the United Kingdom in which the instrumented DCIS pile tests were performed. The location and geological history of each site is initially presented, followed by the results of in-situ and laboratory tests (where applicable). Parameters relevant to pile behaviour are also discussed. The test sites are identified in Figure 4.1. It is important to highlight that the majority of sites had relatively basic ground investigations, typically comprising only borehole logs with SPT results, and these were conducted prior to involvement from NUI Galway. Cone penetration testing (CPT) was therefore commissioned at the instrumented DCIS pile test area of each site as part of this research in order to help classify the soil conditions and enhance the interpretation of the results of the instrumented pile tests.

![Figure 4.1 - Summary of pile test locations](image-url)
4.2 PONTARDDULAIS, WALES

4.2.1 Site location
The pile test site was located in the western portion of the Lye Industrial Estate (National Grid reference SN 596 053), ≈ 1.5 km north of the village of Pontarddulais, Wales and approximately 16 km northwest of the city of Swansea. The total area of the site is ≈ 6 hectares, most of which is occupied by a large distribution warehouse. A smaller warehouse structure was previously situated at the western section of the site, but was demolished in the summer of 2010 to accommodate further expansion of the larger warehouse. The site is accessed by Glanffrwd Road to the south, with the northwest perimeter bounded by a railway line, while vegetation and unsurfaced access tracks are present along the western perimeter. The site was previously used as a metal plating facility until the mid-1960s, after which it remained relatively derelict until the construction of the existing buildings in the 1990s.

Figure 4.2 shows a general layout of the site, while a plan view of the instrumented DCIS pile test area is illustrated in Figure 4.3. The topography of the site slopes approximately 2 m from north to south over a distance of ≈ 350 m, with an average elevation of 98 m AOD (above Ordnance Datum). The instrumented DCIS pile test area was located at the northern end of the site, approximately 20 m from the footprint of the new structure (see Figure 4.2) in order to avoid any potential constraints with the on-going construction programme. The results of the instrumented pile test are described in detail in Section 5.2 of Chapter 5.

4.2.2 Geology
The bedrock comprises thick layers of sandstone with thin interbedded seams of mudstones, shales and coal seams, known as the Upper Pennant Measures which were formed 305 million years ago during the Carboniferous Period. Superficial deposits of glacial till and fluvioglacial sands and gravels overlie the bedrock, with layers of peat and alluvium present throughout the site.
Figure 4.2 - Site layout - Pontarddulais

Figure 4.3 - Test pile layout - Pontarddulais
4.2.3 Ground investigation

A basic ground investigation was conducted by Mini-Soil Surveys Ltd. in July 2010, prior to demolition of the existing warehouse structure, and comprised 5 no. cable percussive boreholes to maximum depths of ≈ 4 m below ground level (bgl), although the presence of a reinforced concrete slab prevented construction of borehole BH05 below 0.2 m bgl.

The soil profile of borehole BH04, which was located ≈ 30 m from the instrumented DCIS test pile location, is shown in Figure 4.4(a), while Figure 4.4(b) presents the corresponding profile of SPT \( N_{SPT} \) values with depth during construction of the borehole. The stratigraphy comprises medium dense to dense made ground, containing sand, gravel, cinder and ash, to a depth of ≈ 1.6 m bgl, followed by a loose stratum of grey fine to coarse sand and gravel, becoming very silty between 2.3 m and 3.4 m bgl. A layer of dark brown fibrous peat, 0.4 m thick, is present below 3.4 m bgl, overlying soft grey sandy very silty clay to the bottom of the borehole at 4.0 m bgl. SPT \( N_{SPT} \) values of ≈ 5 were typical during penetration in the loose sand and soft clay and peat layers at depths of 1.6 m or greater. A dense obstruction was encountered at 2 m bgl within the loose sand and gravel, resulting in a temporarily high \( N_{SPT} \) value (≫ 50) between 1.6 m and 2.3 m bgl. Groundwater was observed at ≈ 2 m bgl approximately 20 minutes after borehole construction.

![Figure 4.4 - (a) Soil profile at borehole BH04 and (b) corresponding SPT profile – Pontarddulais](image-url)
4.2.4 Cone penetration tests

A series of cone penetration tests (CPT) were conducted by Fugro Ltd. in October 2010 at several locations across the site (see Figure 4.2), including at the test pile area. The tests were performed by Fugro Ltd. using a 36 mm diameter piezocone, enabling measurement of pore pressures during penetration, with reaction provided by a 20 tonne CPT truck.

Figure 4.5 presents the results of CPT101 which was conducted directly on the location of the subsequently-installed instrumented pile test; the inferred soil profile comprised ≈ 1.8 m of made ground (sand and gravel), overlying soft clay between 1.8 m and 4.7 m bgl which in turn was underlain by medium dense sand to a depth of ≈ 6.1 m bgl. A further layer of firm clay was present between 6.1 m and 7.3 m bgl, followed by loose to medium dense sand becoming dense below 9.1 m until the test was terminated at 10.4 m bgl. High friction ratios ($R_f > 5\%$) and positive pore pressures observed during penetration in the soft clay layer between 2.0 m and 4.1 m bgl were indicative of amorphous peat (Lunne et al., 1996). A negative pore pressure response was evident at depths in excess of 4.1 m bgl, which may be the result of insufficient saturation of the piezocone.

The CPT classification chart by Robertson (1990), in which the normalised cone resistance $Q_t = (q_t - \sigma_{v0})/\sigma'_{v0}$ is plotted against normalised friction ratio $F_r = f_s/(q_t - \sigma_{v0})$, was used to assess the stress history of each layer; Figure 4.6 shows the normalised data of each layer for CPT101, while a summary of the soil types associated with each zone of the chart is given in Table 4.1. The majority the data between 1.8 m and 7.3 m bgl plots to the upper-right of the chart, indicating potential overconsolidation or ageing effects. In contrast, the sand layer between 7.3 m and 10 m bgl (in which the base of the instrumented DCIS test pile was socketed) predominantly plots in the central normally consolidated zone.
Figure 4.5 - Results of CPT101 - Pontarddulais
As laboratory tests were not performed as part of the ground investigation, the soil parameters for pile design were derived using existing popular correlations with CPT data. The relative density $D_r$ of each granular layer was estimated using the following correlation with CPT $q_c$ proposed by Jamiolkowski et al. (2003):

$$D_r = 0.35 \ln \left( \frac{q_{c1N}}{20} \right)$$

where $q_{c1N} = (q_c/p_{ref})(\sigma'_v/p_{ref})^{0.5}$ and $p_{ref} = 100$ kPa.
The undrained shear strength $s_u$ of the clay layers was derived from the measured CPT $q_c$ profile using the following expression:

$$s_u = \frac{q_t - \sigma_v}{N_k}$$  \hspace{1cm} 4.2$$

where $q_t$ is the cone resistance corrected for unequal end effects, $\sigma_v$ is the total vertical effective stress and $N_k$ is the cone factor. The value of $N_k$ depends on the properties of the clay and typically ranges from 12 to 17. However, given the relatively limited data regarding the clay layer properties at Pontarddulais (as well as the other layered sites described in forthcoming sections), a value of $N_k = 15$ was assumed when estimating $s_u$ for each layer using Equation 2.2, based on the assumptions by Lehane et al. (2013) for deriving $s_u$ profiles from CPT data during compilation of a database of load tests on displacement piles in clay.

Table 4.2 provides a summary of the soil parameters obtained for each layer using CPT correlations given by Equations 4.1 and 4.2.

### Table 4.2 - Summary of soil layer parameters based on CPT101 - Pontarddulais

<table>
<thead>
<tr>
<th>Stratum No.</th>
<th>Depth (m)</th>
<th>Soil Description</th>
<th>Bulk unit weight $\gamma_b$ (kN/m$^3$)</th>
<th>Relative density $D_r$ (%)</th>
<th>Undrained shear strength $s_u$ ($N_k = 15$) (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0 - 1.8</td>
<td>Made ground</td>
<td>19</td>
<td>75</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>1.8 - 4.7</td>
<td>Soft clay/peat</td>
<td>16</td>
<td>-</td>
<td>35</td>
</tr>
<tr>
<td>3</td>
<td>4.7 - 6.1</td>
<td>Medium dense sand</td>
<td>19</td>
<td>50</td>
<td>-</td>
</tr>
<tr>
<td>4</td>
<td>6.1 - 7.3</td>
<td>Firm clay</td>
<td>18</td>
<td>-</td>
<td>65</td>
</tr>
<tr>
<td>5</td>
<td>7.3 - 9.3</td>
<td>Loose Sand</td>
<td>19</td>
<td>20</td>
<td>-</td>
</tr>
<tr>
<td>6</td>
<td>9.3 - 10.2</td>
<td>Medium dense to dense sand</td>
<td>19</td>
<td>50</td>
<td>-</td>
</tr>
</tbody>
</table>

2. $D_r = 0.35\ln(q_{c10}/20)$ (Jamiolkowski et al., 2003)
3. $s_u = (q_t - \sigma_v)/N_k$
4.3 DAGENHAM, ENGLAND

4.3.1 Site location

The test site was located at Beach Ream Industrial Park (National Grid reference TQ503 826), approximately 1.6 km southeast of Dagenham, Essex. The site has a total area of ≈ 12 hectares and is bounded to the north by the Channel Tunnel Rail Link (CTRL) railway line, to the south by the A13 Choats Manor Way road, to the west by Thames Avenue and Marsh Way to the east.

A general overview of the western section of the site (where the instrumented DCIS pile test was located) is shown in Figure 4.7, while Figure 4.8 illustrates the layout of the instrumented DCIS test pile area. The site was previously occupied by Ford Motors as part of their adjacent car manufacturing facility at Thames Avenue and contained several small derelict buildings, stockpiles of debris and overgrown vegetation. Clearance works began in the summer of 2011 to facilitate the construction of a large distribution centre for a major supermarket chain.

The topography of the site was relatively flat prior to commencement of clearance works, with the western section having an average elevation of 1.9 m AOD. The underlying soft alluvial deposits (see Section 4.3.2) could not support the anticipated loads of the new structure, so driven cast-in-situ pile foundations were constructed into the underlying river terrace gravels. The instrumented DCIS pile test was performed to verify design parameters for the contract piles, the results of which are presented in Section 5.3 of Chapter 5.

4.3.2 Geology

The bedrock geology of the area comprises chalk, overlain by dense sand (known as Thanet Sand), followed by gravelly sand and clay of the Lambeth Beds. The superficial strata comprises flood plain gravel (referred to as river terrace deposits) containing sand, silt, clay and flint gravel, followed by Flandrian deposits of soft blue to grey silt and clay (i.e. alluvium) and interbedded layers of peat which formed as a result of rising sea levels and temperatures between 2000 and 10000 years before present (Marsland, 1986).
Figure 4.7 - Site layout - Dagenham

Figure 4.8 - Test pile layout - Dagenham
4.3.3 Ground investigation

A ground investigation was conducted by Dunelm Geotechnical & Environmental Ltd. in September 2010, comprising over 40 cable percussive boreholes at various locations across the site (see Figure 4.7). Boreholes BH108 and 109A were located within ≈ 15 m of the instrumented DCIS test pile area and the general soil profile inferred from these boreholes is shown in Figure 4.9(a), while the corresponding profiles of SPT $N_{SPT}$ with depth are illustrated in Figure 4.9(b). The soil strata comprises ≈ 1.4 m of made ground, containing light grey sand and gravel, overlying loose green grey silty fine sand to a depth of ≈ 2.1 m bgl. An alluvial cohesive layer exists between 2.1 m and 4.7 m bgl, which can sub-divided into an upper layer of soft spongy dark brown pseudo-fibrous peat, approximately 1.6 m thick and characterised by $N_{SPT}$ values of ≈ 6, and a lower soft sandy clay layer between 3.9 m and 4.7 m bgl. The soft alluvial layer is underlain by dense silty sand and gravel which is present for the remainder of the borehole (which was terminated at 11 m bgl). Some variability in measured $N_{SPT}$ values exists between the two boreholes in the upper layer of the sandy gravel, although values appear to converge to ≈ 25 blows per 300 mm below 8.5 m. Groundwater was observed at 4.0 m bgl in both boreholes approximately 20 minutes after construction, with a perched water table present at 1.5 m bgl in Borehole 109A.

![Figure 4.9 - (a) Soil profile at borehole BH108 and (b) corresponding SPT profile – Dagenham](image-url)
4.3.4 Cone penetration tests

A series of CPTs were conducted by Fugro Ltd. at the instrumented DCIS test area (see Figure 4.8) in September 2011. The elevation of the site at the time of the tests had been increased to 3.45 m AOD due to the placement of a granular piling platform to accommodate the DCIS piling rigs. The tests were performed using a 36 mm diameter piezocone, with a 20 tonne CPT truck providing reaction during each test. With the exception of CPT A6, each CPT sounding met refusal during penetration into the dense sandy gravel at depths of between 7.1 m and 7.7 m bgl.

Figure 4.10 shows the profiles of cone resistance, sleeve friction, friction ratio and pore pressure during CPTA2 at the instrumented DCIS pile location. The stratigraphy inferred from the results comprises a medium dense to dense made ground layer between the surface and 3.4 m bgl, which is underlain by ∼ 0.5 m of firm to soft clay, followed by soft to firm peat to a depth of ∼ 6.0 m bgl. Soft to firm clay exists between 6.0 m and 7.1 m bgl, after which a sharp increase in $q_c$ occurs as the cone entered dense to very dense sand and gravel, with refusal ($q_c \approx 50$ MPa) occurring at a depth of 7.7 m bgl. A positive pore pressure response was observed during penetration in the made ground layer between the surface and 3.4 m bgl, but gradually reduced as the cone penetrometer entered the soft alluvial clay and peat layers, becoming negative below a depth of ∼ 4.2 m bgl. This behaviour is similar to the pore pressure response observed in the cohesive layers at Pontarddulais (which was also performed by the same CPT contactor).

The normalised CPT data is plotted on the soil classification chart by Robertson (1990) in Figure 4.11 which suggests that the made ground and alluvial layers are overconsolidated, with the layer of soft to firm clay between 6.0 m and 7.1 m bgl classified as organic soil. In contrast to the upper layers, the sand and gravel layer below 6.0 m plots in the normally consolidated region of Zone 6 (i.e. clean sand to silty sand).

Due to the lack of information on soil properties, the relative density $D_r$ of the granular layers was derived using Equation 4.1 (Jamiolkowski et al., 2003), while Equation 4.2 was used to estimate the undrained shear strength of the alluvial deposits using a cone factor $N_k = 15$; the resulting soil parameters for each layer are summarised in Table 4.3.
Figure 4.10 - Results of CPTA2 - Dagenham
Figure 4.11 - Soil classification - Dagenham

Table 4.3 - Summary of soil layer parameters based on CPTA2 - Dagenham

<table>
<thead>
<tr>
<th>Stratum No.</th>
<th>Depth (m)</th>
<th>Soil Description</th>
<th>Bulk unit weight $\gamma_b$ (kN/m$^3$)</th>
<th>Relative density $D_r$ (%)</th>
<th>Undrained shear strength $s_u$ (kPa) (N$^k$ = 15)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0 - 3.4</td>
<td>Medium dense to dense sand</td>
<td>19</td>
<td>95</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>3.4 - 3.9</td>
<td>Soft to firm clay</td>
<td>16</td>
<td>-</td>
<td>50</td>
</tr>
<tr>
<td>3</td>
<td>3.9 - 6.0</td>
<td>Soft to firm peat</td>
<td>16</td>
<td>-</td>
<td>20</td>
</tr>
<tr>
<td>4</td>
<td>6.0 - 7.1</td>
<td>Soft to firm clay</td>
<td>16</td>
<td>-</td>
<td>15</td>
</tr>
<tr>
<td>5</td>
<td>7.1 - 7.7</td>
<td>Dense to very dense sand</td>
<td>19</td>
<td>85</td>
<td>-</td>
</tr>
</tbody>
</table>

2. $D_r = 0.35\ln(q_{tc}/20)$ (Jamiolkowski et al., 2003)
3. $s_u = (q_t-\sigma_{vo})/N_k$
4.4 ERITH, ENGLAND

4.4.1 Site location
The test site was located in Belvedere Industrial Estate (National Grid reference TQ506 795), approximately 1.5 km north of Erith, Kent. The site, which occupies a total area of ≈ 5 hectares, is bounded to the north and south by industrial plants, to the northeast by the River Thames and to the west and northwest by Bronze Access Way and Churchmanor Way roads respectively. Access to the site is provided via Churchmanor Way. An overview of the site, including the location of the various ground investigation tests, is shown in Figure 4.12, while Figure 4.13 presents a layout of the instrumented DCIS pile test area. The results of the pile test are described in detail in Section 5.4 of Chapter 5.

The site was previously owned by a high-voltage cable manufacturing company whose premises were located adjacent to the site, and contained a car park and drainage pond, as well as berthing facilities at the boundary with the River Thames to enable goods from the cable plant to be loaded onto vessels. The site was subsequently cleared in September 2012 to facilitate construction of a large distribution warehouse which was founded on driven cast-in-situ piles.

4.4.2 Geology
The geology of the site is similar to that described at Dagenham which is located ≈ 3 km to the northwest. The bedrock is primarily composed of chalk deposits which are overlain by the Thanet Sand Formation, followed by gravelly sand and clay of the Lambeth Beds. Superficial deposits comprise alluvial clay and silt, with bands of fibrous peat. The thickness of the alluvial deposits is considerably greater than at Dagenham due to the proximity of the River Thames.
Figure 4.12 - Site layout - Erith

Figure 4.13 - Pile test layout - Erith
4.4.3 Ground investigation

A site investigation was conducted by WSP Environmental Ltd. in June 2004 as part of a larger ground investigation of an adjacent derelict area to the west of the site. The investigation included a total of 13 no. trial pits and 2 no. cable percussive boreholes across the site, the locations of which are shown in Figure 4.12. The trial pits were excavated to maximum depths of 4.7 m bgl.

The soil profile of BH02, located ≈ 100 m from the location of the instrumented DCIS pile test, is presented in Figure 4.14(a) and the corresponding SPT $N_{SPT}$ profile is shown in Figure 4.14(b). The stratigraphy comprised a made ground layer with a maximum thickness of 1.8 m, overlying soft grey and brown clay, with bands of fibrous peat, to a maximum depth of 9.9 m bgl, which in turn was underlain by medium dense sandy fine to coarse angular flint gravel (Lambeth Beds) to a depth of ≈ 13.8 m bgl, followed by dense fine grey sand (Thanet Sand).

The clay can be sub-divided into two layers – an upper layer of firm to stiff mottled grey clay, ≈ 2 m thick, overlying soft moist brown to grey silty clay containing significant portions of fibrous peat. The silt content of lower clay layer was also observed to increase with depth. Average SPT $N_{SPT}$ values of ≈ 2 were observed between 2.0 m and 8.0 m bgl, highlighting the soft nature of the material. The gravel particles of the Lambeth Beds were described as angular and coarse. SPT $N_{SPT}$ values were typically greater than 30 in the granular deposits below a depth of ≈ 10 m.

Groundwater strikes were observed at depths of between 8.45 m and 9.9 m bgl, coinciding with the interface between the alluvial clay layer and the Lambeth Beds. However, the levels rose to between 2.7 m and 3.0 m bgl approximately 20 minutes after striking, indicating that the groundwater in the sand and gravel is confined by the low permeability of the overlying alluvial layer.
4.4.4 Cone penetration tests

A total of 4 CPTs were conducted by Lankelma Ltd. in November 2012 at various locations across the site (including the instrumented test pile area). The tests were conducted using a 44 mm diameter piezocone with a 20.5 tonne truck providing reaction during each test. The results of CPT03, which was located ≈ 5.5 m from the instrumented DCIS test pile, are shown in Figure 4.15; the inferred soil profile comprises a thin (≈ 0.4 m) layer of made ground at the surface, overlying soft alluvial clay between 0.4 m and 8.0 m bgl, with bands of fibrous peat \( (F_r > 5\%) \) between 0.75 and 3.0 m bgl, and once again between 7.0 and 8.1 m bgl. A layer of sand and gravel is present below 8.1 m bgl, with \( q_c \) values fluctuating between 5 MPa and 27 MPa. Higher \( q_c \) values are reached in the dense Thanet Sands, with a maximum of 58 MPa at ≈ 14 m bgl. However, \( q_c \) values subsequently reduce to ≈ 25 MPa for the remainder of the sounding.

Large excess pore pressures were evident during penetration in the alluvial clay layer, although the magnitude of \( \Delta u \) reduced in proximity to the underlying granular layer. Pore pressures remained hydrostatic (assuming the piezometric surface was located at 1 m bgl) in the sands and gravels of the Lambeth Beds until a depth of ≈13 m bgl, at which point negative pore pressures were observed for the remainder of the sounding.
Figure 4.15 - Results of CPT03 - Erith
Figure 4.16 shows the normalised CPT parameters plotted on the soil classification chart by Robertson (1990); the majority of the data from the alluvial layer plots in Zone 3 (i.e. clay and silty clay), with the sands and gravels of the Lambeth Beds between 8.2 m and 12.9 m bgl plotting in the normally consolidated region of Zones 6 and 7, indicating clean dense sand and gravel. The Thanet Sand layer below 12.9 m plots in the upper section of Zone 6, implying a high degree of over-consolidation. Table 4.4 provides a summary of the estimated relative density and undrained shear strength for the granular and cohesive layers respectively.

![Figure 4.16 - Soil classification – Erith](image)

**Table 4.4 - Summary of soil layer parameters based on CPTE3 - Erith**

<table>
<thead>
<tr>
<th>Stratum No.</th>
<th>Depth (m)</th>
<th>Soil Description</th>
<th>Bulk unit weight $\gamma_b$ (kN/m$^3$)</th>
<th>Relative density $D_r$ (%)</th>
<th>Undrained shear strength $s_u$ (kPa) ($N_k = 15$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0 - 0.5</td>
<td>Medium dense silty sand</td>
<td>19</td>
<td>80</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>0.5 - 2.2</td>
<td>Soft to firm clay</td>
<td>16</td>
<td>-</td>
<td>35</td>
</tr>
<tr>
<td>3</td>
<td>2.2 - 2.8</td>
<td>Very soft peat</td>
<td>16</td>
<td>-</td>
<td>10</td>
</tr>
<tr>
<td>4</td>
<td>2.8 - 8.2</td>
<td>Soft to firm clay</td>
<td>16</td>
<td>-</td>
<td>20</td>
</tr>
<tr>
<td>5</td>
<td>8.2 - 12.9</td>
<td>Dense sand</td>
<td>19</td>
<td>75</td>
<td>-</td>
</tr>
<tr>
<td>6</td>
<td>12.9 - 17.2</td>
<td>Very dense sand</td>
<td>19</td>
<td>95</td>
<td>-</td>
</tr>
</tbody>
</table>

2. $D_r = 0.35\ln(q_t/20)$ (Jamiolkowski et al., 2003)
3. $s_u = (q_t - \sigma_{vo})(N_k)$
4.4.5 Laboratory tests

Classification tests were performed by WSP Environmental Ltd. as part of the ground investigation phase on samples retrieved from trial pits and boreholes at various depths in both the alluvial and underlying granular layers. The moisture content $w$ of the alluvial deposits increased from $\approx 51\%$ at $2.0\text{ m bgl}$ to $\approx 148\%$ within the fibrous peat layer at $7.4\text{ m bgl}$, with a liquid limit $w_L$ of $\approx 45\%$ measured in the upper clay layer ($\approx 2.0\text{ m bgl}$) and between $65\%$ and $141\%$ in the lower soft clay/peat layer, while both layers had a plastic limit $w_p$ of $\approx 45\%$. The organic content of the lower clay layer between $3.8\text{ m}$ and $9.9\text{ m bgl}$, determined by the loss on ignition method, was $\approx 10.5\%$.

Sieving was performed in accordance with BS1377 (BSI, 1990) on samples retrieved in the underlying Lambeth Beds ($10.25\text{ m bgl}$) and Thanet Sands ($15\text{ m bgl}$). The resulting particle size distributions are shown in Figure 4.17; the sample retrieved from the Lambeth Beds at $10.25\text{ m bgl}$ is predominantly composed of sand ($\approx 64\%$) with the gravel and silt fractions accounting for about $25\%$ and $10\%$ respectively, while the Thanet Sands sample ($15.0\text{ m bgl}$) had a significantly greater portion of clay ($\approx 10\%$). The mean particle size $D_{50}$ reduces from $\approx 0.4\text{ mm}$ at $10.5\text{ m bgl}$ to $\approx 0.18\text{ mm}$ at $15.0\text{ m bgl}$.

![Grading curves - Erith](image-url)
4.5 SHOTTON, WALES

4.5.1 Site location
The test site was located in the northwest section of Deeside Industrial Estate (National Grid reference SJ 299 715), approximately 3 km north of the village of Shotton in Flintshire, north Wales. The site is bounded to the north and west by Weighbridge Road, to the south by a large drainage pond and to the east by a car park and a large paper mill. The River Dee is located ≈ 1.5 km to the south of the site.

Several stockpiles of rubble material were present on the site prior to the commencement of clearance works in March 2011 to facilitate construction of a new electricity station which was founded on driven cast-in-situ piles. The ground levels at the site varied between 7.5 m and 8.5 m AOD prior to clearance works, with the final ground level at 8.0 m AOD coinciding with the top of the piling platform.

An overview of the northeastern section of the site, including the instrumented DCIS pile test area, is shown in Figure 4.18, while the layout of the anchor piles, CPTs and instrumented DCIS test pile S1 is illustrated in Figure 4.19. The results of the pile test are described in detail in Section 6.2 of Chapter 6.

4.5.2 Geology
The geological history of the Deeside area, summarised by Nichol and Wilson (2002), comprises Carboniferous coal measures containing mudstone, sandstone and siltstone, which are overlain by Quaternary deposits of brown and grey boulder clay (i.e. glacial till), followed by alluvial sand, silt and gravel from the River Dee. Nichol and Wilson (2002) note that hydraulically-placed sand and pulverised fuel ash deposits are widespread in the industrial area (where the pile test site is located) to the north of the River Dee.
Figure 4.18 - Site layout - Shotton

Figure 4.19 - Test pile layout - Shotton
4.5.3 Ground investigation

A site investigation was performed by Fugro Ltd. in January and February 2009, comprising of 14 no. cable percussion boreholes and 25 no. trial pits. Standard penetration tests (SPT) were conducted during construction of the boreholes, with disturbed samples taken from the boreholes and trial pits for classification purposes and particle size analyses (see Section 4.5.5).

The general soil profile inferred from boreholes BH06 and BH10B, located ≈ 100 m from the instrumented DCIS test pile location is shown in Figure 4.20(a), while Figure 4.20(b) illustrates the corresponding SPT $N_{SPT}$ profiles of each borehole. The underlying stratigraphy comprises a layer of made ground, approximately 1.4 m thick, containing grey gravelly fine to coarse sand with occasional fragments of concrete, brick and slag, which in turn is underlain by medium dense light brown fine to medium sand to a depth of ≈ 4.6 m bgl. The density of the sand begins to increase significantly at depths in excess of 5.0 m bgl, with SPT $N_{SPT}$ values typically greater than 40 blows per 300 mm below a depth of 6.0 m. Occasional fragments of organic shells were present in the upper sections of the sand layer between 4.6 m and 6.7 m bgl. Groundwater was not observed during construction of the two boreholes in the vicinity of the instrumented DCIS test pile area.

![Soil profile and SPT profile](image)

Figure 4.20 - (a) Soil profile at borehole BH108 and (b) corresponding SPT profile – Shotton
4.5.4 Cone penetration tests

A total of 5 CPTs were conducted by InSitu Site Investigation Ltd. at the instrumented DCIS pile test area (see Figure 4.19) in May 2011. The tests were performed using a 36 mm diameter piezocone with a 21 tonne CPT truck providing reaction, and were specified to penetrate to a minimum depth of 10 m bgl.

The results of CPT01 at the test pile location are shown in Figure 4.21 where the inferred soil profile comprises ≈ 2 m of made ground, characterised by variable $q_c$ values ranging between 5 MPa and 40 MPa, overlying stiff sandy silt between 2.0 m and 2.3 bgl, which in turn overlies medium dense to dense sand. Intermittent lenses of cohesive material between the ground surface and 4.0 m bgl are evident by friction ratios $R_f$ in excess of 2 %. The sand layer becomes very dense at ≈ 5.2 m bgl, with values of cone resistance $q_c$ increasing from 15 MPa to 25 MPa.

A positive pore pressure response was observed within the made ground and sandy silt layers between the surface and 2.3 m bgl, before reducing somewhat during the initial stages of penetration in the underlying sand layer and subsequently increasing again below a depth of ≈ 4.0 m. However, pore pressures were less than the idealised hydrostatic profile (based on an assumed piezometric surface at 1.0 m bgl).

Figure 4.22 shows a plot of the normalised CPT data on the soil classification chart by Robertson (1990); the made ground layer plots in Zones 6 to 9, implying a mixture of sand, gravel and stiff clay. The majority of data for the sand layer between 2.5 m and 10.4 m bgl plots in the normally consolidated section of Zone 6 (occasional lenses of silt and clay were present between 2.5 m and 4.1 m bgl). A summary of the soil parameters for each layer is given in Table 4.5; relative densities were estimated using the correlation by Jamiolkowski et al. (2003), while the bulk unit weights were assumed based on values recommended by Salgado (2008).
Figure 4.21 - Results of CPT01 - Shotton
Table 4.5 - Summary of soil layer parameters based on CPT01 - Shotton

<table>
<thead>
<tr>
<th>Stratum No.</th>
<th>Depth (m)</th>
<th>Soil description</th>
<th>Bulk unit weight $\gamma_b$ (kN/m$^3$)</th>
<th>Relative density $D_r$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0 - 1.8</td>
<td>Made ground</td>
<td>19</td>
<td>60</td>
</tr>
<tr>
<td>2</td>
<td>1.8 - 2.5</td>
<td>Sandy silt</td>
<td>19</td>
<td>-</td>
</tr>
<tr>
<td>3</td>
<td>2.5 - 5.2</td>
<td>Medium dense to dense sand with occasional lenses of silt and clay</td>
<td>19</td>
<td>65</td>
</tr>
<tr>
<td>4</td>
<td>5.2 - 10.0</td>
<td>Very dense, locally gravelly, sand</td>
<td>19</td>
<td>85</td>
</tr>
</tbody>
</table>

2. $D_r = 0.35\ln(q_c/\sigma_0)$ (Jamiolkowski et al., 2003)
4.5.5 Laboratory tests

Particle size analyses, performed in accordance with BS 1377 (BSI, 1990), were reported for a total of 41 samples obtained from the boreholes and trial pits at various depths ranging from 1.5 m to 15.0 m bgl. Figure 4.23 presents the grading curves for samples taken from borehole BH06 and trial pit TP12 which were located in the vicinity of the test pile location; the sand was classified as uniform fine to medium sand with a mean effective particle size $D_{50} = 0.14 \pm 0.02$ mm and coefficient of uniformity $C_u = 2.17 \pm 0.3$. All samples had a fines content of less than 5%.

![Grading curves – Shotton](image)

**Figure 4.23 - Grading curves – Shotton**
4.6 RYTON-ON-DUNSMORE, ENGLAND

4.6.1 Site location

The pile test site was located at Keller Foundations head office (National Grid reference SP385 735) on the outskirts of the village of Ryton-on-Dunsmore, approximately 8 km southeast of Coventry in Warwickshire, UK. The site occupies an area of ≈ 7.5 hectares and comprises a three storey office building at the entrance to the site with a large yard at the rear containing several workshops and maintenance buildings, as well as storage areas for piling rigs and associated machinery. The site is bounded to the west by a warehouse, to the north by Coventry City Football Club training ground, to the east by a lake and to the south by the A423 Oxford Road.

The pile tests were conducted in an area at the northern end of the yard which is typically used for parking piling rigs. A general layout of the site is shown in Figure 4.24, while an overview of the instrumented DCIS pile test area is illustrated in Figure 4.25. The test area was underlain by several 250 mm wide precast concrete piles laid out horizontally immediately below the surface to provide a stable working platform for the piling machinery; the presence of these piles led to difficulties in performing the CPTs during the ground investigation in June 2013 and were subsequently excavated in October 2013, with the original site levels at the time of the CPTs subsequently restored by back-filling the area with made ground prior to installing the anchor piles and instrumented test piles. The results of instrumented pile tests are described in detail in Section 6.3 of Chapter 6.

4.6.2 Geology

The geological history of the Warwickshire area comprises Triassic Mercia Mudstone bedrock which is overlain by superficial Pleistocene deposits of boulder clay, river terrace gravels and alluvial sands; the complex formation of these drift deposits is described extensively by Shotton (1953). The majority of the Ryton-on-Dunsmore area (including the test site location) is underlain by Baginton Sand, described as rusty brown in colour with occasional black-stained layers, which was deposited within a valley which traversed the area in a north easterly direction (Shotton, 1953).
Figure 4.24 - Site layout - Ryton-on-Dunsmore

Figure 4.25 – Test pile layout - Ryton-on-Dunsmore
4.6.3 Cone penetration tests

A total of three CPTs were conducted by InSitu Site Investigation Ltd. in June 2013 to establish the ground conditions in the test area. The CPTs were performed using a 36 mm diameter piezocone with a 21 tonne truck providing reaction for the tests and penetrated to maximum depths of between 8.3 m and 9.7 m bgl (the details of each test are summarised in Table 4.6). Figure 4.26 shows the average profiles of cone resistance $q_c$, sleeve friction $f_s$, friction ratio $R_f$ and pore pressure $u_2$ at the test location. The inferred soil profile comprises $\approx 0.5$ m of dense made ground, overlying stiff sandy silt to a depth of $\approx 1.8$ m bgl, which in turn is underlain by medium dense to dense sand. Values of $q_c$ in the sand layer were observed to increase with depth from $\approx 10$ MPa at 2 m bgl to $\approx 20$ MPa at 3.2 m bgl, before reducing again to a minimum of $\approx 10$ MPa at 4.5 m bgl. Variable values of $q_c$ values were evident below 6.5 m due to the presence of dense gravelly sand. Pore pressures were generally negligible ($<10$ kPa) during penetration in all layers, with a dissipation test conducted at 7.0 m bgl during CPT01 to assess if the pore pressure response was a result of insufficient saturation of the piezocone; pore pressures remained negligible during the 5 minute period dissipation period, suggesting that the deposit is essentially unsaturated.

A seismic cone penetration test (SCPT) was also performed in a central position within the test area to determine the small-strain shear stiffness $G_0$ of the soil. Shear waves were generated by striking a ground beam beneath the CPT truck with a hammer at 0.5 m intervals of penetration (with the exception of 2.5 m bgl where the seismic cone was unable to successfully measure the generated shear waves). Values of $G_{0,SCPT}$ were subsequently derived using the expression $G_{0,SCPT} = \rho V_s^2$ where $\rho$ is the density of the soil and $V_s$ is the shear wave velocity measured by the seismic cone. The resulting profile of $G_{0,SCPT}$ (assuming $\rho = 1890$ kg/m$^3$) with depth is also shown in Figure 4.26 where a relatively constant $G_{0,SCPT}$ of $\approx 100$ MPa was evident below a depth of 3.5 m bgl.

<table>
<thead>
<tr>
<th>CPT</th>
<th>Date of test</th>
<th>Maximum depth (m bgl)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>01</td>
<td>10th June 2013</td>
<td>9.5</td>
<td>Dissipation test conducted at 7.0 m bgl</td>
</tr>
<tr>
<td>02</td>
<td>10th June 2013</td>
<td>9.7</td>
<td>Refusal in gravelly sand</td>
</tr>
<tr>
<td>03</td>
<td>10th June 2013</td>
<td>8.3</td>
<td>Refusal in gravelly sand</td>
</tr>
<tr>
<td>SCPT</td>
<td>10th June 2013</td>
<td>9.0</td>
<td>Shear wave measurements at 0.5 m intervals</td>
</tr>
</tbody>
</table>
Figure 4.26 - Results of average CPT - Ryton-on-Dunsmore
Figure 4.27 shows the normalised CPT data plotted on the soil classification chart by Robertson (1990); the majority of the sand layer between 1.8 m and 8.3 m bgl plots in the normally consolidated section of the clean sand (i.e. Zone 6), while the made ground and sandy silt layers are classified as stiff, overconsolidated sand to clayey sand. A summary of the assumed bulk unit weights (Salgado, 2008) and relative density (obtained using Equation 4.1) is provided in Table 4.7.

![Soil classification chart](image)

**Figure 4.27 - Soil classification - Ryton-on-Dunsmore**

<table>
<thead>
<tr>
<th>Stratum No.</th>
<th>Depth (m)</th>
<th>Soil Description</th>
<th>Bulk unit weight $\gamma_b^1$ (kN/m$^3$)</th>
<th>Relative density $D_r^2$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0 - 0.5</td>
<td>Dense sand (made ground)</td>
<td>19</td>
<td>100</td>
</tr>
<tr>
<td>2</td>
<td>0.5 - 1.8</td>
<td>Stiff to very stiff clay</td>
<td>19</td>
<td>-</td>
</tr>
<tr>
<td>3</td>
<td>1.8 - 8.3</td>
<td>Medium dense to dense sand</td>
<td>19</td>
<td>70</td>
</tr>
</tbody>
</table>

2. $D_r = 0.35\ln(q_t/c_v/20)$ (Jamiolkowski et al., 2003)
4.6.4 Laboratory tests

Disturbed samples of clay and sand were retrieved in a somewhat unconventional manner by driving a steel DCIS installation tube open-ended (i.e. without the sacrificial base plate) to a depth of 3.5 m bgl at a position approximately 2 m from the location of Pile R3. The tube became fully plugged during driving in the sand layer and was subsequently withdrawn and subjected to a number of blows after extraction in order to retrieve the plugged soil. Inspection of the samples immediately after extraction indicated that the clay and sand layers were unsaturated. The soil was placed into air-tight containers and subsequently transported to the geotechnical laboratory at NUI Galway for further analysis and testing.

Classification tests on the sand and clay samples were conducted in accordance with BS 1377 (BSI, 1990). Five samples were used to determine the specific gravity \( G_s \) of the sand particles using the gas jar method, resulting in an average \( G_s \) value of 2.65 ± 0.01. The minimum \( e_{\text{min}} \) and maximum void ratio \( e_{\text{max}} \) of the sand was 0.49 and 0.85 respectively. Visual inspections of the particles indicated that they were predominantly sub-rounded to sub-angular. The moisture contents \( w \) of the sand and clay were ≈ 5 % and ≈ 16 % respectively. The clay has a liquid limit \( w_L \) of ≈ 26 % and a plastic limit \( w_p \) of ≈ 15 %, and is therefore described as a clay of low plasticity, based on the plasticity chart of BS 5930 (BSI, 1999).

The grading characteristics of the sand were assessed by dry sieving ten samples in accordance with BS 1377 (BSI, 1990). Figure 4.28 shows the average grading curve obtained; the soil was classified as brown uniform fine to medium sand, with a mean effective particle size \( D_{50} \) of 0.3 mm and uniformity coefficient \( C_u (= D_{60}/D_{10}) \) of 1.69. The average portion passing the 63 μm sieve was ≈ 3%.

A total of 20 sand-on-sand direct shear tests were conducted on reconstituted samples at various densities and normal stresses ranging from 40 kPa to 100 kPa using a Wykeham Farrance 25400 Series shear box apparatus. Dense samples were prepared by air pluviation in three layers using a funnel with a drop height of ≈ 20 mm, with each layer subjected to 100 blows using a wooden tamper. Loose samples were obtained using the procedure by Miura et al. (1997) whereby the sand was placed using a funnel which was slowly lifted from the base of the mould with the tip of the funnel remaining in contact with the placed sand in order to
minimise the drop height (and hence density), after which the sample was levelled using a hand-held vacuum device prior to placement of the upper grid plates of the shear box. Samples were sheared at a rate of 1 mm per minute to a maximum horizontal displacement of 12 mm. Figure 4.29 shows the resulting variation in friction angle with void ratio $e$ where a constant-volume friction angle $\phi_{cv}'$ of $\approx 35^\circ$ was obtained which was relatively independent of void ratio and in reasonable agreement with $\phi_{cv}' = 33^\circ$ recommended for quartz sands by Bolton (1986).

Figure 4.28 - Average grading curve for sand samples - Ryton-on-Dunsmore

Figure 4.29 - Variation in sand friction angle with void ratio - Ryton-on-Dunsmore
4.7 SUMMARY

This chapter presented the ground conditions at the five sites in which instrumented DCIS piles were installed and tested. While three of the sites contain significant layers of soft cohesive soils, the shaft and base resistance generated in the granular layers is of most interest to the author during the subsequent DCIS pile tests in Chapters 5 and 6; a summary of the soil properties of the granular layers at each site is given in Table 4.8 while Figure 4.30 shows a comparison of the normalised CPT data for all sites plotted on the soil classification chart by Robertson (1990).

Table 4.8 - Summary of granular layers at all test sites

<table>
<thead>
<tr>
<th>Site</th>
<th>Depth (m)</th>
<th>Soil Description</th>
<th>Bulk unit weight $\gamma_b$ (kN/m$^3$)</th>
<th>Relative density $D_r$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pontarddulais</td>
<td>4.7 - 6.1</td>
<td>Medium dense sand</td>
<td>19</td>
<td>50</td>
</tr>
<tr>
<td>Pontarddulais</td>
<td>7.3 - 9.3</td>
<td>Loose sand</td>
<td>19</td>
<td>20</td>
</tr>
<tr>
<td>Dagenham</td>
<td>0 - 3.4</td>
<td>Medium dense to dense sand</td>
<td>19</td>
<td>95</td>
</tr>
<tr>
<td>Dagenham</td>
<td>7.1 - 7.7</td>
<td>Dense to very dense sand</td>
<td>19</td>
<td>85</td>
</tr>
<tr>
<td>Erith</td>
<td>8.2 - 12.9</td>
<td>Dense sand</td>
<td>19</td>
<td>75</td>
</tr>
<tr>
<td>Shotton</td>
<td>2.5 - 5.2</td>
<td>Medium dense to dense sand</td>
<td>19</td>
<td>65</td>
</tr>
<tr>
<td>Shotton</td>
<td>5.2 - 10.0</td>
<td>Very dense, locally gravelly, sand</td>
<td>19</td>
<td>85</td>
</tr>
<tr>
<td>Ryton-on-Dunsmore</td>
<td>1.8 - 8.3]</td>
<td>Medium dense to dense sand</td>
<td>19</td>
<td>70</td>
</tr>
</tbody>
</table>

2. $D_r = 0.35\ln(q_{tc}/20)$ (Jamiolkowski et al., 2003)

Figure 4.30 - Granular soil layer classification - all test sites
CHAPTER 5  TESTS ON INSTRUMENTED DCIS PILES WITH GRANULAR SOCKETS
5.1 INTRODUCTION
This chapter presents the results of a series of tests conducted on instrumented DCIS piles at Pontarddulais, Dagenham and Erith in the United Kingdom. These piles are grouped together in this chapter as all three rely heavily upon a socket in granular soil for their capacity. General details of each test pile are summarised, followed by the results of the installation, curing and compressive maintained load test phases, and consideration of shaft and base behaviour. Much of this interpretation draws upon data from the strain gauges within the pile. The experimental procedures for each of these phases were described in detail in Chapter 3, while Chapter 4 provided a detailed description of the soil conditions and parameters relevant to pile behaviour at each test site.

5.2 INSTRUMENTED PILE TEST AT PONTARDDULAI, WALES

5.2.1 Introduction
This section describes the installation, curing and maintained compression load testing of an instrumented DCIS pile at Pontarddulais, Wales in November 2010. Details of the test pile, designated Pile P1, are summarised in Table 5.1, while Figure 5.1 presents a schematic of the test pile (including strain gauge levels and pile cap dimensions) and the corresponding ground profile inferred from CPT101 at the pile location. The pile had a nominal shaft diameter of 320 mm and a length of 8.5 m, and was socketed ≈ 1.2 m into the relatively loose stratum of sand ($D_r \approx 20\%$) at 7.3 m below ground level (bgl). Instrumentation comprised vibrating wire sister bar strain gauges in groups of four at depths of 2.0 m, 4.5 m, and 6.25 m bgl in order to measure the reduction in load across each soil layer, with an additional level of four gauges placed in the granular socket approximately 0.5 m from the base of the pile. These gauge levels were not chosen by the author (the only one of the series where this happened) however and were not placed in optimal positions to enable an accurate assessment of the load transfer characteristics of the pile (particularly the base resistance) as will be highlighted later.
Table 5.1 - Test pile details - Pile P1

<table>
<thead>
<tr>
<th>Test pile details</th>
<th>Pile P1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pile reference</td>
<td>Pile P1</td>
</tr>
<tr>
<td>Length $L$ (m)</td>
<td>8.50</td>
</tr>
<tr>
<td>Nominal shaft diameter $D_s$ (mm)</td>
<td>320</td>
</tr>
<tr>
<td>Nominal base diameter $D_b$ (mm)</td>
<td>380</td>
</tr>
<tr>
<td>Date of installation</td>
<td>08/11/10</td>
</tr>
<tr>
<td>Date of static load test</td>
<td>17/11/10</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Reinforcement details</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Main reinforcement</td>
<td>4 x B20 x 8.7 m</td>
</tr>
<tr>
<td>Shear reinforcement</td>
<td>240 mm OD B8 helical cage at 250 mm pitch</td>
</tr>
<tr>
<td>Cover (mm)</td>
<td>40</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Concrete details</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Mix</td>
<td>C35/45</td>
</tr>
<tr>
<td>28-day characteristic cube</td>
<td></td>
</tr>
<tr>
<td>compressive strength $f_{cu}$ (MPa)</td>
<td>45</td>
</tr>
<tr>
<td>Maximum aggregate size</td>
<td>10 mm</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Piling rig details</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Keller DCIS piling rig reference</td>
<td>3190</td>
</tr>
<tr>
<td>Hammer</td>
<td>Junttan HHK5ASS</td>
</tr>
<tr>
<td>Drive tube length (m)</td>
<td>16</td>
</tr>
<tr>
<td>Drive tube dimensions (OD, ID, wall thickness $t$; mm)</td>
<td>320, 280, 20</td>
</tr>
<tr>
<td>Drive plate dimensions (D, thickness $t$; mm)</td>
<td>380, 11</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Instrumentation details</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Strain gauge type</td>
<td>Geosense G100-040 Vibrating Wire Sister Bar</td>
</tr>
<tr>
<td>Levels (m bgl)</td>
<td>2.0, 4.5, 6.25, 8.0</td>
</tr>
<tr>
<td>Normalised distance from pile tip to mid-point between gauge levels $h/D_s$</td>
<td>15.5, 9.2, 4.0</td>
</tr>
</tbody>
</table>

Figure 5.1 - Schematic of Pile P1, CPT $q_c$ profile and corresponding soil profile
5.2.2 Installation

The test pile was installed on November 8th 2010 using a 320 mm outer diameter, 20 mm thick, open-ended steel driving tube, fitted with a 380 mm diameter steel driving plate at the base, which was driven to a final depth of 8.5 m using a Junttan HHK5ASS hydraulic hammer. No driveability issues were encountered during installation. The total number of blows recorded during installation and corresponding number of blows per 250 mm of penetration are shown in Figure 5.3(a) and 5.3(b) respectively, while Figure 5.3(c) shows the variation in installation resistance (derived from the piling rig instrumentation) with depth during driving. It is evident that:

- The majority of installation resistance, designated \( q_{cp} \), during driving was provided by the made ground and sand layers, with plunging of the tube occurring during penetration in the soft peat and clay layer between 2.0 m and 4.5 m bgl.
- Typical sets per blow of between 60 mm and 100 mm were evident below 7.3 m bgl, highlighting the loose state of the sand in which the pile was socketed.
- The installation resistance profile (measured by the piling rig instrumentation) showed good resemblance with the base resistance \( q_{b,0.1Db} \) of a closed-ended driven displacement pile predicted by the UWA-05 method (Lehane et al., 2005c) described in Section 2.5.3 of Chapter 2 using the cone resistance \( q_c \) profile at the test pile location (CPT101).

The hammer was retracted when the tube reached the final embedment depth of 8.5 m bgl, after which C35/45 strength, high-slump concrete, with a maximum aggregate size of 10 mm, was poured into the top of tube using a specialised skip. The tube was then extracted without difficulty, with the sacrificial driving plate remaining at the base of the pile. The instrumented reinforcement cage was carefully inserted into the freshly-cast concrete, with a set of strain readings taken after insertion to provide reference readings for assessing strain behaviour during curing (see Section 5.2.3). Finally, the soil surrounding the pile head was excavated to a depth of 500 mm to accommodate a 700 mm x 700 mm, 900 mm deep, square concrete pile cap, with the strain gauge wiring threaded out through one of the corners of the cap in order to provide clearance for the hydraulic jack and displacement transducers for the subsequent maintained load test described in Section 5.2.4.
Figure 5.2 - Installation data for Pile P1
5.2.3 Curing

The pile was left to cure for a period of 9 days to allow the concrete to gain sufficient strength prior to conducting the maintained compression load test. While variations in strain and temperature during this period were not monitored continuously, the strain readings taken immediately after casting and once again prior to commencing the static load test enabled the net change in strain at each gauge level to be determined which is shown in Figure 5.3(a):

- A large tensile strain of ≈ 75 με was present at the upper-most level at 2.0 m bgl.
- Tensile strains reduced with depth, reaching a peak compressive value of ≈ 7 με at 6.25 m bgl, before becoming slightly tensile (≈ 16 με) again near the base.

As will be subsequently shown during curing of the instrumented pile at Dagenham (see Section 5.3.3), the reduction in tensile strain with depth implies that compressive strains developed during curing due to the presence of residual loads within the pile as the cohesive layers settled after consolidation. Unfortunately, the precise magnitude of such loads could not be determined directly as strain was not monitored continuously after casting. The negative local shear stresses $\tau_{s,\text{res}}$ mobilised along the shaft between the surface and the clay/sand interface at 7.3 m bgl were therefore estimated using the expression $\tau_{s,\text{res}} = 0.3\sigma'_v$0 (Meyerhof, 1976). The resulting residual load distribution is shown in Figure 5.3(b) where a maximum compressive load of 117 kN was estimated at 7.3 m bgl. Positive shear stresses were likely to be mobilised in the socket to counteract the downward forces from the settling layers, resulting in a compressive residual load of ≈ 100 kN at the base of the pile.

![Figure 5.3 - (a) Net change in strain after curing of Pile P1 and (b) estimated dragload distribution](image-url)
5.2.4 Load testing

The test pile was subjected to a maintained compression load test in accordance with the Institution of Civil Engineers Specification for Piling and Embedded Retaining Walls (ICE, 2007) on November 17\textsuperscript{th} 2010. The load schedule, summarised in Appendix C, comprised two unload/reload cycles (designated Cycles 1 and 2) to maximum loads of 313 kN and 471 kN respectively, after which the pile was loaded incrementally (Cycle 3) to a maximum load of 935 kN. The total duration of the load test was \(\approx 41\) hours. Figure 5.4 shows the measured load-displacement and load-time responses at the pile head where the following was noted:

- The head displacement at the design verification load (DVL) of 313 kN was \(\approx 4\) mm, which was within the maximum permitted displacement of 5 mm.
- Reloading resulted in a stiff linear load-displacement response until the applied load exceeded the maximum load from the previous cycle.
- Creep displacements increased significantly during the latter stages of the loading as the pile approached failure.
- While a plunging failure did not occur, the pile displacement was \(\approx 37\) mm at the maximum applied load of 935 kN. Extrapolation of the load-displacement curve to a displacement of 38 mm (i.e. \(0.1D_b\)) resulted in a total pile capacity of 949 kN.

![Figure 5.4 - Load-displacement response at pile head for Pile P1](image-url)
5.2.5 Strain analysis

Figure 5.5 shows the variation in average strain (uncorrected for creep) at each gauge level with time during the load test. Strains were typically logged at 5 minute intervals for the first 30 minutes of each hold, followed by intervals of between 15 and 60 minutes thereafter, depending on the duration of the hold. It is apparent that:

- The largest strains were mobilised at the uppermost level nearest to the pile head at (2.0 m bgl) and decreased with depth towards the base of the pile, with the greatest reduction in strain occurring between 2.0 m and 4.5 m bgl (i.e. across the soft peat and clay layer); this anomaly is discussed further in Section 5.2.6.
- Minor increases in strain occurred at all levels during each load hold, particularly at 2.0 m and 4.5 m bgl during the 6 hour holds at 313 kN and 471 kN; such increases were attributed to creep within the concrete.
- With the exception of the uppermost gauge level, tensile strains were mobilised at all levels when the pile was unloaded to 0 kN, the magnitude of which increased after each successive load cycle. In contrast, the strains at 2.0 m were compressive, and increased with each successive load cycle.
- The latter stages of the test were characterised by increases in strain at 2.0 and 4.5 m bgl while values at 6.25 and 8.0 m bgl remained relatively constant.

![Figure 5.5 - Variation in strain with time during loading - Pile P1](image-url)
The elastic strain at each gauge level was derived by removing creep strains which developed within the concrete during each hold using the procedure by Lam and Jefferis (2011) described in Section 3.7.3 of Chapter 3. The resulting variation in elastic strain at each level with applied stress at the pile head is shown in Figure 5.6(a); the unload-reload cycles have been excluded for clarity purposes. The elimination of creep strains had only a minor effect of the overall strain response at each gauge level, resulting in a total reduction in strain of ≈ 20 με at 2.0 m for example.

The stress-strain response at the uppermost gauge level was used to derive the strain-dependent pile stiffness $E_p$ during loading using the secant method (described in Section 3.7.5 of Chapter 3). The variation in $E_p$ with strain at 2.0 m bgl for each loading cycle is illustrated in Figure 5.6(b) where it is apparent that:

- The pile stiffness increased with increasing strain level; such behaviour contrasts with that expected for typical concrete structures whereby stiffness reduces with increasing strain level (Neville and Brooks, 1987).
- Load cycling resulted in a higher pile stiffness in comparison to the previous cycle. Given that the duration of the test was ≈ 41 hours, time-dependent increases in stiffness may also have contributed to the higher values of stiffness in the latter stages of loading.

A polynomial was fitted to the data in Figure 5.6(b) to enable the measured elastic strains at the remaining gauge levels to be converted to axial loads using the expression $P = \varepsilon_{\text{elastic}} A_p E_p$, where $P$ is the axial load and $A_p$ is the cross-sectional area of the pile.

Figure 5.6 - Variation in elastic strain with (a) applied stress and (b) pile modulus - Pile P1
5.2.6 Shaft resistance

Figure 5.7 shows the variation in local shear stress \( \tau_{s,\text{loc}} \), inferred by assuming a linear distribution of load between two successive gauge levels, with shaft displacement \( w_s \) during loading. It is evident that:

- Negative local shear stresses were present along the shaft of the pile during the initial stages of loading; these stresses were a consequence of the residual loads which developed during curing, as mentioned previously.

- The initial reloading stage of each cycle resulted in local shear stresses along the upper levels of the shaft being considerably larger for a given load in comparison to the previous cycle. Such behaviour is clearly unrealistic and can only be attributed to the difficulties associated with interpreting the strains and pile stiffness during unloading and reloading.

- Displacements in excess of 30 mm (i.e. \( w_s/D_s > 8.8 \% \)) were required to mobilise local shear stresses in both the soft cohesive and sand layers between 2.0 m and 6.25 m bgl.

- The maximum local shear stress \( \tau_{s,\text{loc}} \) of \( \approx 150 \) kPa measured across the soft peaty clay layer between 2 m and 4.5 m bgl was over four times the estimated undrained shear strength \( s_u \) of \( \approx 35 \) kPa of the clay (using the CPT \( q_c \) profile as described in Chapter 4) and cannot therefore be considered realistic.

![Figure 5.7 - Variation in local shear stress with shaft displacement - Pile P1](image)

A possible explanation for the large reduction in strain between 2.0 m and 4.5 m bgl is an increase in pile diameter within the soft layer at 4.5 m bgl after casting; this would have resulted in lower strains being mobilised at this level due to the larger cross-section. However,
this assumption has the knock-on effect of under-predicting the local shear stress in the sand layer between 4.5 m and 6.25 m bgl. Unfortunately, the true diameter of the pile at each level could not be determined as integrity tests were not conducted and the pile was not extracted after testing.

5.2.7 Base resistance
The fact that the lowermost gauge level (8.0 m bgl) was located 0.5 m above the base of the pile prevented direct measurement of the base resistance during loading. Linear extrapolation of the load distribution from 8.0 m to 8.5 m bgl was also considered inaccurate as the shear stress measured between 6.25 m and 8.0 m bgl contained both the firm clay and loose sand layers. The reduction in load below 8.0 m during loading was therefore estimated using the hyperbolic load-transfer curve technique by Niazi and Mayne (2010) with the peak local shear stress between 8.0 m and 8.5 m bgl derived using the correlation $\tau_{sf} = q_{c}/150$ proposed by Bustamante and Gianeselli (1982). The resulting variation in base resistance $q_b$ and stiffness $E_{b,eq}$ with base displacement $w_b$ is shown in Figure 5.8(a) and 5.8(b) respectively. Observations of base behaviour include:

- A residual stress $q_{b,\text{res}}$ of $\approx 0.95$ MPa was present at the base prior to loading, corresponding to $\approx 66\%$ of the base resistance at failure $q_{b,0.1D_b} (= 1.44$ MPa)$
- Degradation in base stiffness $E_{b,eq}$ began at a base displacement of $\approx 4.5$ mm, corresponding to a normalised displacement $w_b/D_b = 1.2\%$.
- The peak base stiffness of $\approx 11.4$ MPa was substantially lower than the estimated in-situ small-strain stiffness $E_0$ of $\approx 155$ MPa at the base of the pile, derived using the correlation with the CPT cone resistance $q_c$ by Mayne (2006).

![Figure 5.8 - Variation in (a) base resistance and (b) stiffness with displacement during loading - Pile P1](image)

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5.3 INSTRUMENTED PILE TEST AT DAGENHAM, ENGLAND

5.3.1 Introduction

This section describes the results of the installation, curing and load testing of the second instrumented DCIS pile constructed in a layered soil deposit at a site in Dagenham, Essex in September 2011. The test pile had a nominal shaft diameter of 320 mm and a length of 7.7 m, and was socketed \( \approx 0.5 \) m into dense sandy gravel. A schematic of the test pile and corresponding ground profile are presented in Figure 5.9, while Table 5.2 provides a summary of the test pile details.

Instrumentation comprised vibrating wire strain gauges, in groups of four, placed at levels of 1.3 m, 3.55 m, 6.5 m and 7.5 m bgl to assess the local shear stress profile within the upper sand, alluvial cohesive layer and sandy gravel layers respectively. The lowermost strain gauges had reduced sister-bar lengths to enable placement as close to the base of the pile as possible (\( \approx 0.2 \) m).

Table 5.2 - Test pile details - Pile D1

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<td>Cover (mm)</td>
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<td>Keller DCIS piling rig reference</td>
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<td>Hammer</td>
<td>Junttan HHK5ASS</td>
</tr>
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<td>Drive tube length (m)</td>
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<tr>
<td>Drive tube dimensions (OD, ID, wall thickness ( t ); mm)</td>
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</tr>
<tr>
<td>Drive plate dimensions (OD, thickness ( t ); mm)</td>
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</table>

<table>
<thead>
<tr>
<th>Instrumentation details</th>
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</tr>
</thead>
<tbody>
<tr>
<td>Strain gauge type</td>
<td>Geosense G100-040 Vibrating Wire Sister Bar</td>
</tr>
<tr>
<td>Levels (m bgl)</td>
<td></td>
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<tr>
<td>Normalised distance from pile tip to midpoint between gauge levels ( h/D_s )</td>
<td>15.5, 7.9, 2.1</td>
</tr>
</tbody>
</table>
5.3.2 Installation

The tube installation data is presented in Figure 5.10 which is summarised as follows:

- The majority of the blows required to install the pile occurred in the made ground and sand layers between the ground surface and 3.4 m bgl, with plunging of the tube observed during driving in the soft alluvial layer between 3.4 m and 7.2 m bgl.
- The number of blows per 250 mm of penetration increased rapidly in the sandy gravel layer below 7.2 m bgl, with an installation resistance of \( \approx 14 \) MPa measured at the end-of-driving (EOD).
- The installation resistance (measured by the piling rig instrumentation) in the made ground and sand layers between the surface and 3.4 m bgl again closely resembled the corresponding profile of base resistance derived from the CPT profile using the UWA-05 method. Comparisons in the dense sandy gravel layer below 7.2 m could not be determined as the CPT cone penetrometer met refusal at 7.7 m bgl.

The pile was cast using C40/50 strength, high-slump concrete via skipping, followed by instrumented reinforcement cage installation and pile cap construction. No issues with tube extraction were encountered.
Figure 5.10 - Installation data for Pile D1
5.3.3 Curing

The pile was left to cure in-situ for a total of 13 days prior to testing. Given the development of significant residual loads after casting at Pontarddulais, the variation in strain and temperature within the test pile was continuously monitored during the curing period using a datalogger which was connected to the strain gauges immediately after pile cap construction. The logger was programmed to record strain and temperature at 15 minute intervals for the first 48 hours after casting, reducing to hourly intervals for the remainder of the curing period. The variation in strain and temperature with curing time are presented in Figure 5.11(a) and 5.11(b) respectively; the following was noteworthy:

- The initial set period after connection of the datalogger ≈ 30 minutes after casting was characterised by increases in compressive total strain and hydration temperatures throughout the pile. Peak temperatures were greatest (≈ 36 °C) near the pile head at 1.3 m bgl and reduced with depth, presumably due to variations in soil temperature.
- Compressive strains subsequently reduced at all gauge levels after hydration temperatures had peaked and began to stabilise during the curing phase, with the largest reduction occurring near the head of the pile (stabilised tensile strain of ≈ 60 με).
- With the exception of the tensile strain near the head (which remained steady at ≈ 60 με), compressive strains began to develop in the pile once again after 80 hours and continued to slowly increase for the remainder of the measurement period.
- Minor increases in compressive strain occurred at all levels immediately prior to commencing the load test due to the placement of the loading frame and associated equipment.

The compressive strains which developed at ≈ 80 hours were attributed to the formation of residual loads within the test pile due to consolidation settlement of the alluvial layer between 3.4 m and 7.2 m bgl. The magnitude of residual load was calculated from these strains using Method 3 (Siegel and McGillivray, 2009) described in Section 3.5, assuming the onset of residual load development began approximately 80 hours after casting (i.e. the point at which the strain behaviour within the pile began to deviate from the uppermost gauge level at 1.3 m bgl). The resulting distribution of residual load with depth prior to conducting the load test is shown in Figure 5.12, where reasonable agreement is achieved with the predicted profile using the expression $\tau_{s, \text{res}} = 0.3\sigma'_{v0}$ proposed by Meyerhof (1976).
Figure 5.11 - Variation in (a) total strain and (b) temperature with time after casting - Pile D1

Figure 5.12 - Residual load distribution - Pile D1
5.3.4 Load testing

The test pile was subjected to a maintained compression load test on September 22\textsuperscript{nd} 2011 in accordance with the ICE Specification (see Appendix C for details of the loading schedule). However, unlike the previous load test at Pontarddulais described in Section 5.2.4, the loading cycles at 100\% and 150\% DVL were not performed due to the difficulties associated with strain interpretation and derivation of pile stiffness during reloading. An unplanned unload/reload cycle was necessary about midway through the load test however, as the original hydraulic jack had insufficient capacity to complete the pile test and had to be replaced. The total duration of the load test was \( \approx 68 \) hours. The measured load-displacement response at the pile head is shown in Figure 5.13. The following behaviour was noted:

- The head displacement was \( \approx 3.65 \) mm at the DVL of 735 kN.
- Creep-related displacements were increasingly apparent when the applied load exceeded 1600 kN.
- The load-displacement response during the initial stage of reloading was considerably stiffer in comparison to that observed during virgin loading.
- The pile reached a maximum head displacement of \( \approx 41 \) mm at an applied load of 2566 kN, resulting in a total capacity \( Q_{l,0.1D_0} \) of 2493 kN.

Figure 5.13 - Load-displacement response of Pile D1
5.3.5 Strain analysis

Figure 5.14 illustrates the variation in average uncorrected strain at each gauge level with time during the load test. Key observations include:

- Anomalies in strain behaviour were evident at 3.55 m bgl, where the lowest strains of all levels were mobilised, particularly during the latter stages of the test.
- Significant creep strains developed at all levels during each load hold, accounting for \( \approx 200 \, \mu \varepsilon \) of the total strain response per level at the maximum applied load of 2566 kN.
- Residual compressive strains were mobilised at all levels when the pile was unloaded to 0 kN.

The reduced strains mobilised at 3.55 m bgl were once again consistent with an increased diameter at that level. However, given that the pile was not extracted after loading, the true diameter of the pile at this level could not be determined and it was therefore decided to ignore the strains at 3.55 m bgl during the subsequent strain analysis described below.

![Figure 5.14 - Variation in strain with time during loading - Pile D1](image)

Figure 5.14 shows the variation in average elastic strain at the remaining gauge levels (i.e. 1.3 m, 6.5 m and 7.5 m bgl) with applied stress at the pile head during loading; the residual strains during the unloading cycle at 1835 kN are not presented for clarity purposes. It is evident that:

- The largest strains were mobilised at the uppermost gauge level and reduced with depth towards the base of the pile.
The reduction in strain between 1.3 m and 6.5 m bgl reached a quasi-constant value of \( \approx 95 \ \mu \varepsilon \) when the applied stress exceeded 8 MPa.

A soft, albeit temporary, stress-strain response was evident at all levels during the initial stage of reloading after replacement of the hydraulic jack.

The variation in the secant pile stiffness \( E_p \) with strain level at 1.3 m bgl during the two load cycles of the test is shown in Figure 5.15(b); the first load cycle was characterised by an initial \( E_p \) of \( \approx 38 \) GPa which reduced with increasing strain, but reloading resulted in a reversal in behaviour, with stiffness increasing with strain level in a similar manner to that observed during the previous pile test at Pontarddulais.

![Figure 5.15 - Variation in strain with (a) applied stress and (b) pile modulus - Pile D1](image)

### 5.3.6 Shaft resistance

The variation in local shear stress \( \tau_{s,loc} \) (assuming a uniform reduction in load between gauge levels) with local shaft displacement \( w_s \) is illustrated in Figure 5.16. The following observations were noteworthy:

- The majority of shaft resistance was provided by the sandy gravel socket, where a local shear stress \( \tau_{s,loc} \) of \( \approx 460 \) kPa was mobilised when the test was terminated at a displacement of 36 mm.

- The unreliable strain readings at 3.55 m bgl prevented separate measurements of the local shear stress in the medium dense sand (1.3 - 3.4 m bgl) and soft clay (3.4 - 6.5 m bgl) layers. However, a peak ‘average’ shear stress of \( \approx 35 \) kPa was mobilised between 1.3 m and 6.5 m bgl at a displacement of \( \approx 4 \) mm \( (w_s/D_s = 1.25 \%) \) and remained constant for the remainder of the load test.
The large value of $\tau_{s,loc}$ mobilised across the granular socket during loading was attributed to
the greater degree of radial displacement $\Delta y$ (and hence dilation-related increases in radial stress $\Delta \sigma'_{rd}$) of the gravel particles ($D_{50} > 1$ mm) within the shear band adjacent to the pile shaft; similarly large values of $\tau_{s,loc}$ were reported by Rollins et al. (2005) during tension load tests on drilled shafts in dense overconsolidated sandy gravels.

While the peak local shear stress in the upper sand layer was not measured directly (due to the strain anomalies at 3.55 m bgl mentioned previously), the reduction in load across the soft peaty clay was estimated using the expression $\tau_{sf} = q_t/35$ (where $q_t$ is the cone resistance corrected for end effects) proposed by Schneider et al. (2008) and then subtracted from the total reduction in load between 1.3 m and 6.5 m bgl. The resulting estimate of peak local shear stress $\tau_{sf}$ in the medium dense sand layer between 1.3 m and 3.4 m bgl was $\approx 65$ kPa.
5.3.7 Base resistance

The base resistance was determined by linearly extrapolating the load distribution to 7.7 m bgl, as the lowermost gauges were located at a distance of \( \approx 0.2 \) m above base level (unlike at Pontarddulais). Figure 5.17(a) shows the resulting base stress-displacement response during loading, while the variation in base stiffness with displacement is illustrated in Figure 5.17(b). Points of interest include:

- A residual base stress \( q_{b,\text{res}} \) of \( \approx 0.6 \) MPa was present prior to loading.
- The initial stages of loading were characterised by a relatively stiff base stress-displacement response, with a base resistance of \( \approx 9 \) MPa mobilised at a base displacement of 10 mm.
- Extrapolation of the base stress-displacement response to 38 mm resulted in a base capacity \( q_{b,0.1D_b} \) of \( \approx 16.2 \) MPa and a bearing capacity factor \( N_q = 294 \).
- The initial non-linear base stress-displacement response resulted in a gradual degradation in base stiffness with increasing base displacement.

Comparisons of the base resistance with the CPT \( q_c \) subjected to averaging techniques could not be determined due to the fact that the cone penetrometer met refusal during penetration in the very dense sandy gravel at 7.7 m bgl.

![Figure 5.17 - Variation in (a) base resistance and (b) stiffness with displacement - Pile D1](image-url)
5.4 INSTRUMENTED PILE TEST AT ERITH, ENGLAND

5.4.1 Introduction

This section presents the results of the instrumented DCIS pile test at Erith, Kent. The site was located approximately 1 km south of the previous pile test site at Dagenham described in Section 5.3 and therefore provided an ideal opportunity to confirm the behaviour of DCIS piles in the layered soil deposits of the Thames Estuary.

Table 5.3 provides a summary of the test pile details, while a schematic of the instrumentation and associated ground profile inferred from CPT03 ≈ 5.5 m from the pile location is shown in Figure 5.18. The pile was instrumented with vibrating wire gauges in groups of four at levels of 8.3 m, 9.55 m and 10.8 m bgl in the granular socket to assess the distribution of local shear stress. An additional level of gauges was placed at 0.5 m bgl to capture the reduction in load across the soft alluvial clay layer.

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<td>Shear reinforcement</td>
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<td>Cover (mm)</td>
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<th>Piling rig details</th>
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<td>Keller DCIS piling rig reference</td>
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<td>Drive tube length (m)</td>
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<td>Drive plate dimensions (OD, thickness $t$; mm)</td>
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<td>Strain gauge type</td>
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<td>Levels (m bgl)</td>
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<tr>
<td>Normalised distance from pile tip to the midpoint of the gauge levels $h/D_s$</td>
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</table>
5.4.2 Installation

The pile, designated Pile E3, was installed on November 9th 2012 by driving a 320 mm outer diameter steel tube, fitted with a 380 mm diameter base plate, using a Junttan HHK5ASS hydraulic hammer. The pile installation data is shown in Figure 5.19 where the following behaviour was noted:

- The installation tube plunged through the soft alluvial clay layer; similar behaviour was observed in the soft cohesive deposits at Pontarddulais and Dagenham.
- The number of blows per 250 mm steadily increased within the sandy gravel layer below 7.9 m bgl, with \( \approx 13 \) blows per 250 mm occurring at the end-of driving (EOD).
- The installation resistance closely resembled the corresponding profile of base resistance \( q_{b,0.1Dh} \) derived by the UWA-05 method using the CPT \( q_c \) profile adjacent to the pile location (CPT03). This is compatible with observations during installation of the test piles at the previous layered sites in Pontarddulais and Dagenham.

![Figure 5.18 - Schematic of Pile E3, CPT \( q_c \) profile and corresponding soil profile](image)
Figure 5.19 - Installation data for Pile E3
Unlike the previous tests at Pontarddulais and Dagenham, the pile was concreted using the pumped system on the DCIS piling rig, followed by tube extraction without difficulty. Vertical alignment of the reinforcement cage during installation in the freshly-cast concrete proved difficult and it subsequently became stuck in the granular socket below 8.2 m bgl. The cage was eventually pushed to a final depth of 11.0 m but strain readings taken afterwards revealed that one gauge at 8.3 m, three gauges at 9.55 m and all four gauges at 10.8 m bgl were damaged and provided no readings. As a consequence of this, the base resistance could not be measured and measurement of local shaft friction between the two gauge levels in the upper part of the granular socket was severely hampered as only one gauge was operating correctly at 9.55 m bgl. The soil in the vicinity of the pile head was excavated to a depth of 300 mm (as opposed to 500 mm under normal operations) in order to prevent the uppermost gauge level at 0.5 m from being cast into the cap.

5.4.3 Curing
Despite the strain gauge damage during cage installation, a datalogger was connected to the gauges to assess the variation in strain and temperature during curing, the results of which are shown in Figure 5.20. It is evident that:

- Large irregularities exist between gauge levels, with the uppermost level (0.5 m bgl) in tension, compressive strains occurring at 8.3 m bgl and negligible strains evident at 9.55 m bgl.
- Peak hydration temperatures were only $\approx 2^\circ$C greater than during casting.
- Cyclic variations in strain were apparent at 8.3 m and 9.55 m bgl which were attributed to tidal effects from the nearby River Thames.

The erratic strain behaviour during curing prevented accurate assessment of residual loads in the pile prior to load testing. Furthermore, the curing behaviour was in stark contrast with that observed at Dagenham, whereby the strains in the lower section of the pile become increasingly compressive with time due to consolidation settlement of the alluvial layer.
5.4.4 Load testing

The pile was subjected to a maintained compression load test on November 26\textsuperscript{th} 2012, the loading schedule for which is summarised in Appendix C. The pile was incrementally loaded (without loading cycles) to failure over a period of \( \approx 53 \) hours. The variation in pile head displacement with (a) applied load and (b) time is shown in Figure 5.21(a) and (b) respectively where the following behaviour was noted:

- The pile exhibited a stiff response during the initial stages of loading, with negligible levels of creep displacement evident.
• A sudden increase in pile head displacement occurred when the load was increased to 1759 kN which was suspected to be due to the formation of a crack in the concrete near the pile head.

• The maximum applied load of 1975 kN resulted in a pile head displacement of \( \approx 45 \) mm; the corresponding total pile capacity \( Q_{t,0.1Dh} \) was 1879 kN.

Figure 5.21 - Variation in pile head displacement with (a) applied load and (b) time during loading of Pile E3
5.4.5 Strain analysis

The variation in average uncorrected strain at each gauge level with time during the load test is shown in Figure 5.22 where it is evident that:

- The measured strain behaviour during the initial stages of loading was similar to that observed in the previous tests, with the largest strains mobilised at the uppermost gauge level and reducing with depth.
- Creep strains were relatively minor during load holds.
- The strains at the uppermost gauge level began to increase significantly as the load test progressed, until abnormalities in readings occurred for each gauge when the applied load was increased to 1544 kN (see Figure 5.23 for the individual strain gauge response at the uppermost gauge level). Such behaviour is synonymous with the presence of a crack within the concrete at the gauge level. The strains recovered somewhat during the latter stages of loading which was attributed to the closure of the crack at higher loads.
- The strain gauges at 8.3 m and 9.55 m bgl continued to function normally throughout the test, although relatively minor increases in strain with increasing applied load occurred at 9.55 m bgl during the latter stages of the test.

![Figure 5.22 - Variation in strain with time during loading - Pile E3](image-url)
Figure 5.23 - Variation in measured strains at 0.5 m with time during loading - Pile E3

Figure 5.24(a) shows the corresponding variation in elastic strain at each gauge level with applied stress at the pile head, where the erratic variation in strain at 0.5 m bgl is evident at applied stresses greater than 12 MPa. The variation in secant pile stiffness \( E_p \) with strain at 0.5 m bgl is illustrated in Figure 5.24(b); values of \( E_p \) reduce from \( \approx 38 \) GPa during the initial stages of loading to \( \approx 31 \) GPa at 460 \( \mu e \). Values of \( E_p \) beyond this strain level were considered unreliable due to the anomalies described previously.

Figure 5.24 - Variation in elastic strain with (a) applied stress and (b) pile stiffness - Pile E3
5.4.6 Shaft resistance

The variation in local shear stress $\tau_{s,loc}$ in the soft alluvial and upper dense sandy gravel layers with displacement during loading is shown in Figure 5.25. It is important to note that the derived values of $\tau_{s,loc}$ should be treated with caution due to the number of damaged strain gauges at 8.3 m and 9.55 m bgl, and the subsequent inability to measure residual loads during curing. With this in mind, the following behaviour was evident:

- A peak shear stress of $\approx 37$ kPa was mobilised across the soft alluvial layer between 0.5 m and 8.3 m bgl.
- Fluctuations in $\tau_{s,loc}$ were evident during loading in the dense sand layer between 8.3 m and 9.55 m bgl, temporarily mobilising a peak value of $\approx 85$ kPa at a displacement of 3.6 mm, before subsequently increasing to $\approx 105$ kPa at 5.5 mm.
- Profiles of $\tau_{s,loc}$ beyond a shaft displacement of 6 mm could not be determined due to the irregularities in strain at 0.5 m bgl.

![Figure 5.25 - Variation in local shear stress with displacement - Pile E3](image)

5.4.7 Base resistance

Given the damage to the lowermost gauge level, the base resistance during loading could not be determined and extrapolation of the local shaft resistance measured across the upper section of the socket was considered unreliable.
6.1 INTRODUCTION
This chapter presents the results of a series of tests on instrumented DCIS piles installed in uniform sand sites at Shotton and Ryton-on-Dunsmore in the United Kingdom. Both sites provided an ideal opportunity to assess the behaviour of DCIS piles in sand, in particular the variation in local shear stress with normalised distance from the pile tip, i.e. friction fatigue. The procedures employed during installation, curing and load testing were previously described in Chapter 3, while in Chapter 4, the ground profiles and soil properties relevant to pile behaviour at both sites was presented.

6.2 INSTRUMENTED PILE TEST AT SHOTTON, WALES

6.2.1 Introduction
This section presents the results of the installation, curing and maintained compression load testing of an instrumented DCIS pile at Shotton, Wales in May 2011. An overview of the test pile (including instrumentation levels and pile cap dimensions), CPT101 $q_c$ profile and corresponding ground profile at the pile location is shown in Figure 6.1, while the test pile details are summarised in Table 6.1.

The pile had a nominal shaft diameter of 320 mm and a total length of 5.75 m, and was instrumented with vibrating wire strain gauges in groups of four at levels of 0.3 m, 2.5 m, 4.0 m and 5.5 m below ground level (bgl). The uppermost gauge level was placed as close as possible to the head of the pile to enable strain readings which were not influenced by shaft resistance. The gauges at the base (5.5 m bgl) had reduced-length sister bars to enable placement as close to the base as possible. As will be shown later, the installation of 6 no. DCIS tension piles to a depth of 10 m bgl to provide reaction for the loading frame during the subsequent load test on the test pile led to significant densification of the surrounding soil, resulting in the CPT $q_c$ profile being somewhat unrepresentative of the actual soil conditions during installation of the test pile.
### Table 6.1 - Test pile details - Pile S1

<table>
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<tr>
<th>Test pile details</th>
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</thead>
<tbody>
<tr>
<td>Pile reference</td>
</tr>
<tr>
<td>Length $L$ (m)</td>
</tr>
<tr>
<td>Shaft diameter $D_s$ (mm)</td>
</tr>
<tr>
<td>Base diameter $D_b$ (mm)</td>
</tr>
<tr>
<td>Date of installation</td>
</tr>
<tr>
<td>Date of static load test</td>
</tr>
</tbody>
</table>

**Reinforcement details**

| Main reinforcement                | 4 x H40 x 5.85 m |
| Shear reinforcement               | 240 mm OD H10 helical cage at 200 mm pitch |
| Cover (mm)                        | 40               |

**Concrete details**

| Mix                                | C45/55           |
| 28-day characteristic cube         |                  |
| compressive strength $f_{cu}$ (MPa)| 55               |
| Maximum aggregate size             | 10 mm            |

**Piling rig details**

| Keller DCIS piling rig reference   | 3192             |
| Hammer                             | Junttan HHK5ASS  |
| Drive tube length (m)              | 12               |
| Drive tube dimensions (OD, ID, wall thickness $r$; mm) | 320, 280, 20 |
| Drive plate dimensions (D, thickness $r$; mm)         | 380, 11          |

**Instrumentation details**

| Strain gauge type                  | Geosense G100-040 Vibrating Wire Sister Bar |
| Levels (m bgl)                     | 0.3, 2.5, 4.0, 5.5 |
| Normalised distance from pile tip to mid-point between gauge levels $h/D_s$ | 11.4, 6.6, 2.6 |

---

![Figure 6.1 - Schematic of Pile S1, CPT $q_c$ profile and corresponding soil profile](image-url)
6.2.2 Installation

The pile was installed on May 9\textsuperscript{th} 2011 using a 320 mm outer diameter, 20 mm thick, open-ended steel installation tube with a 380 mm diameter circular steel sacrificial driving plate at the base, which was driven to a final depth of 5.75 m bgl using a Junttan HHK5ASS hydraulic hammer. Figure 6.2 summarises the driving data in terms of (a) the total number of blows during installation, (b) the number of blows per 250 mm of penetration and (c) the installation resistance derived from the piling rig instrumentation. It is evident that:

- Over 250 blows were required to drive the tube to a final depth of 5.75 m bgl.
- The majority of resistance during installation occurred in the dense sand at depths greater than \( \approx 4 \) m bgl.
- The installation resistance was consistently greater than the profile of base resistance \( q_{b,0.1Db} \) predicted by the UWA-05 design method using the \( q_c \) profile at the pile location (CPT01).

The poor agreement between the installation resistance and base resistance profile was attributed to the distortion of the surrounding sand during installation of the six 10 m DCIS tension piles for the subsequent load test. As shown in Figure 6.2(c), an improved relationship is evident with the installation resistance of AP1 which was the first anchor pile to be driven.

High-slump concrete with a characteristic 28 day cube strength \( f_{cu} \) of 55 MPa was skipped into the top of the tube after driving, after which the tube was then extracted (with the driving plate remaining at the base) and the instrumented reinforcement cage was slowly inserted into the freshly-cast concrete. The soil surrounding the pile head was excavated to a depth of 500 mm below the surface to accommodate the 700 mm wide concrete pile cap; this resulted in the uppermost strain gauge level being inadvertently cast inside the cap which led to anomalies in strain behaviour during loading (see Section 6.2.3). An additional set of strain readings were taken after casting to confirm that the gauges were working correctly, after which the gauges were connected to a datalogger in order to monitor strain and temperature behaviour during curing.
Figure 6.2 - Installation data for Pile S1
6.2.3 Curing

The pile was left to cure for a period of 15 days in order to allow the concrete to gain sufficient strength prior to load testing. In a similar manner to the previous test piles installed in layered soil at Dagenham and Erith in Chapter 5, the development of residual loads during curing was investigated by continuously monitoring strain and temperature within the test pile using a datalogger. The variation in strain and temperature with time after casting is shown in Figure 6.3(a) and 6.3(b) respectively, where the following behaviour is noteworthy:

- Compressive strains developed throughout the pile during initial set, with peak hydration temperatures occurring \( \approx 14 \) hours after casting. However, peak compressive strains and temperatures were considerably greater at the uppermost gauge level (0.3 m bgl) in comparison to the rest of the pile, presumably due to the larger cross-sectional area of the pile cap in which the gauges were located.
- A reversal in strain behaviour occurred during the curing phase, with tensile strains developing at all levels as hydration temperatures receded.
- Pile temperatures reached equilibrium approximately 150 hours after casting, with relatively minor increases in tensile strain occurring at or below 2.5 m bgl for the remainder of the curing period.
- Cyclic variations in strain and temperature at 0.3 m bgl were attributed to contraction and expansion of the pile cap due to the diurnal variation in air temperature.

Given that the strain behaviour during curing showed no sign of compressive strains developing within the lower section of the pile over time and consolidation-related settlement was considered unlikely due to the stratigraphy of the site (i.e. uniform sand), residual loads were assumed to be non-existent in the pile upon commencement of the pile load test.
Figure 6.3 - Variation in (a) strain and (b) temperature with time after casting - Pile S1
6.2.4 Load testing

A maintained compression load test was conducted on the pile in accordance with the Institution of Civil Engineers Specification for Piling and Embedded Retaining Walls (ICE, 2007) on May 24\textsuperscript{th} 2011. The loading schedule, summarised in Appendix C, specified two compressive load cycles in increments of 250 kN to maximum loads of 1000 kN and 1500 kN respectively. The pile was subsequently reloaded to 1500 kN, followed by 100 kN increments up to a maximum load of 2400 kN. The measured load-displacement response at the pile head is shown in Figure 6.4, where the following was evident:

- The pile head displacement at the design verification load (DVL) of 1000 kN was $\approx 7.6$ mm, which was within the maximum permitted settlement of 10 mm.
- A stiff load-displacement was apparent during reloading, with a displacement of $\approx 4.3$ mm necessary to remobilise the DVL of 1000 kN, for example.
- Creep displacements were increasingly evident during the latter stages of the test at applied loads of 2000 kN or greater.
- The pile head displacement at the maximum pile load of 2400 kN was $\approx 53$ mm, resulting in a total pile capacity $Q_{0.1D_b}$ of 2214 kN.

![Figure 6.4 - Load-displacement response - Pile S1](image-url)
6.2.5 Strain analysis

The variation in average strain at each gauge level with time during the load test is illustrated in Figure 6.5, where the following behaviour was apparent:

- Significantly lower strains were mobilised at 0.3 m bgl compared to the rest of the pile, primarily due to the larger cross-section of the pile cap.
- Creep-related increases in strain were evident at all levels during load holds, accounting for ≈11% of the measured total strain at 2.5 m bgl at the maximum applied load of 2400 kN.
- Unloading resulted in the mobilisation of residual compressive strains throughout the pile, the magnitude of which increased with each successive unloading cycle.
- With the exception of the strains at 0.3 m bgl, a consistent reduction in strain with depth was evident between 2.5 m and 5.5 m bgl during all stages of the load test.

Given that the strains mobilised in the pile cap at 0.3 m bgl were significantly lower in comparison to the rest of the pile, the true pile stiffness could not be determined at this level. The stress-strain relationship at 2.5 m bgl was therefore used instead, with the stress at this level estimated by calculating the reduction in load due to shaft resistance between 0.3 m and 2.5 m bgl using correlations with CPT $q_c$ by Bustamante and Gianeselli (1982).

![Figure 6.5 - Variation in average strain with time during loading - Pile S1](image-url)
The creep strains which developed during each load hold were removed from the total strain response using the procedure described in Section 3.7.3. The resulting variation in average elastic strain at each level with the estimated stress at 2.5 m bgl is shown in Figure 6.6(a). It is evident that:

- The stress-strain response was similar at all levels.
- The magnitude of reduction in strain between 2.5 m and 5.5 m was relatively constant when the stress was greater than 12 MPa.
- The load cycles led to minor fluctuations in stress-strain response.

Figure 6.6(b) shows the corresponding variation in secant pile stiffness $E_p$ with strain level derived using the stress-strain response at 2.5 m bgl. The initial stages of loading were characterised by a sharp reduction in pile stiffness with increasing strain, although a relatively constant stiffness of $\approx 41$ GPa was evident during the second load cycle. In line with stiffness behaviour during the previous instrumented tests described in Chapter 5, the latter stages of loading led to increases in pile stiffness with increasing strain level.

![Figure 6.6 - Variation in elastic strain with (a) estimated stress at 2.5 m bgl and (b) pile stiffness - Pile S1](image-url)
6.2.6 Shaft resistance

Figure 6.7 shows the average shaft friction $\tau_{s,\text{avg}}$ in the sand layer below 2.5 m bgl with shaft displacement during loading. Points of interest include:

- A soft ductile response was evident during loading, with the average shear stress reaching a peak value of $\approx 125$ kPa at a displacement of $\approx 40$ mm, corresponding to a normalised displacement $w/D_s$ of 12.5%.
- The load cycles led to reductions in $\tau_{s,\text{avg}}$ at a given load during reloading.

![Graph showing variation in average shear stress below 2.5 m bgl with displacement during loading](image)

**Figure 6.7 - Variation in average shear stress below 2.5 m bgl with displacement during loading - Pile S1**

The mobilised local shear stresses in the sand layers below 2.5 m bgl are plotted against the local shaft displacement in Figure 6.8, where it is apparent that:

- A relatively stiff response in shaft resistance was evident during the initial stages of loading, with a peak shear stress $\tau_{sf}$ of $\approx 105$ kPa being mobilised between 4.0 m and 5.5 m bgl at a shaft displacement of $\approx 10$ mm ($w_s/D_s = 3.2\%$).
- Load cycling led to reductions in $\tau_{s,\text{loc}}$ during reloading, i.e. friction fatigue.
- The local shear stress between 4.0 m and 5.5 m bgl began to reduce slightly in the latter stages of loading, while values of $\tau_{s,\text{loc}}$ between 2.5 m and 4.0 m bgl continued to increase.
6.2.7 Base resistance

The base resistance was derived by linearly extrapolating the load distribution to 5.75 m bgl (as the lowermost gauge level was only $\approx 0.2$ m from the base of the pile). The resulting base stress-displacement response during loading is shown in Figure 6.9(a), while Figure 6.9(b) illustrates the variation in base stiffness with displacement. Observations of base behaviour include:

- A stiff linear base stress-displacement was evident during the initial stages of loading, resulting in a relatively constant base stiffness $E_{b,eq}$ of $\approx 290$ MPa which was $\approx 13\%$ less than the estimated in-situ small-strain stiffness $E_0$ derived using the correlation with CPT $q_c$ by Mayne (2006).
- Residual base stresses during unloading increased with each successive load cycle.
- The degradation in base stiffness was also affected by load cycling, eventually commencing when the base displacement exceeded $\approx 9$ mm (i.e. $w/b/D_b = 2.4\%$).
- The base resistance at failure $q_{b,0.1D_b}$ was $\approx 15.8$ MPa, resulting in a bearing capacity factor $N_q$ of $\approx 213$.

The above observations confirm that the pile behaved in a predominantly end-bearing manner during loading, with the base resistance accounting for $\approx 80\%$ of the total pile resistance at a head displacement of 38 mm (i.e. $0.1D_b$). Normalising $q_{b,0.1D_b}$ by the corresponding average cone resistance at the base $q_{c,avg}$ using the Dutch averaging method resulted in $q_{b,0.1D_b}/q_{c,avg} \approx 1.08$ which is significantly greater than the $q_{b,0.1D_b}/q_{c,avg} = 0.6$ recommended for driven piles by the UWA-05 method (Lehane et al., 2005c). As mentioned previously in Section 6.2.1, the

![Figure 6.8 - Variation in local shear stress with displacement during loading - Pile S1](image-url)
installation of the anchor piles prior to the test pile most likely resulted in significant increases in soil density, making the profile of $q_c$ in the vicinity of the base of the pile (carried out several weeks before the test) unrepresentative of the true conditions during loading.

Figure 6.9 - Variation in (a) base resistance and (b) stiffness with displacement during loading - Pile S1
6.3 INSTRUMENTED PILE TESTS AT RYTON-ON-DUNSMORE

6.3.1 Introduction

This section presents the results of installation, curing and maintained compression load testing of three instrumented DCIS piles in medium dense sand at Ryton-on-Dunsmore, near Coventry, United Kingdom. These tests were primarily performed to assess the shaft behaviour of DCIS piles in sand, in particular the development of friction fatigue during installation and its subsequent influence on the local shear stresses during loading. Details of each test pile are summarised in Table 6.2, while Figure 6.10 shows a schematic of the piles, the average CPT $q_c$ profile and the corresponding ground profile at the test location. Each pile was instrumented with a total of 16 vibrating-wire embedment strain gauges, with four gauges placed at four separate levels (see Table 6.2 for further details). The levels were chosen to optimise measurements of local shear stress over a wide range of $h/D_s$ values.

Table 6.2 - Test pile details - Piles R1, R2 and R3

<table>
<thead>
<tr>
<th>Test pile details</th>
<th>Pile R1</th>
<th>Pile R2</th>
<th>Pile R3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pile reference</td>
<td>6.00</td>
<td>7.00</td>
<td>5.50</td>
</tr>
<tr>
<td>Length $L$ (m)</td>
<td>320</td>
<td>320</td>
<td>320</td>
</tr>
<tr>
<td>Shaft diameter $D_s$ (mm)</td>
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<td>380</td>
<td>380</td>
</tr>
<tr>
<td>Date of installation</td>
<td>10/10/13</td>
<td>11/10/13</td>
<td>11/10/13</td>
</tr>
<tr>
<td>Date of static load test</td>
<td>30/10/13</td>
<td>01/11/13</td>
<td>04/11/13</td>
</tr>
</tbody>
</table>

Reinforcement details

- Main reinforcement: 4 x B25 x 6.35 m 4 x B25 x 7.35 m 4 x B25 x 5.85 m
- Shear reinforcement: B8 x 240 mm OD helical link at 200 mm pitch B8 x 200 mm OD helical link at 200 mm pitch B8 x 200 mm OD helical link at 200 mm pitch

Cover (mm)

- 40
- 60
- 60

Concrete details

- Mix: C35/45
- 28-day characteristic cube compressive strength $f_{cu}$ (MPa): 45
- Maximum aggregate size: 10 mm

Piling rig details

- Keller DCIS piling rig reference: 3187
- Hammer: Junttan HHK5ASS
- Drive tube length (m): 16
- Drive tube dimensions: (OD, ID, wall thickness $t$; mm) 320, 280, 20
- Drive plate dimensions: (D, thickness $t$; mm) 380, 11

Instrumentation details

- Strain gauge type: Geosense VWS-2100 vibrating wire embedment strain gauges
- Levels (m bgl): 2.0, 3.0, 4.5, 5.85 2.0, 4.0, 5.25, 6.85 2.0, 3.5, 4.5, 5.35
- Normalised distance from pile tip $h/D_s$: 11.0, 7.0, 2.6 12.5, 7.4, 3.0 8.6, 4.7, 1.8
Figure 6.10 - Schematic of test piles R1, R2 and R3, average CPT $q_c$ profile and corresponding soil profile
6.3.2 Installation
Pile R1 was installed on October 10th 2013 followed by Piles R2 and R3 the following day. Each pile was constructed using a 320 mm outer diameter installation tube with a 380 mm diameter base plate which was driven using a Junttan HHK5ASS hydraulic hammer. No driveability issues were encountered during installation of the test piles. The installation data for the three pile in terms of (a) total blow count, (b) the number of blows per 250 mm of penetration and (c) installation resistance with depth is illustrated in Figure 6.11, where it apparent that:

- Despite differences in pile length, the total number of blows required to install Piles R1 and R2 were relatively similar (≈ 400 blows); this observation was attributed to the greater installation resistance of Pile R1 in the made ground and clay layers between the surface and 1.8 m bgl.
- The number of blows per 250 mm penetration was relatively comparable for all piles in the medium dense to dense sand layer below 1.8 m.
- In a similar manner to the previous tests at the other sites, the installation resistance of the three piles closely resembled the base resistance \( q_{b,0.1Db} \) derived from the average CPT \( q_c \) profile using the UWA-05 method.

The fact that the installation profiles were similar to the UWA-05 \( q_{b,0.1Db} \) profile suggests that the installation of the anchor piles prior to the test piles did not lead to significant changes in soil state (unlike at Shotton).

6.3.3 Curing
The test piles were allowed to cure for a period of between 19 and 23 days in order to allow the concrete to gain sufficient strength prior to conducting the maintained compression load tests. Continuous monitoring of strain and temperature behaviour within the test piles during curing was deemed unnecessary based on the experience of similar sand conditions at Shotton. However, strain readings after casting and immediately prior to commencing each load test allowed the net change in strain with the test piles to be determined. The resulting distributions of net change in strain with depth after curing for the test piles is shown in Figure 6.12, where relatively uniform tensile strain profiles of ≈ 65 με were evident for all three piles. Such observations are in keeping with the strain profiles measured after curing of Pile S1 at Shotton and suggest that residual loads were not present in the test piles prior to loading.
Figure 6.11 - Installation data for Ryton-on-Dunsmore test piles
6.3.4 Load testing

The test piles were subjected to maintained compression load tests in a consecutive manner, beginning with Pile R1 on October 30th 2013, followed by Pile R2 on November 1st and Pile R3 on November 4th. The loading schedules, summarised in Appendix C, were typically performed using load increments equivalent to 10% of the predicted pile capacity. The magnitude of each increment was subsequently reduced when the pile appeared to be approaching failure. The duration of the tests varied from ≈ 24 hours for Pile R2 to ≈ 60 hours for Pile R3. Figure 6.13 shows the measured load-displacement response of the three test piles, where the following behaviour was noted:

- A near-identical load-displacement response was evident for all piles during the initial stages of loading.
- Significant creep displacements developed when applied loads exceeded 1400 kN during loading of Piles R1 and R2, but were considerably less for Pile R3, despite the fact that Pile R2 and R3 were constructed using the same concrete mix.
- The total pile capacities \( Q_{t,0.1Db} \) for Piles R1 and R2 were 1622 kN and 1578 kN respectively.
- Pile R3 continued to exhibit a stiff response in comparison to the other piles at applied loads greater than 1400 kN, with the test eventually being terminated when the maximum safe working load of the loading frame was reached. Hyperbolic
extrapolation of the load-displacement curve to 38 mm (i.e. $0.1D_h$) using the method by Chin (1972) resulted in an estimated total pile capacity of 2276 kN.

Given the similarity in ground conditions and installation data at the test location, the significantly greater capacity of Pile R3 in comparison to Piles R1 and R2 was somewhat unexpected and was subsequently investigated further during analysis of shaft and base resistance in Sections 6.3.6 and 6.3.7 respectively.

![Figure 6.13 - Measured load-displacement response of Piles R1, R2 and R3](image)

### 6.3.5 Strain analysis

The variation in total strain with time at each gauge level for the three test piles is shown in Figure 6.14. The following observations were noteworthy:

- Significant creep strains developed within the piles during each load hold, with the magnitude of such strains tending to be greatest in the lower sections of each pile.
- Anomalies in strain behaviour at the uppermost gauge levels (i.e. 2.0 m bgl) in Piles R2 and R3 resulted in strains becoming relatively constant on either a temporary (Pile R3) or permanent (Pile R2) basis, despite significant increases in load. Localised cracking of the concrete in the vicinity of the gauges was suspected due to excessive drying shrinkage in the unsaturated sand. This was confirmed in laboratory-scale curing tests on instrumented concrete specimens by O’Ceallaigh and Burke (2012).
- The rate of increase in strain at 3.0 m and 4.5 m bgl during the latter stages of loading of Pile R1 tended to be lower in comparison to those at 2.0 m and 5.85 m bgl.
- The unloading phase at the end of the tests resulted in residual compressive strains being mobilised throughout each test pile.

![Graph](attachment:image.png)

**Figure 6.14 - Variation in strain with time for Piles (a) R1 (b) R2 and (c) R3**
In a similar manner to the previous load tests, the mobilised total strains in Figure 6.14 were corrected for the effects of creep within the concrete using the procedure described in Chapter 3. The resulting variation in elastic strain with applied stress at each gauge level for the three test piles is shown in Figure 6.15. Key observations include:

- With the exception of the erratic strain behaviour at the uppermost gauge levels (i.e. 2.0 m bgl) of Piles R2 and R3, a systematic reduction in elastic strain (and hence load) with depth was evident for each pile. This observation suggests that negligible variations in pile cross-section had occurred after casting (unlike the test piles in layered soils described previously in Chapter 5).
- The stress-strain responses at 3.0 m and 4.5 m bgl of Pile R1 began to deviate from the remaining levels (i.e. 2.0 m and 5.85 m bgl) during the latter stages of loading which was attributed to pile bending in the middle sections of the pile, resulting in the average strain at these gauge levels being somewhat biased.
- The reduction in strain between 2.0 m and 4.0 m bgl during loading of Pile R2 reached a quasi-constant value of \( \approx 55 \, \mu \varepsilon \) prior to the irregularities in strain occurring in the uppermost gauge level (2.0 m bgl).
- The mobilised strains at 2.0 m bgl in Pile R3 became temporarily constant during the initial stages of loading, before recovering somewhat when the applied stress exceeded 15 MPa.

The variation in stiffness of Piles R2 and R3 with strain level during loading was difficult to ascertain given the abnormalities in strain behaviour at the uppermost gauge level (2.0 m bgl) in both piles. However, a reasonable estimate of the strain response for Pile R2 was achieved by assuming the constant difference in strain observed between 2.0 m and 4.0 m bgl prevailed for the remainder of the test; the resulting predicted strain profile at 2.0 m bgl is shown in dashed lines in Figure 6.15(b). This method of correction could not be used to derive the strain response of Pile R3 however, as the reduction in strain between 2.0 m and 3.5 m bgl had not reached an ultimate value before anomalies in strain behaviour began to occur. A peak reduction in strain of \( \approx 40 \, \mu \varepsilon \) was subsequently obtained by plotting the change in strain between the two gauge levels against pile head displacement and hyperbolically extrapolating the relationship to an ultimate value.
Figure 6.15 - Variation in strain with applied stress for Piles (a) R1 (b) R2 and (c) R3
Prior to calculating the pile stiffness $E_p$, the reduction in load between the surface and 2.0 m bgl was accounted for using the method described in Section 3.7.5 whereby the peak shear stress was derived using relationships with the CPT $q_c$ profile by Bustamante and Gianeselli (1982) and subsequently incorporated into a theoretical load transfer curve using the method by Niazi and Mayne (2010) in order to estimate the variation in shaft resistance with displacement. The resulting stiffness response of each pile, derived using the stress-strain relationship at the uppermost gauge level, with strain level is shown in Figure 6.16; relatively constant values of $E_p$ of $\approx 34$ GPa and $\approx 40$ GPa were evident for Piles R1 and R2 respectively during the latter stages of loading, but Pile R3 exhibited strong increases in stiffness ($E_p > 45$ GPa) when the elastic strain exceeded $300 \, \mu e$. Polynomial equations fitted to the stiffness response of each pile in Figure 6.16 enabled the strains in the remaining gauge levels to be converted to loads.

![Figure 6.16 - Variation in pile modulus with strain level for Piles R1, R2 and R3](image)

### 6.3.6 Shaft resistance

Figure 6.17(a) shows the variation in average shear stress $\tau_{s,\text{avg}}$ mobilised on the shaft of each pile in the sand layer below 2.0 m bgl. A relatively soft shaft resistance response was evident for the piles, with a displacement of $\approx 15$ mm required to mobilise a peak average shear stress of $\approx 128$ kPa along the shaft of Pile R2. In contrast, the average shear stresses for Pile R1 and R3 did not reach peak values by the time the test was terminated. Normalising the peak average shear stresses by the corresponding average cone resistance along the shaft of each pile resulted in values of $\tau_{s,\text{avg}}/q_c$ of $\approx 0.009 - 0.013$ (see Figure 6.17b) which, with the exception of Pile R2, were greater than the typical range for closed-ended steel displacement piles reported by Gavin and Lehane (2003).
The local shear stress $\tau_{s,loc}$ was derived based on the assumption of a linear reduction in load between each gauge level. Figure 6.18 shows the mobilised local shear stresses during loading of each pile; while local shaft displacements of $\approx 15$ mm were typically required to mobilise peak shear stresses at the majority of sections of the test piles, the local shear stress between 3.5 m and 4.5 m bgl during loading of Pile R3 appeared to reach a peak value of $\approx 115$ kPa at displacement of $\approx 10$ mm, before increasing again and failing to reach an ultimate value when the test was terminated. Post-peak softening of $\tau_{s,loc}$ was evident during loading of Pile R2, although this behaviour is most likely due to the influence of pile bending on strain readings during the latter stages of the test, as discussed previously.

The variation in peak local shear stress $\tau_{sf}$ with depth for the three piles is shown in Figure 6.19(a) where it is apparent that:

- The profiles of peak local shear stress on the shafts of the three piles resembled the corresponding average $q_c$ profile in Figure 6.19(b), with lower values of $\tau_{sf}$ mobilised across the medium dense layer between 4.0 m and 5.0 m bgl in comparison to the denser layers.
- Peak shear stresses measured between 2.0 m and 3.5 m bgl were between 18 % and 50 % less than values mobilised near the base of each pile, despite the density of the soil in the upper layer being considerably greater ($q_c > 15$ MPa).
Figure 6.18 - Variation in local shear stress with shaft displacement during loading for Piles (a) R1 (b) R2 and (c) R3
The Coulomb friction equation \( \tau_{sf} = \sigma'_{rf} \tan \delta_{cv} \) (see Section 2.3.4 for more details) was used to back-calculate the local radial effective stresses at failure \( \sigma'_{rf} \) from the measured peak local shear stresses using a constant-volume interface friction angle \( \delta_{cv} = \phi'_{cv} = 35^\circ \) based on the results of direct shear tests on sand samples taken from 3.5 m bgl at the test site. Values of \( \sigma'_{rf} \) were subsequently normalised by the corresponding average cone resistance \( q_{c,avg} \) between gauge levels and plotted against normalised distance from the pile base \( h/D_s \) in Figure 6.20(a). While some scatter exists, a trend of \( \sigma'_{rf}/q_c \) reducing with increasing \( h/D_s \) is evident, suggesting that the test piles were affected by friction fatigue. Figure 6.20(a) also shows the profile of normalised radial stress after installation \( \sigma'_{rc}/q_c \) with \( h/D_s \) predicted by the UWA-05 method which is in close agreement with the measured data from the three test piles. This observation further validates the theory that friction fatigue is a characteristic of DCIS piles, but also suggests that increases in radial stress due to interface dilation during loading represents a small proportion of the overall radial stress at failure \( \sigma'_{rf} \).

![Figure 6.19 - Profiles of (a) peak local shear stress for Piles R1, R2 and R3 and (b) average CPT q_c](image)

The review of studies of friction fatigue during installation of preformed displacement piles in Section 2.3.2 demonstrated the strong relationship between the magnitude of degradation in radial stress and the number of load cycles \( N \) during installation; this was also investigated for the three test piles in Figure 6.20(b) where, despite scatter in the data, values of \( \sigma'_{rf}/q_c \) appear to reduce with increasing \( N \).
The data presented in Figure 6.20(a) and (b) suggests that friction fatigue is a key mechanism governing the shaft behaviour of DCIS piles in sand and is therefore investigated in greater detail in Chapter 7 using the results of previous load tests in the uniform sand at Shotton and the layered deposits described in Chapter 5.

6.3.7 Base resistance

The base stress-displacement response of the three test piles during loading is shown in Figure 6.21. It is evident that:

- The piles responded in a stiff linear manner during the initial stages of loading, with the base-displacement curves for Pile R1 and R2 essentially identical.
- The base stiffness $E_{b,eq}$ of Pile R3 during loading was considerably greater than the other two piles.
- Values of $E_{b,eq}$ for Piles R1 and R2 during initial loading were $\approx 45\%$ less than the in-situ small-strain stiffness $E_0$ of $\approx 240$ MPa measured during the seismic CPT; this value of $E_0$ was essentially identical to the predicted values using the CPT $q_c$ correlation by Mayne (2006).
- Degradation in base stiffness degradation began at a gradually rate for all piles during loading. Comparable rates of reduction in $E_{b,eq}$ were also evident.
- Pile R3 mobilised a greater base resistance for a given displacement in comparison to Piles R1 and R2 during the latter stages of loading.
Figure 6.21 - Variation in (a) base resistance and (b) stiffness with displacement during loading for Piles R1, R2 and R3

Table 6.3 provides a summary of the base resistance at failure $q_{b,0.1Db}$, normalised base resistance $q_{b,0.1Db}/q_{c,avg}$ and bearing capacity factor $N_q$ for the three test piles. Values of $q_{b,0.1Db}/q_{c,avg}$ for Pile R1 and R2 were in reasonable agreement with $q_{b,0.1Db}/q_{c,avg} = 0.6$ recommended for driven piles in sand by the UWA-05 method. However, Pile R3 had a normalised base resistance of $\approx 0.93$, implying that the average cone resistance in the vicinity of the pile base was considerably greater than that measured by the nearest CPT approximately 3 m away (despite installation data being similar to the other piles). It was therefore concluded that the greater total capacity of Pile R3 in comparison to Piles R1 and R2 was a consequence of the significantly larger base resistance of the pile during loading.

<table>
<thead>
<tr>
<th>Pile</th>
<th>Base resistance at failure $q_{b,0.1Db}$ (MPa)</th>
<th>Normalised base resistance at failure $q_{b,0.1Db}/q_{c,avg}$</th>
<th>Bearing capacity factor $N_q$</th>
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</thead>
<tbody>
<tr>
<td>R1</td>
<td>6.26</td>
<td>0.54</td>
<td>55</td>
</tr>
<tr>
<td>R2</td>
<td>6.02</td>
<td>0.52</td>
<td>45</td>
</tr>
<tr>
<td>R3</td>
<td>10.84$^1$</td>
<td>0.93</td>
<td>103</td>
</tr>
</tbody>
</table>

$^1$Extrapolated to 0.1Db using hyperbolic method by Chin (1972)
CHAPTER 7  ANALYSIS AND DISCUSSION
7.1 INTRODUCTION
This chapter provides a discussion of the overall behaviour of DCIS piles in sand based on the results of the instrumented tests presented in Chapters 5 and 6, with reference to other relevant data from the literature. Firstly, the pile installation results are analysed, including comparisons with the base resistance profiles predicted by three CPT $q_c$ averaging methods. This is then followed by an assessment of the curing temperatures and strains within the test piles at Dagenham and Shotton, based on the review of residual loads in cast-in-situ piles presented in Chapter 3. The behaviour of the instrumented DCIS test piles during maintained compression loading is then discussed, including aspects of shaft behaviour such as friction fatigue and correlations with cone resistance, and the variation in base resistance and stiffness during loading. The performance of seven CPT-based displacement pile design methods for estimating total, shaft and base capacity of DCIS piles in sand is subsequently examined. Finally, the load-displacement behaviour of DCIS piles is investigated using a database of over 100 proof load tests in sand.

7.2 INSTALLATION

7.2.1 Installation resistance
As discussed in Section 2.4.2, close agreement between the base resistance $q_b$ and the corresponding CPT $q_c$ profile was observed by Lehane (1992) and Chow (1997) during jacked installation of the model ICP closed-ended steel pile in the uniform sand deposits at Labenne and Dunkirk respectively. Layered deposits on the other hand tend to highlight significant scale effects due to the differences in diameter between a displacement pile and a cone penetrometer, and a suitable averaging method must therefore be applied to the CPT $q_c$ profile to account for such effects prior to assessing potential relationships with the base resistance $q_{b,0.1D_b}$. Xu and Lehane (2005) demonstrated the superiority of the Dutch $q_c$ averaging method in accounting for scale effects during jacked installation of a 350 mm wide precast pile during jacked installation in a layered deposit in comparison to the arithmetic and geometric methods (see Section 2.4.5 for more details). However, Lehane et al. (2005c) showed that jacked piles tend to mobilise higher $q_{b,0.1D_b}/q_{c,avg}$ values in comparison to driven piles due to the large jacking strokes and higher residual stresses. As a result, displacement pile design methods typically apply reduction factors to the average cone resistance $q_{c,avg}$ to account for such partial mobilisation effects on the base resistance. These factors, summarised in Table 7.1, are subsequently used to examine the relationship between the average cone resistance and the installation resistance of the instrumented DCIS piles.
Table 7.1 - Summary of base resistance formulations for closed-ended displacement piles

<table>
<thead>
<tr>
<th>Method</th>
<th>Base resistance at failure (MPa)</th>
<th>Averaging technique</th>
</tr>
</thead>
<tbody>
<tr>
<td>ICP-05 (Jardine et al., 2005)</td>
<td>[ q_{b,0.1D_b} = 1 - 0.5 \log \left( \frac{D_b}{D_{CPT}} \right) q_{c,avg} ]</td>
<td>Arithmetic</td>
</tr>
<tr>
<td>EF-97 (Eslami and Fellenius, 1997)</td>
<td>[ q_{b,0.1D_b} = q_{c,avg} ]</td>
<td>Geometric</td>
</tr>
<tr>
<td>UWA-05 (Lehane et al., 2005c)</td>
<td>[ q_{b,0.1D_b} = 0.6q_{c,avg} ]</td>
<td>Dutch</td>
</tr>
</tbody>
</table>

The base resistances of the instrumented DCIS piles in Chapters 5 and 6 were not measured during installation due to the impracticalities associated with instrumenting the detachable sacrificial driving plate with a load cell. The installation resistance \( q_{cp} \), derived using the rig instrumentation (described in Section 3.3 and Appendix A), was therefore used as an alternative measure of base resistance. The profiles of base resistance \( q_{b,0.1D_b} \) estimated by the ICP-05, EF-97 and UWA-05 methods, using the \( q_c \) averaging methods and reduction factors specified in Table 7.1, are compared with two example installation resistance profiles of Anchor Pile AP1 in uniform sand at Shotton in Figure 7.1(a) and Pile P1 in the highly-layered stratigraphy at Pontarddulais in Figure 7.1(b). It is apparent that:

- The EF-97 method, derived using the geometric average, significantly over-predicts the installation resistance at both sites.
- Profiles of \( q_{b,0.1D_b} \) derived using the ICP-05 and UWA-05 method show close agreement with the \( q_{cp} \) profile of AP1 in the uniform sand.
- The installation resistance of Pile P1 during driving in the sand layer between 4.7 m and 6.1 m bgl at Pontarddulais was significantly over-estimated by the ICP-05 method (using the arithmetic average cone resistance), highlighting the inability of the arithmetic averaging method to account for scale effects in layered soils.
- The base resistance profile predicted by the UWA-05 using the Dutch averaging technique yields the closest match to the measured profiles of installation resistance at both sites. Similar results were also observed during installation of the instrumented DCIS test piles at Dagenham, Erith and Ryton-on-Dunsmore, as discussed in Chapters 5 and 6.
Despite the approximate nature of the Danish pile driving formula, the similarities in the profiles of installation resistance and predicted base resistance are encouraging. Furthermore, dynamic CAPWAP tests conducted on the steel tube during driving confirmed that the majority of the installation resistance was generated by the base of the tube rather than through shaft resistance (details of the CAPWAP dynamic test results are presented in Appendix B). The use of the pile driving formula for pile lengths of up to 10 m therefore seems justified (greater embedded lengths will inevitably result in the shaft resistance contributing a greater portion of the total resistance to driving).

The installation resistance is compared with the base resistances derived using the ICP-05, EF-97 and UWA-05 methods at 0.25 m intervals during driving at depths greater than 2.0 m bgl in cohesionless layers for Pontarddulais (Pile P1), Dagenham (Piles D1, S117, S121, S397), Erith (Anchor Piles AP3, AP8 and AP16), Shotton (Anchor pile AP1) and Ryton-on-Dunsmore (Piles R1, R2 and R3) in Figure 7.2(a), (b) and (c) respectively. While significant scatter is clearly evident for all methods, the ICP-05 and UWA-05 methods show better agreement with the installation resistance $q_{cp}$ in comparison to the EF-97 method (which assumes $q_{b,0.1Db} = q_{c,avg}$). No attempt is made to correlate $q_{cp}$ with either method due to the reliability issues associated with pile driving formulae in general (as discussed by Fellenius 2014), but the installation resistance may provide an initial, albeit crude estimate of the base
resistance at failure $q_{b,0.1Db}$ of a DCIS pile during a subsequent maintained compression load test, as well as useful information regarding the variation in the ground profile across a site during piling operations.

Figure 7.2 - Comparison of installation resistance with theoretical base resistance derived using (a) ICP-05 (b) EF-97 and (C) UWA-05 methods
7.3 CURING

7.3.1 Hydration temperatures

The review of published hydration temperature data within cast-in-situ piles in Section 2.6.2 of Chapter 2 demonstrated that the time to peak temperature after casting is a function of pile diameter. Figure 7.3 shows the variation in time to peak temperature after casting with pile shaft diameter for the test piles at Dagenham, Erith and Shotton (the temperature data within the 700 mm wide pile cap at Shotton is also included), together with the published data from Figure 3.15. The time to peak temperature for the 320 mm diameter instrumented DCIS piles varies from 8 hours to 19 hours, with the variability likely to be affected by ambient ground temperature, concrete mix and ground conditions such as water level and soil type. Despite the variability in the measured data, a linear trend of time to peak temperature is apparent (over the range 0.3 m < $D_s$ < 3.2 m) which can be described as:

$$T_{\text{peak}} = 23.11D_s$$  \hspace{1cm} (7.1)

Equation 2.2 provides a useful reference for the time to peak temperature in order to assess the onset of residual load development based on the strain interpretation Method 2 by Pennington (1995).

The time for hydration temperatures to reduce from peak values to 50 % of the excess $T_{50\%}$ with pile shaft diameter for the DCIS test piles and the published data in Table 2.7 of Chapter 2 is illustrated in Figure 7.4; some scatter in the data is once again present for the test piles, in particular for Pile E3 at Erith where the vibrating wire strain gauges exhibited somewhat erratic strain and temperature behaviour after casting. However, in a similar manner to Figure 7.3, a linear relationship between $T_{50\%}$ and pile shaft diameter is evident which can be expressed as:

$$T_{50\%} = 80.24D_s$$  \hspace{1cm} (7.2)

Equation 7.2 may provide a useful reference for estimating the period in which hydration temperatures will have an influence on the measured strains during a maintained load test with a view to separating out other influences.
Figure 7.3 - Variation in time to peak temperature after casting with shaft diameter

Figure 7.4 - Variation in time between peak and 50% excess temperature with shaft diameter
7.3.2 Curing strains

In order to assess the potential development of residual loads during curing, continuous measurements of temperature and strain after casting were obtained within the instrumented test piles installed in layered soil at Dagenham (Pile D1) and Erith (Pile E3) in Chapter 5, and uniform sand at Shotton (Pile S1) in Chapter 6. This section examines the three residual load interpretation methods (reviewed in Section 3.5) using the strain profiles measured during curing and how each profile influences the subsequent load distribution at failure. Figure 7.5(a) presents the interpreted residual load profiles for Pile D1 Dagenham, while the distribution of residual load at Shotton is shown in Figure 7.5(b). It is apparent that:

- The residual load profiles tend to vary considerably with interpretation method, regardless of whether the pile is installed in cohesive or granular soil.
- Significant tensile loads (> 400 kN) were derived using Method 1 for Pile D1 and Pile S1. Such tensile loads would inevitably lead to cracking of the concrete, but no evidence of cracking was apparent from the measured strain data during subsequent loading testing.
- The distribution of residual load obtained using Method 3 for Pile D1 is near-identical to the expected dragload profile of a pile experiencing negative skin friction due to consolidation-related settlement of the alluvial layer (see Figure 5.12 in Section 5.3.3). Method 3 also gives minor tensile loads (∼ 20 kN) for Pile S1 at Shotton which would appear reasonable due to shrinkage effects.

The interpreted profiles of residual load were added to the measured load distribution derived from the subsequent maintained compression load test in order to obtain the ‘true’ overall load distribution with the test piles at failure. The resulting distributions of load at failure for Pile D1 and Pile S1 are shown in Figure 7.6(a) and (b) respectively where it evident that:

- The distributions derived using Methods 1 and 2 show increases in load with depth, implying that negative skin friction is present along the shaft of the pile at failure; this result is considered unrealistic as both piles experienced large settlements (and hence fully mobilised positive shear stresses) in the latter stages of each load test.
- The profile for Pile D2 using Method 2 suggests that higher loads are present in the pile at failure than that applied at the pile head; this result is also unrealistic.
- Method 3 provides the most sensible load distribution for both piles, with the loads monotonically decreasing with depth towards the base (as would be expected).
Figure 7.5 - Interpreted residual load profiles for (a) Pile D1 in soft layered soil at Dagenham and (b) Pile S1 in sand at Shotton

Figure 7.6 - Load distribution at failure for (a) Pile P1 at Dagenham and (b) Pile S1 at Shotton
Based on the distributions in Figure 7.5 and Figure 7.6, it is concluded that Method 3 provides the most plausible distribution of residual loads after installation and curing of the instrumented DCIS piles in Chapters 5 and 6. Based on the results for Pile S1, residual loads for DCIS piles in uniform sand are considered to be negligible and can therefore be ignored. On the other hand, the quantity of residual load in a soft cohesive layer can be reasonably estimated using standard design equations for negative skin friction such as those by Meyerhof (1976) as demonstrated in Figure 5.12 of Chapter 5.

Fellenius et al. (2009) postulated that the tensile strains measured in cast-in-situ piles during cooling is a consequence of swelling of the concrete as it absorbs moisture during hydration. While tensile strains were apparent in the instrumented DCIS piles at Pontarddulais, Dagenham, Shotton and Ryton-on-Dunsmore prior to commencing the maintained compression load tests, the fact that the test piles at Ryton-on-Dunsmore were installed in unsaturated sand does not support the theory that the tensile strains are a result of swelling. A more plausible reason for the tensile strain response is restraint of the pile by the soil as it shrinks during cooling (such behaviour is widely-known to occur in concrete slabs for example). The level of restraint is unlikely to be sufficient to mobilise shear stresses along the shaft of the pile (Fellenius, 2012b), but may lead to fractures within the concrete if the surrounding soil is relatively stiff (Sinnreich, 2012). This may also provide an explanation for the discontinuous strain measurements during loading of Pile R2 and R3 at Ryton-on-Dunsmore (see Section 6.3.5).
7.4 LOAD TESTING

7.4.1 Introduction

In this section, the results of the maintained compression load tests on the instrumented DCIS pile tests presented in Chapters 5 and 6 are discussed. The results are examined in terms of shaft and base resistance, with references made throughout to the behaviour of preformed closed-ended pile behaviour as reviewed in Chapter 2.

7.4.2 Shaft resistance

7.4.2.1 Average shear stress

Figure 7.7 shows the variation in normalised average shear stress $\tau_{s,avg}/q_{c,avg}$ generated within the sand layers with normalised pile head displacement $w/D_s$ for Pile S1 at Shotton and Piles R1, R2 and R3 at Ryton-on-Dunsmore. As discussed in Section 6.3.6 of Chapter 6, the values of $\tau_{s,avg}/q_c$ for Piles R1 and R3 at Ryton-on-Dunsmore are greater than the typical range for closed-ended steel displacement piles reported by Gavin and Lehane (2003) which is attributed to the greater interface roughness of the DCIS piles due to in-situ concreting. A slightly reduced normalised average shear stress $\tau_{s,avg}/q_c$ of $\approx 0.01$ was mobilised for Pile S1 at Shotton which was partly-explained by the two unload/reload cycles which Pile S1 experienced during the load test which led to reductions in shear stress due to friction fatigue.

Figure 7.7 - Variation in normalised average shear stress with displacement for Pile S1 at Shotton and Piles R1, R2 and R3 at Ryton-on-Dunsmore
7.4.2.2 Alpha shaft resistance coefficients

As discussed in Section 2.5.2 of Chapter 2, traditional CPT-based methods relate the cone resistance $q_c$ to the local shear stress $\tau_{sf}$ of a displacement pile in sand using the expression $\alpha_s = q_c / \tau_{sf}$. Figure 7.8 examines the relationship between cone resistance $q_c$ and the shaft coefficients $\alpha_s$ back-figured from the local shear stresses measured within the cohesionless layers of the instrumented DCIS pile in Chapters 5 and 6. The $\alpha_s$ values specified by the LCPC-82 method (Bustamante and Gianeselli, 1982) for DCIS piles (category 1B) and driven piles (category 2A) are also included in Figure 7.8 for comparative purposes. It is noteworthy that:

- Considerable variability in the measured $\alpha_s$ values is evident, ranging from $\approx 34$ to $\approx 260$ for $q_c > 12$ MPa for example, which is attributed not only to the strain interpretation process outlined previously, but also to the inability of a single coefficient to capture complex phenomena such as friction fatigue which influence displacement pile behaviour in sand (Lehane, 2008).

The shaft coefficients for DCIS piles recommended by the LCPC-82 method tend to over-predict the measured values of $\alpha_s$ significantly, with closer agreement obtained with the measured data using the driven coefficients (i.e. category 2A) for $q_c$ values ranging from 5 MPa to 12 MPa.

![Figure 7.8 - Variation in shaft resistance coefficient with cone resistance](image-url)
7.4.2.3 Local shear and radial effective stresses at failure

The peak local shear stresses at failure $\tau_{sf}$ for the instrumented DCIS pile tests in Chapter 6, as well as the sections of the instrumented piles within the sand and gravel layers in Chapter 5, were normalised by the corresponding local cone resistance $q_c$ between the gauge levels in question and plotted against normalised distance from the pile base $h/D_s$ in Figure 7.9(a). While some scatter is evident (which is understandable given the variable nature of concrete and the aforementioned difficulties associated with the interpretation of the measured strains), a trend of $\tau_{sf}/q_c$ reducing with increasing $h/D_s$ is apparent. Equation 2.6 was used to back-figure radial effective stresses at failure $\sigma'_{rf}$ from the measured local shear stresses using constant-volume friction angles $\phi'_{cv}$ which were either measured in direct shear tests (i.e. at Ryton-on-Dunsmore) or assumed based on the results of studies by Bolton (1986) and Paul et al. (1994) for sand and gravel respectively. A summary of the shaft resistance parameters $\tau_{sf}$, $q_c$ and $\phi'_{cv}$ is provided in Table 7.2.

Figure 7.9(b) shows the variation in normalised radial effective stress $\sigma'_{rd}/q_c$ with $h/D_s$. In a similar manner to Figure 7.9(a), some scatter is present in the results (primarily for Pile S1 at Shotton), but evidence of friction fatigue is apparent by the reduction in $\sigma'_{rd}/q_c$ with increasing $h/D_s$. Also shown in Figure 7.9(b) is the empirical profile of normalised radial effective stress after installation and equalisation $\sigma'_{rc}/q_c$ obtained using the UWA-05 method for comparative purposes. As discussed in the review of the shaft resistance of closed-ended displacement piles in sand in Section 2.3, the primary mechanism governing friction fatigue in sand is the cyclic shearing of the soil particles at the pile-soil interface during driven or jacked installation. Given that DCIS piles are installed by driving a closed-ended steel tube, this mechanism is considered applicable and the data in Figure 7.9(b) provides strong evidence that the radial stress acting on the shaft of a DCIS pile reduces with increasing distance from the pile base. Figure 7.10 shows the variation in normalised local shear stress at failure $\tau_{sd}/q_c$ with the number of cycles $N$ experienced during both installation and subsequent maintained compression load testing (if load cycles were performed); while variability in the data is once again observed for reasons outlined previously, $\tau_{sd}/q_c$ appears to reduce with $N$ in a similar manner to that observed for model steel centrifuge piles by White and Lehane (2004).
### Table 7.2 - Summary of shaft resistance parameters from instrumented pile tests

<table>
<thead>
<tr>
<th>Site</th>
<th>Pile ref</th>
<th>Mid-point between gauge levels (m)</th>
<th>Peak local shear stress $\tau_{sf}$ (kPa)</th>
<th>Cone resistance between gauges $q_c$ (MPa)</th>
<th>Constant-volume friction angle $\phi'_c$ (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pontarddulais</td>
<td>Pile P1</td>
<td>7.65</td>
<td>41.5</td>
<td>2.77</td>
<td>33°</td>
</tr>
<tr>
<td>Dagenham</td>
<td>Pile D1</td>
<td>2.25</td>
<td>66.8</td>
<td>16.11</td>
<td>33°</td>
</tr>
<tr>
<td></td>
<td></td>
<td>7.30</td>
<td>459.6</td>
<td>14.87</td>
<td>38°</td>
</tr>
<tr>
<td>Erith</td>
<td>Pile E3</td>
<td>8.93</td>
<td>105.9</td>
<td>10.04</td>
<td>33°</td>
</tr>
<tr>
<td>Shotton</td>
<td>Pile S1</td>
<td>3.25</td>
<td>119.9</td>
<td>7.80</td>
<td>33°</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4.75</td>
<td>104.4</td>
<td>14.47</td>
<td>33°</td>
</tr>
<tr>
<td>Ryton-on-Dunsmore</td>
<td>Pile R1</td>
<td>2.50</td>
<td>162.2</td>
<td>14.02</td>
<td>35</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3.75</td>
<td>127.0</td>
<td>15.41</td>
<td>35</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5.18</td>
<td>196.2</td>
<td>11.78</td>
<td>35</td>
</tr>
<tr>
<td></td>
<td>Pile R2</td>
<td>3.00</td>
<td>99.9</td>
<td>16.05</td>
<td>35</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4.63</td>
<td>107.2</td>
<td>9.83</td>
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</tr>
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<td></td>
<td></td>
<td>6.05</td>
<td>202.9</td>
<td>14.80</td>
<td>35</td>
</tr>
<tr>
<td></td>
<td>Pile R3</td>
<td>2.75</td>
<td>150.5</td>
<td>16.50</td>
<td>35</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4.00</td>
<td>120.0</td>
<td>12.39</td>
<td>35</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4.93</td>
<td>174.9</td>
<td>10.05</td>
<td>35</td>
</tr>
</tbody>
</table>

*Bolton (1986); *Paul et al. (1994)*

![Figure 7.9](image-url) - Variation in (a) normalised local shear stress and (b) normalised radial effective stress at failure with normalised distance from pile base
A linear regression analysis, shown in Figure 7.11(a), has already been performed on the normalised radial stress data for the three test piles at Ryton-on-Dunsmore (see Figure 6.20a in Section 6.3.6) using the Ordinary Least Squares (OLS) method, resulting in the following best-fit relationship for the reduction in normalised radial stress at failure $\sigma'_{rf}/q_c$ with $h/D_s$:

$$\sigma'_{rf} = 0.0313 q_c \left( \frac{h}{D_s} \right)^{-0.4211}$$  \hspace{1cm} 7.3

Figure 7.11(b) shows the linear regression analysis performed for all data points of the instrumented DCIS piles in Chapters 5 and 6 in cohesionless layers. The overall expression for the variation in radial effective stress $\sigma'_{ef}$ with normalised distance from the pile base $h/D_s$ is given by:

$$\sigma'_{ef} = 0.0342 q_c \left( \frac{h}{D_s} \right)^{-0.467}$$  \hspace{1cm} 7.4

Equations 7.3 and 7.4 are surprisingly similar to the relationship between the radial effective stress after installation and equalisation $\sigma'_{re}$ and normalised distance from the base $h/D_s$ specified by the UWA-05 method (Lehane et al., 2005c). However, Equation 7.4 has a coefficient of determination $R^2$ of $\approx 58\%$ which is considerably less than values of $80\%$ or greater observed for similar regression analyses on steel displacement piles by Lehane (1992) and Chow (1997). Given the issues associated with the data points of Pile S1 at Shotton (due to the misrepresentative CPT $q_c$ profile as discussed previously), a final regression analysis was
conducted on the overall dataset with Pile S1 excluded. The analysis, shown in Figure 7.11(c), resulted in an improved $R^2$ value of $\approx 84\%$ with the following expression for $\sigma'_\text{rf}$:

$$\sigma'_\text{rf} = 0.0387q_c \left( \frac{h}{D_s} \right)^{0.542}$$  \hspace{1cm} 7.5

As discussed in Section 2.3 of Chapter 2, increases in radial stress observed by Lehane (1992) and Chow (1997) during loading of a 102 mm diameter steel model pile in sand were primarily attributed to dilation of the sand particles at the pile-soil interface during shearing, with the radial effective stress at failure $\sigma'_{\text{rf}}$ given by:

$$\sigma'_{\text{rf}} = \sigma'_{\text{rc}} + \Delta \sigma'_{\text{rd}}$$  \hspace{1cm} 7.6

Given the close agreement between the measured normalised radial effective stress at failure $\sigma'_{\text{rd}}/q_c$ and the normalised radial stress after installation and equalisation $\sigma'_{\text{rc}}/q_c$ predicted by the UWA-05 method (as illustrated in Figure 7.10b), it is reasonable to conclude that, in a similar manner to the results for CFA piles in sand by Gavin et al. (2009) discussed in Section 2.3.6, the dilation-related increase in radial stress during loading $\Delta \sigma'_{\text{rd}}$ represents a small proportion of the overall radial effective stress at failure $\sigma'_{\text{rf}}$ for the instrumented DCIS test piles. One exception to this trend was the shaft resistance measured within the sandy gravel socket during maintained compression load testing of Pile D1 at Dagenham where a local shear stress $\tau_{s,\text{loc}}$ in excess of 400 kPa was observed during the latter stages of loading. The rough interface created during pile construction most likely led to greater magnitudes of radial displacement of the sandy gravel particles during dilation, resulting in large increases in radial stress as failure approached. The data in Figure 7.9, 7.10 and 7.11 provide strong evidence that friction fatigue is a characteristic of DCIS piles and that the radial stresses induced during driven installation of the steel tube are not erased upon concreting of the pile. It is therefore concluded that the shaft resistance of a DCIS in sand is comparable to that of a preformed displacement pile.
Figure 7.11 - Linear regression analyses on radial stress data for (a) Ryton-on-Dunsmore, (b) overall instrumented DCIS pile tests and (c) overall excluding Shotton Pile S1
7.4.3 Base resistance and stiffness

The instrumented pile tests in Chapters 5 and 6 enabled an accurate assessment of the base resistance of DCIS piles during maintained compression load testing in sand. The measured base resistances at failure \( q_{b,0.1Db} \) of the instrumented DCIS piles were normalised by their corresponding average cone resistance \( q_{c,avg} \) at the base, derived using the Dutch averaging method described in Section 2.4.5, and plotted against (a) average cone resistance \( q_{c,avg} \), (b) base diameter \( D_b \) and (c) relative density \( D_r \) at the pile base in Figure 7.12(a), (b) and (c) respectively. The results of the instrumented 430 mm diameter DCIS pile at Le Havre reported by Evers et al. (2003) and the interpreted normalised base resistances of the piles at Kallo (De Beer et al., 1979; Chow, 1997; Xu et al., 2008) are also included, as well as the database for bored cast-in-situ piles compiled by Gavin et al. (2013) for comparative purposes. It is apparent that:

- With the exception of the data for Pile S1 at Shotton and Pile R3 at Ryton-on-Dunsmore (which are discussed below), no systematic variation in \( q_{b,0.1Db}/q_{c,avg} \) with average cone resistance \( q_{c,avg} \), base diameter \( D_b \) or relative density \( D_r > 40 \% \) is evident. This observation is in agreement with the investigation of the base resistance of closed-ended displacement piles in sand by Xu et al. (2008).

- The measured \( q_{b,0.1Db}/q_{c,avg} \) values show excellent agreement with the normalised base resistance \( q_{b,0.1Db}/q_{c,avg} = 0.6 \) recommended for closed-ended driven piles by the UWA-05 method (Xu et al., 2008). However, the data points for Pile S1 at Shotton and Pile R3 at Ryton-on-Dunsmore yielded \( q_{b,0.1Db}/q_{c,avg} \) ratios in excess of 0.95; such values are typically observed in jacked piles only. As discussed in Section 6.2.7, the high ratio at Shotton (\( q_{b,0.1Db}/q_{c,avg} \approx 1.08 \)) was attributed to increases in soil density following nearby anchor pile installation, resulting in the value of \( q_{c,avg} \) being underestimated. Pile R3 at Ryton-on-Dunsmore exhibited an unexpectedly high base resistance, despite the installation resistance showing good agreement with the measured CPT \( q_c \) profile. It was therefore concluded that a dense layer was present beneath the pile which was not encountered by the CPT conducted \( \approx 3 \) m away.

- The values of \( q_{b,0.1Db}/q_{c,avg} \) are significantly greater than \( q_{b,0.1Db}/q_{c,avg} \approx 0.2 \) typically reported for traditional bored cast-in-situ pile types in sand (Lee and Salgado, 1999; Lehane, 2008; Gavin et al., 2013). However, such an observation is hardly surprising given that DCIS piles led to significant displacement and pre-stressing of the soil beneath the base during installation and loading.
Figure 7.12 - Variation in normalised base resistance with (a) average cone resistance at base (b) diameter and (c) relative density
The equivalent linear secant base stiffness $E_{b,eq}$ of each test pile during loading was backfigured from the measured base resistance-displacement curves using Equation 2.10 and subsequently normalised by the corresponding Dutch average cone resistance $q_{c,avg}$. Figure 7.13 shows the resulting variation in $E_{b,eq}/q_{c,avg}$ with normalised base displacement $w_b/D_b$ for the instrumented DCIS test piles in Chapters 5 and 6. Based on the comments made in relation to Figure 7.12, the values of $q_{c,avg}$ for Pile D1 at Dagenham, Pile S1 at Shotton and Pile R3 at Ryton-on-Dunsmore were derived from the measured base resistance at failure $q_{b,0.1D_b}$ using the expression $q_{c,avg} = q_{b,0.1D_b}/0.6$ recommended by the UWA-05 method. It is apparent in Figure 7.13 that the degradation in base stiffness occurs gradually for all piles with increasing normalised displacement $w_b/D_b$ during loading and is therefore in constrast with the normalised base displacement $w_b/D_b$ of 1.5% advocated by Gavin and Lehane (2007) for stiffness degradation of closed-ended steel displacement piles. This outcome is unsurprising however, as the closed-ended displacement piles in Gavin and Lehane’s (2007) study were installed by jacking and hence exhibited a stiffer base resistance-displacement response during loading (due to greater residual loads after each jacking stroke) in comparison to driven piles.

\[ E_{b,eq} = q_b D_b (1 - \nu^2) \pi / 4 w_b \]

*Assumes $q_{c,avg} = q_{b,0.1D_b,meas}/0.6$

**Figure 7.13 - Variation in normalised base stiffness with normalised base displacement**

Figure 7.14(a) shows the variation in bearing capacity factor $N_{q,0.1D_b}$, back-figured from the measured base resistance at failure $q_{b,0.1D_b}$ using Equation 2.9, with relative density $D_r$ derived using the correlation with $q_c$ by Jamiolkowski et al. (2003). The value of $D_r$ for Dagenham was derived using the cone resistance $q_c$ at refusal as the average resistance could not be obtained over a distance of $1.5D_b$ above and below the tip. The bearing capacity factor is seen to increase significantly with relative density, with values in excess of 50 for $D_r > 50\%$. As
shown in Figure 7.14(b), the bearing capacity factors for the instrumented DCIS piles are also in keeping with the range of $N_q$ values reported for closed-ended displacement piles by Chow (1997).

Figure 7.14 - Comparison of the variation in bearing capacity factor with relative density for (a) DCIS piles and (b) overall database of closed-ended piles by Chow (1997)
7.5 ASSESSMENT OF CPT-BASED DESIGN METHODS FOR ESTIMATING DCIS PILE CAPACITY IN SAND

7.5.1 Introduction
The analysis of the results of the instrumented DCIS pile tests in the previous sections of this chapter demonstrated that DCIS piles behave in a near-identical manner to preformed displacement piles in sand during maintained compression loading. It is therefore of interest to assess the applicability of current CPT-based design methods for estimating the total, shaft and base capacities of DCIS piles in sand. To this end, a database of load tests on DCIS piles (including the instrumented tests in Chapters 5 and 6) has been developed and the corresponding CPT $q_c$ profiles used to estimate the total, shaft and base capacities using the various CPT-based methods described in Section 2.5 of Chapter 2.

7.5.2 DCIS pile database
In order to assess the applicability of current CPT-based displacement pile design methods in estimating the capacity of DCIS piles in sand, a database of maintained compression load tests with adjacent CPT $q_c$ profiles was assembled. Given the dearth of reported studies of tests on DCIS piles in the literature, the average quality of the tests in the database may be lower in comparison to other databases used to develop the recent CPT-based design methods such as the ICP-05 and UWA-05 methods (see Section 2.5.3 for further details). The overall database contains a total of 26 piles which are subdivided into 17 non-expanded base DCIS piles and 9 expanded-base ‘Franki’ piles. Details of the DCIS and Franki databases are given in Table 7.3 and Table 7.4 respectively. The bases of all piles were founded in cohesionless soil, i.e. sand or gravel, and the piles were subjected to maintained compression load tests. With the exception of the instrumented tests presented in Chapters 5 and 6, the database contains only 2 instrumented DCIS pile tests which were reported by Evers et al. (2003) at a layered site in Le Havre, France. Three simplified CPT methods, EF-97, LCPC-82 and Van Impe-86, were assessed, together with the four recent methods - Fugro-05, ICP-05, NGI-05 and UWA-05. The LCPC-82 method was assessed using the shaft coefficients for both DCIS piles (category 1B) and driven closed-ended piles (category 2A). The CPT at Dagenham met refusal at a depth corresponding to the base of the test pile (Pile D1), preventing appropriate averaging of the $q_c$ profile below the base and was therefore excluded during assessment of method performance in estimating the base and total capacity of the DCIS piles.
Table 7.3 - DCIS pile database with CPT profiles

<table>
<thead>
<tr>
<th>Site; Pile Ref</th>
<th>Shaft Diameter</th>
<th>Base Diameter</th>
<th>Length</th>
<th>Pile Capacity</th>
<th>Slenderness ratio</th>
<th>Cone resistance at pile base</th>
<th>Weighted normalised cone resistance along shaft</th>
<th>Normalised cone resistance at base</th>
<th>Weighted relative density along shaft</th>
<th>Relative density at base</th>
<th>Instrumented</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pontarddulais; P1</td>
<td>320</td>
<td>380</td>
<td>8.50</td>
<td>949&lt;sup&gt;a&lt;/sup&gt;</td>
<td>25.00</td>
<td>4.13</td>
<td>122.42</td>
<td>37.40</td>
<td>53.44</td>
<td>28.74</td>
<td>✔</td>
<td>Section 5.2</td>
</tr>
<tr>
<td>Shotton; S1</td>
<td>320</td>
<td>380</td>
<td>5.75</td>
<td>2214</td>
<td>15.13</td>
<td>21.01</td>
<td>165.67</td>
<td>261.91</td>
<td>65.62</td>
<td>81.50</td>
<td>✔</td>
<td>✔</td>
</tr>
<tr>
<td>Dagenham; D1</td>
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<td>380</td>
<td>7.70</td>
<td>2493</td>
<td>20.26</td>
<td>51.99</td>
<td>254.72</td>
<td>74.96</td>
<td>65.62</td>
<td>78.53</td>
<td>✔</td>
<td>✔</td>
</tr>
<tr>
<td>Erith; E1</td>
<td>320</td>
<td>380</td>
<td>10.80</td>
<td>2872&lt;sup&gt;b&lt;/sup&gt;</td>
<td>28.42</td>
<td>18.31</td>
<td>122.42</td>
<td>4.13</td>
<td>37.40</td>
<td>53.44</td>
<td>✔</td>
<td>✔</td>
</tr>
<tr>
<td>Erith; E3</td>
<td>320</td>
<td>380</td>
<td>11.10</td>
<td>1879</td>
<td>24.58</td>
<td>104.07</td>
<td>188.23</td>
<td>63.15</td>
<td>84.02</td>
<td>74.96</td>
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<td>✔</td>
</tr>
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<td>Ryton; R1</td>
<td>320</td>
<td>380</td>
<td>6.00</td>
<td>1622</td>
<td>15.79</td>
<td>15.38</td>
<td>102.13</td>
<td>37.40</td>
<td>53.44</td>
<td>28.74</td>
<td>✔</td>
<td>✔</td>
</tr>
<tr>
<td>Ryton; R2</td>
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<td>380</td>
<td>7.00</td>
<td>1578</td>
<td>18.42</td>
<td>20.24</td>
<td>107.99</td>
<td>37.40</td>
<td>53.44</td>
<td>28.74</td>
<td>✔</td>
<td>✔</td>
</tr>
<tr>
<td>Ryton; R3</td>
<td>320</td>
<td>380</td>
<td>5.50</td>
<td>2276&lt;sup&gt;c&lt;/sup&gt;</td>
<td>14.47</td>
<td>13.93</td>
<td>97.91</td>
<td>37.40</td>
<td>53.44</td>
<td>28.74</td>
<td>✔</td>
<td>✔</td>
</tr>
<tr>
<td>Kallo; K5</td>
<td>406</td>
<td>406</td>
<td>9.33</td>
<td>1666</td>
<td>22.98</td>
<td>25.90</td>
<td>141.38</td>
<td>37.40</td>
<td>53.44</td>
<td>28.74</td>
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<td>✔</td>
</tr>
<tr>
<td>Kallo; K6&lt;sup&gt;a,b&lt;/sup&gt;</td>
<td>406</td>
<td>406</td>
<td>11.39</td>
<td>4306</td>
<td>28.05</td>
<td>32.60</td>
<td>270.89</td>
<td>319.92</td>
<td>72.48</td>
<td>90.74</td>
<td>×</td>
<td>×</td>
</tr>
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<td>Kallo; K7</td>
<td>406</td>
<td>609</td>
<td>9.37</td>
<td>2795</td>
<td>15.39</td>
<td>26.50</td>
<td>149.82</td>
<td>289.92</td>
<td>51.10</td>
<td>81.29</td>
<td>×</td>
<td>×</td>
</tr>
<tr>
<td>Le Havre; A4</td>
<td>430</td>
<td>430</td>
<td>10.50</td>
<td>1673</td>
<td>24.42</td>
<td>10.90</td>
<td>191.23</td>
<td>127.17</td>
<td>74.66</td>
<td>62.33</td>
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<td>✔</td>
</tr>
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<td>Le Havre; C1</td>
<td>410</td>
<td>430</td>
<td>10.50</td>
<td>1882</td>
<td>24.42</td>
<td>10.90</td>
<td>191.23</td>
<td>127.17</td>
<td>74.66</td>
<td>62.33</td>
<td>✔</td>
<td>✔</td>
</tr>
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<td>Ringsend; TP20</td>
<td>425</td>
<td>425</td>
<td>12.50</td>
<td>3275&lt;sup&gt;c&lt;/sup&gt;</td>
<td>29.41</td>
<td>12.50</td>
<td>122.82</td>
<td>274.23</td>
<td>67.89</td>
<td>78.53</td>
<td>×</td>
<td>×</td>
</tr>
<tr>
<td>Franki-Grundbau; FG1</td>
<td>420</td>
<td>420</td>
<td>26.00</td>
<td>5199&lt;sup&gt;c&lt;/sup&gt;</td>
<td>61.90</td>
<td>15.80</td>
<td>396.35</td>
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<td>47.02</td>
<td>80.73</td>
<td>×</td>
<td>×</td>
</tr>
<tr>
<td>Gwizdala; G1</td>
<td>457</td>
<td>650</td>
<td>20.50</td>
<td>4557&lt;sup&gt;c&lt;/sup&gt;</td>
<td>31.54</td>
<td>18.69</td>
<td>155.90</td>
<td>52.47</td>
<td>68.62</td>
<td>×</td>
<td>×</td>
<td>Gwizdala &amp; Krasinski (2013)</td>
</tr>
<tr>
<td>Gdansk Port; GP1</td>
<td>508</td>
<td>620</td>
<td>13.50</td>
<td>4053&lt;sup&gt;c&lt;/sup&gt;</td>
<td>21.77</td>
<td>19.00</td>
<td>160.27</td>
<td>59.24</td>
<td>67.87</td>
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<td>×</td>
<td>Gwizdala &amp; Dyka (2002)</td>
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<tr>
<td>Gdansk; GK1</td>
<td>508</td>
<td>560</td>
<td>21.50</td>
<td>6704&lt;sup&gt;c&lt;/sup&gt;</td>
<td>38.39</td>
<td>31.50</td>
<td>153.87</td>
<td>75.41</td>
<td>66.62</td>
<td>×</td>
<td>×</td>
<td>Gwizdala &amp; Krasinski (2013)</td>
</tr>
</tbody>
</table>

<sup>a</sup>Permanent steel casing; <sup>b</sup>Increase in diameter to 558 mm from 8.3 m to 9.6 m bgl; <sup>c</sup>Extrapolated to 0.1D<sub>b</sub> using method by Chin (1972).
### Table 7.4 - Franki pile database with CPT profiles

<table>
<thead>
<tr>
<th>Site; Pile Ref</th>
<th>Shaft Diameter $D_s$ (mm)</th>
<th>Base Diameter $D_b$ (mm)</th>
<th>Length $L$ (m)</th>
<th>Pile Capacity $Q_{1D,1D_{m,b}}$ (kN)</th>
<th>Slenderness ratio $L/D_b$</th>
<th>Cone resistance at pile base $q_{c_{tip}}$ (MPa)</th>
<th>Weighted normalised cone resistance along shaft $q_{cN,s}$</th>
<th>Normalised cone resistance at base $q_{cN,b}$</th>
<th>Weighted relative density along shaft $D_{r,s}$ (%)</th>
<th>Relative density at base $D_{r,b}$ (%)</th>
<th>Instrumented</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kallo; K1</td>
<td>520</td>
<td>908</td>
<td>9.69</td>
<td>6103</td>
<td>38.39</td>
<td>27.90</td>
<td>156.44</td>
<td>294.22</td>
<td>55.75</td>
<td>82.27</td>
<td>×</td>
<td>De Beer et al. (1979)</td>
</tr>
<tr>
<td>Kallo; K2</td>
<td>323</td>
<td>539</td>
<td>9.71</td>
<td>2697</td>
<td>10.67</td>
<td>28.10</td>
<td>193.88</td>
<td>321.80</td>
<td>59.71</td>
<td>82.44</td>
<td>×</td>
<td>De Beer et al. (1979)</td>
</tr>
<tr>
<td>Kallo; K3</td>
<td>406</td>
<td>615</td>
<td>9.82</td>
<td>3205</td>
<td>18.01</td>
<td>29.20</td>
<td>155.27</td>
<td>317.66</td>
<td>51.12</td>
<td>83.34</td>
<td>×</td>
<td>De Beer et al. (1979)</td>
</tr>
<tr>
<td>Kallo; K4</td>
<td>406</td>
<td>815</td>
<td>9.80</td>
<td>5975</td>
<td>15.97</td>
<td>28.90</td>
<td>126.68</td>
<td>309.78</td>
<td>47.36</td>
<td>83.08</td>
<td>×</td>
<td>De Beer et al. (1979)</td>
</tr>
<tr>
<td>Berlin; P1</td>
<td>420</td>
<td>870</td>
<td>10.00</td>
<td>1828$^b$</td>
<td>11.49</td>
<td>3.75</td>
<td>48.76</td>
<td>44.10</td>
<td>36.33</td>
<td>×</td>
<td>Briek (1993)</td>
<td></td>
</tr>
<tr>
<td>Berlin; P5</td>
<td>500</td>
<td>860</td>
<td>10.00</td>
<td>5659$^b$</td>
<td>11.63</td>
<td>3.65</td>
<td>33.93</td>
<td>46.30</td>
<td>29.47</td>
<td>×</td>
<td>Briek (1993)</td>
<td></td>
</tr>
<tr>
<td>Hamburg; H1</td>
<td>420</td>
<td>850</td>
<td>8.50</td>
<td>5801$^b$</td>
<td>10.00</td>
<td>12.60</td>
<td>150.69</td>
<td>51.33</td>
<td>66.23</td>
<td>×</td>
<td>Briek &amp; Garbers (2011)</td>
<td></td>
</tr>
<tr>
<td>Ghent; P85</td>
<td>520</td>
<td>800</td>
<td>13.00</td>
<td>5108$^b$</td>
<td>16.25</td>
<td>9.13</td>
<td>96.04</td>
<td>71.63</td>
<td>54.48</td>
<td>×</td>
<td>Goossens &amp; Van Impe (1991)</td>
<td></td>
</tr>
<tr>
<td>Ghent; P585</td>
<td>520</td>
<td>800</td>
<td>13.00</td>
<td>4031$^b$</td>
<td>16.25</td>
<td>9.13</td>
<td>96.04</td>
<td>71.63</td>
<td>54.48</td>
<td>×</td>
<td>Goossens &amp; Van Impe (1991)</td>
<td></td>
</tr>
</tbody>
</table>

$^a$Permanent steel casing; $^b$Extrapolated to 0.1D$_b$ using method by Chin (1972)
7.5.3 Capacity calculation procedures

The following procedures and assumptions were followed when developing the database:

- The average CPT $q_c$ profiles for the sites in Table 7.3 and Table 7.4 were digitized at intervals of 0.1 m or less and spreadsheets were developed to calculate the shaft and base capacity for each method. No correction was applied at Kallo to account for the use of a mechanical cone penetrometer.

- The depth to the water-table was not reported for 7 of the 15 sites, and was therefore assumed to be at the ground surface. This assumption, in keeping with that made by others using these sites in databases, leads to a lower-bound prediction of capacity for each method due to the reduction in effective stress along the shaft of the pile.

- In general, $\Delta f_i$ was assumed to be equivalent to $\phi'_{cv}$ (33° in sand, 35° at Ryton-on-Dunsmore and 38° in sandy gravel as mentioned previously). Three piles at Kallo had permanently-cased steel shafts, and a value of $\Delta f_i = 29°$ was assumed (Jardine et al., 2005) for the portions of the pile shafts in sand in the absence of grading data.

- The majority of the sites in Table 7.3 and Table 7.4 had a multi-layered stratigraphy comprising soft clay, sand and sandy gravel. Therefore, in order to account for the clay layers in the recent CPT methods, the peak local shaft friction $\tau_{sf}$ in the clay layers was estimated using the expression $\tau_{sf} = q_t/35$, where $q_t$ is the cone resistance (kPa) corrected for end effects, based on the procedure by Schneider et al. (2008). With the exception of the pile tests at Kallo, no sites had clay layers which contributed more than 50% of the shaft capacity.

- The load tests for 13 piles (see Table 7.3 and Table 7.4) were terminated prior to reaching a displacement corresponding to $0.1D_b$, and the results were therefore extrapolated accordingly using the hyperbolic method by Chin (1972).

- Six sites had multiple pile tests and these tests were considered individually, rather than averaged, due to the relatively limited number of pile tests in the overall database. However, it is acknowledged that this may introduce some bias into the prediction performance for each method.

- In order to account for soil layering, the average normalised cone resistance $q_{c1N,s}$ and relative density $D_{rs}$ along the shaft of each pile were weighted in accordance with the distribution of shaft resistance calculated using the UWA-05 method.
7.5.4 Total capacity

Table 7.5 summarizes the resulting ratios of calculated to measured total capacity, $Q_c/Q_m$, for each pile in the database, as well as the predictive performance of each method in terms of the arithmetic average and coefficient of variation COV of $Q_c/Q_m$. The predictive performance was assessed for the overall database, as well as a subset of 17 DCIS piles which excluded the expanded-base Franki piles (different method of installation) and a further subset of 15 DCIS piles in which the test results at Kallo (significant clay layer affecting shaft capacity), Shotton (unrepresentative CPT profile) and Ryton-on-Dunsmore R3 (excessively large base resistance) were not considered. Graphical representations of the mean and COV of $Q_c/Q_m$ of each method are illustrated in Figure 7.15 and Figure 7.16 respectively.

The following observations of predictive performance of the CPT methods were made:

1. The mean value of $Q_c/Q_m$ varied widely with CPT method. The LCPC-82-1B method gave the most conservative estimate, with $Q_c/Q_m = 0.71$, while the Fugro-05 method estimate of DCIS total capacity was the least conservative ($Q_c/Q_m = 1.22$).
2. The NGI-05 method provided the best estimate of capacity, with a mean value of $Q_c/Q_m = 1.04$ and a COV = 34 %. The ICP-05 and UWA-05 methods had mean $Q_c/Q_m$ values of 0.81 and 0.87 respectively, which are slightly conservative, although the COVs (= 31 %) were slightly greater in comparison with the similar study of predictive performance for preformed displacement piles using advanced CPT methods by Schneider et al. (2008).
3. The Van Impe-86 method had a COV of 41 % which indicates poor predictive performance.
4. Interestingly, improved estimates of DCIS capacity were obtained for the LCPC-82 method when the standard driven pile shaft coefficients (category 2A) were selected ($Q_c/Q_m = 0.86$) instead of the DCIS coefficients (category 1B) for calculating shaft resistance ($Q_c/Q_m = 0.71$).
5. Excluding the results of the Shotton S1, Ryton R3, Kallo and the Franki pile database led to improved estimates of mean $Q_c/Q_m$ and COV for the EF-97 and Van Impe-86 methods. However, the mean value of $Q_c/Q_m$ for the LCPC-82-1B method (i.e. DCIS coefficients) reduced from 0.71 to 0.66. The ICP-05 and UWA-05 methods provided improved predictions while the NGI-05 became unconservative ($Q_c/Q_m = 1.18$).
Table 7.5 - Statistical performance of CPT-based design methods in estimating DCIS pile total capacity

<table>
<thead>
<tr>
<th>Site; Pile ref</th>
<th>EF -97</th>
<th>Fugro -05</th>
<th>ICP -05</th>
<th>LCPC -82 -1B</th>
<th>LCPC -82 -2A</th>
<th>NGI -05</th>
<th>UWA -05</th>
<th>Van Impe -86</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pontarddulais; P1</td>
<td>1.15</td>
<td>0.91</td>
<td>0.54</td>
<td>0.48</td>
<td>0.71</td>
<td>0.50</td>
<td>0.55</td>
<td>0.60</td>
</tr>
<tr>
<td>Shotton; S1</td>
<td>0.70</td>
<td>0.97</td>
<td>0.76</td>
<td>0.58</td>
<td>0.68</td>
<td>0.65</td>
<td>0.70</td>
<td>0.64</td>
</tr>
<tr>
<td>Erith; E1</td>
<td>0.69</td>
<td>0.74</td>
<td>0.61</td>
<td>0.45</td>
<td>0.51</td>
<td>0.74</td>
<td>0.56</td>
<td>0.68</td>
</tr>
<tr>
<td>Erith; E3</td>
<td>0.94</td>
<td>1.25</td>
<td>1.08</td>
<td>0.82</td>
<td>0.91</td>
<td>1.05</td>
<td>0.97</td>
<td>1.22</td>
</tr>
<tr>
<td>Ryton; R1</td>
<td>1.17</td>
<td>1.31</td>
<td>0.99</td>
<td>0.60</td>
<td>0.80</td>
<td>1.00</td>
<td>1.02</td>
<td>0.99</td>
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<tr>
<td>Ryton; R2</td>
<td>1.22</td>
<td>1.49</td>
<td>1.18</td>
<td>0.73</td>
<td>0.96</td>
<td>1.25</td>
<td>1.12</td>
<td>1.25</td>
</tr>
<tr>
<td>Ryton; R3</td>
<td>0.82</td>
<td>0.89</td>
<td>0.64</td>
<td>0.38</td>
<td>0.51</td>
<td>0.65</td>
<td>0.71</td>
<td>0.62</td>
</tr>
<tr>
<td>Kallo; K1</td>
<td>0.56</td>
<td>1.53</td>
<td>0.84</td>
<td>1.07</td>
<td>1.10</td>
<td>1.17</td>
<td>1.05</td>
<td>0.50</td>
</tr>
<tr>
<td>Kallo; K2a</td>
<td>0.67</td>
<td>1.36</td>
<td>0.99</td>
<td>0.91</td>
<td>0.96</td>
<td>0.96</td>
<td>0.77</td>
<td>0.84</td>
</tr>
<tr>
<td>Kallo; K3</td>
<td>0.73</td>
<td>1.53</td>
<td>1.07</td>
<td>1.06</td>
<td>1.11</td>
<td>1.18</td>
<td>1.18</td>
<td>0.58</td>
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<tr>
<td>Kallo; K4a</td>
<td>0.48</td>
<td>1.31</td>
<td>0.79</td>
<td>0.92</td>
<td>0.95</td>
<td>1.02</td>
<td>1.20</td>
<td>0.41</td>
</tr>
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<td>Kallo; K5</td>
<td>1.00</td>
<td>1.43</td>
<td>1.11</td>
<td>0.88</td>
<td>0.97</td>
<td>1.04</td>
<td>1.02</td>
<td>0.89</td>
</tr>
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<td>Kallo; K6ab</td>
<td>1.01</td>
<td>0.88</td>
<td>0.66</td>
<td>0.48</td>
<td>0.58</td>
<td>0.76</td>
<td>0.65</td>
<td>0.85</td>
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<td>Kallo; K7</td>
<td>0.76</td>
<td>1.61</td>
<td>1.10</td>
<td>1.06</td>
<td>1.12</td>
<td>1.16</td>
<td>0.60</td>
<td>0.66</td>
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<td>Le Havre; A4</td>
<td>1.20</td>
<td>1.60</td>
<td>1.17</td>
<td>0.83</td>
<td>1.21</td>
<td>1.51</td>
<td>1.05</td>
<td>1.51</td>
</tr>
<tr>
<td>Le Havre; C1</td>
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<td>1.37</td>
<td>1.01</td>
<td>0.72</td>
<td>1.04</td>
<td>1.30</td>
<td>0.85</td>
<td>1.32</td>
</tr>
<tr>
<td>Ringsend; TP20</td>
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<td>0.73</td>
<td>0.54</td>
<td>0.41</td>
<td>0.59</td>
<td>0.71</td>
<td>1.23</td>
<td>0.55</td>
</tr>
<tr>
<td>Franki-Grundbau; FG1</td>
<td>0.74</td>
<td>0.69</td>
<td>0.61</td>
<td>0.43</td>
<td>0.61</td>
<td>1.20</td>
<td>1.13</td>
<td>0.50</td>
</tr>
<tr>
<td>Gwizdala; G1</td>
<td>0.99</td>
<td>1.43</td>
<td>1.13</td>
<td>1.02</td>
<td>1.33</td>
<td>1.96</td>
<td>1.07</td>
<td>0.78</td>
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<tr>
<td>Gdansk Port; GP1</td>
<td>0.76</td>
<td>1.28</td>
<td>0.79</td>
<td>0.68</td>
<td>0.87</td>
<td>1.24</td>
<td>1.16</td>
<td>1.07</td>
</tr>
<tr>
<td>Gdansk; GK1</td>
<td>0.78</td>
<td>1.02</td>
<td>0.93</td>
<td>0.72</td>
<td>0.99</td>
<td>1.66</td>
<td>0.97</td>
<td>1.07</td>
</tr>
<tr>
<td>Berlin; P1</td>
<td>1.22</td>
<td>2.23</td>
<td>0.72</td>
<td>0.99</td>
<td>1.19</td>
<td>1.20</td>
<td>0.87</td>
<td>0.95</td>
</tr>
<tr>
<td>Berlin; P5</td>
<td>0.36</td>
<td>0.58</td>
<td>0.19</td>
<td>0.23</td>
<td>0.29</td>
<td>0.35</td>
<td>0.22</td>
<td>0.24</td>
</tr>
<tr>
<td>Hamburg; H1</td>
<td>0.37</td>
<td>1.06</td>
<td>0.46</td>
<td>0.52</td>
<td>0.56</td>
<td>0.74</td>
<td>0.51</td>
<td>0.42</td>
</tr>
<tr>
<td>Ghent; P85</td>
<td>1.04</td>
<td>1.06</td>
<td>0.51</td>
<td>0.64</td>
<td>0.79</td>
<td>0.86</td>
<td>0.68</td>
<td>1.15</td>
</tr>
<tr>
<td>Ghent; P585</td>
<td>1.32</td>
<td>1.34</td>
<td>0.64</td>
<td>0.81</td>
<td>1.00</td>
<td>1.09</td>
<td>0.86</td>
<td>1.46</td>
</tr>
</tbody>
</table>

All sites
- No. of piles, n | 26 | 26 | 26 | 26 | 26 | 26 | 26 | 26 |
- Average $Q_c/Q_m$ | 0.85 | 1.22 | 0.81 | 0.71 | 0.86 | 1.04 | 0.87 | 0.84 |
- COV | 0.33 | 0.30 | 0.32 | 0.34 | 0.30 | 0.34 | 0.30 | 0.41 |

Excluding Franki piles
- No. of piles, n | 17 | 17 | 17 | 17 | 17 | 17 | 17 | 17 |
- Average $Q_c/Q_m$ | 0.91 | 1.15 | 0.87 | 0.66 | 0.85 | 1.08 | 0.90 | 0.89 |
- COV | 0.24 | 0.27 | 0.27 | 0.32 | 0.29 | 0.36 | 0.26 | 0.34 |

Excluding Shotton S1, Ryton R3, Franki piles and Kallo
- No. of piles, n | 15 | 15 | 15 | 15 | 15 | 15 | 15 | 15 |
- Average $Q_c/Q_m$ | 0.93 | 1.15 | 0.88 | 0.66 | 0.88 | 1.18 | 0.90 | 0.96 |
- COV | 0.26 | 0.29 | 0.27 | 0.32 | 0.29 | 0.30 | 0.26 | 0.34 |

While the results in Table 7.5 provide the practitioner with guidance in selecting an appropriate CPT-based design method for estimating DCIS total pile capacity in sand, they do not imply that the shaft and base capacities have been estimated correctly as compensating errors are possible. The performance of the seven CPT methods in predicting the shaft and base capacity of DCIS piles separately is therefore examined in the following sections using those DCIS piles in Table 7.3 and Table 7.4 that were instrumented.
Figure 7.15 - Mean total capacity ratio for CPT methods

Figure 7.16 - Coefficient of variation in mean total capacity ratio for CPT methods
7.5.5 Shaft capacity

The database for assessing the performance of the CPT-based methods in estimating the shaft resistance of DCIS piles was hampered by the dearth of instrumented tests in which the shaft resistance was measured with an acceptable degree of accuracy. The overall pile database was therefore reduced to a total of 13 piles (including the six instrumented piles from Chapters 5 and 6) which in turn was subdivided into 9 DCIS piles and 4 Franki piles. The inclusion of the DCIS pile tests at Kallo by De Beer et al. (1979) in the database is somewhat questionable given the relatively large clay layer along the shaft of the test piles. However, the base resistance was considered to be interpreted to a high degree of accuracy by Chow (1997), assuming a minimal transfer of load in the soft clay layer (see De Beer et al. 1979 for more details regarding the site conditions).

The statistical performance of the seven CPT methods is summarised in Table 7.6, with the mean and COV of mean shaft capacity ratio \( Q_{s,c}/Q_{s,m} \) illustrated in Figure 7.17 and Figure 7.18 respectively. The following is noteworthy:

- The LCPC-82-2A, ICP-05 and Van Impe-86 methods provide the best estimates of shaft capacity, with mean shaft capacity ratios \( Q_{s,c}/Q_{s,m} \) of 0.96, 0.98 and 1.04 respectively. However, the coefficients of variation (COV) for these methods range from 26% to 41%, indicating considerable variability.

- The Fugro-05 method is highly unconservative, with a mean \( Q_{s,c}/Q_{s,m} = 1.62 \) and COV = 49%. Over-predictions were also evident for the NGI-05 and UWA-05 methods, albeit somewhat less (\( Q_{s,c}/Q_{s,m} \approx 1.27 – 1.48 \)) in comparison to the Fugro-05 method. The use of the DCIS shaft coefficients (i.e. category 1B) for the LCPC-82 method results in considerable under-prediction of shaft capacity, with \( Q_{s,c}/Q_{s,m} = 0.60 \).

- Excluding the Franki piles (due to differences in construction technique) results in an improvement in the mean \( Q_{s,c}/Q_{s,m} \) from 1.27 to 1.05 for the NGI-05 method, while the Fugro-05 method remains unconservative with a mean \( Q_{s,c}/Q_{s,m} \) of 1.32. The COV of the LCPC-82-2A is superior (≈ 23%) in comparison to the other methods.

- The exclusion of Pile S1 at Shotton (due to a potentially unrepresentative CPT \( q_c \) profile) and Kallo piles (due to the soft clay layers) resulted in a significant improvement in prediction performance for the UWA-05 method, resulting in a mean \( Q_{s,c}/Q_{s,m} \) of 0.94, with a COV = 36%.
<table>
<thead>
<tr>
<th>Site; Pile Ref</th>
<th>Measured capacity $Q_{s,m}$ (kN)</th>
<th>Predicted capacity $Q_{s,c}$ (kN)</th>
<th>$Q_{s,c}/Q_{s,m}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>EF -97</td>
<td>Fugro -05</td>
<td>ICP -05</td>
</tr>
<tr>
<td>Pontarddulais; P1</td>
<td>784</td>
<td>752</td>
<td>294</td>
</tr>
<tr>
<td>Dagenham; D1</td>
<td>758</td>
<td>374</td>
<td>507</td>
</tr>
<tr>
<td>Shotton; S1</td>
<td>509</td>
<td>331</td>
<td>659</td>
</tr>
<tr>
<td>Ryton; R1</td>
<td>961</td>
<td>381</td>
<td>971</td>
</tr>
<tr>
<td>Ryton; R2</td>
<td>915</td>
<td>445</td>
<td>1091</td>
</tr>
<tr>
<td>Ryton; R3</td>
<td>590</td>
<td>343</td>
<td>896</td>
</tr>
<tr>
<td>Kallo; K1</td>
<td>296</td>
<td>601</td>
<td>895</td>
</tr>
<tr>
<td>Kallo; K2</td>
<td>258</td>
<td>395</td>
<td>570</td>
</tr>
<tr>
<td>Kallo; K3</td>
<td>315</td>
<td>626</td>
<td>751</td>
</tr>
<tr>
<td>Kallo; K4</td>
<td>1160</td>
<td>464</td>
<td>621</td>
</tr>
<tr>
<td>Kallo; K5</td>
<td>276</td>
<td>486</td>
<td>649</td>
</tr>
<tr>
<td>Kallo; K7</td>
<td>305</td>
<td>548</td>
<td>751</td>
</tr>
<tr>
<td>Le Havre; C1</td>
<td>1202</td>
<td>942</td>
<td>1197</td>
</tr>
</tbody>
</table>

All instrumented sites

No. of piles, $n$: 13 13 13 13 13 13 13 13 13
Average $Q_{s,c}/Q_{s,m}$: 1.13 1.62 0.98 0.60 0.96 1.27 1.48 1.04
COV: 0.55 0.49 0.32 0.39 0.26 0.46 0.43 0.41

Excluding Franki piles

No. of piles, $n$: 9 9 9 9 9 9 9 9 9
Average $Q_{s,c}/Q_{s,m}$: 0.88 1.32 0.90 0.51 0.88 1.05 1.22 0.87
COV: 0.61 0.53 0.34 0.38 0.23 0.50 0.48 0.39

Excluding Shotton S1 & Kallo piles

No. of piles, $n$: 6 6 6 6 6 6 6 6 6
Average $Q_{s,c}/Q_{s,m}$: 0.62 0.96 0.82 0.41 0.81 0.83 0.94 0.69
COV: 0.35 0.42 0.39 0.20 0.22 0.44 0.36 0.37

218
Figure 7.17 - Mean shaft capacity ratio for CPT methods

Figure 7.18 - Coefficient of variation in mean shaft capacity ratio for CPT methods
7.5.6 Base capacity
As discussed in Section 7.4.3, the data in Figure 7.12 provides conclusive evidence that the base resistance of DCIS piles is essentially identical to traditional preformed closed-ended displacement piles in sand. The predictive performance for base capacity was assessed using a total of 12 piles from Table 7.3 and Table 7.4 which were either instrumented (i.e. the instrumented tests in Chapters 5 and 6) or estimated with a high degree of accuracy (i.e. Kallo). Additional assessments were performed using the DCIS piles only \((n = 8)\) and a further subset in which the test results of Pile S1 at Shotton and Pile R3 at Ryton-on-Dunsmore were excluded for reasons outlined previously.

Table 7.2 summarises the statistical performance of each method in terms of mean \(Q_{b,c}/Q_{b,m}\) and COV. Points to note include:

- With the exception of the Fugro-05 method, all methods yield mean \(Q_{b,c}/Q_{b,m}\) values of between 0.95 and 1.05. However, the EF-97 and Van Impe-86 methods have COVs in excess of 70 % which indicates extremely poor predictive performance. The Fugro-05 method is once again over-conservative with \(Q_{b,c}/Q_{b,m} = 1.58\) and COV = 45 %.
- Exclusion of the Franki piles results in mean \(Q_c/Q_m\) values for the EF-97 and Van Impe-86 methods becoming unconservative. On the other hand, the recent CPT methods (Fugro-05, ICP-05, NGI-05 and UWA-05) remain relatively unchanged.
- The COVs of the ICP-05, NGI-05 and UWA-05 reduce dramatically from \(\approx 30\%\) to \(\approx 15\%\) when the results of Shotton Pile S1 and Ryton-on-Dunsmore Pile R3 are also excluded from the statistical analysis. In contrast, the mean \(Q_{b,c}/Q_{b,m}\) values for the EF-97, Fugro-05 and Van Impe-86 increase above 1.40, with the Fugro-05 method predicting, on average, twice the measured base resistance. The COVs for these methods are also greater than 40%, highlighting their poor reliability.
- The mean \(Q_{b,c}/Q_{b,m}\) ratios of the methods in Table 7.6 are in close agreement with the results of a similar study of predictive performance for closed-ended displacement piles by Xu et al. (2008).
Table 7.7 - Statistical performance of CPT-based design methods in estimating DCIS pile base capacity

<table>
<thead>
<tr>
<th>Site; Pile ref</th>
<th>Measured capacity $Q_{b,0.1D_b}$ (kN)</th>
<th>Predicted capacity $Q_{b,c}$ (kN)</th>
<th>$Q_{b,c}/Q_{b,m}$</th>
<th>EF</th>
<th>Fugro</th>
<th>ICP</th>
<th>LCPC</th>
<th>NGI</th>
<th>UWA</th>
<th>Van</th>
<th>Impe</th>
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<tbody>
<tr>
<td>Pontarddulais; P1</td>
<td>163</td>
<td>335 574 214 201 198 164 415</td>
<td>2.05 3.52 1.31 1.23 1.21 1.00 2.54</td>
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<td></td>
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</tr>
<tr>
<td>Shotton; S1</td>
<td>1769</td>
<td>1515 1483 1284 1052 1005 995 557</td>
<td>0.86 0.84 0.73 0.59 0.57 0.56 0.32</td>
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</tr>
<tr>
<td>Ryton; R1</td>
<td>984</td>
<td>1351 1160 801 656 891 754 954</td>
<td>2.14 1.63 1.13 0.92 1.25 1.06 1.34</td>
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<tr>
<td>Ryton; R2</td>
<td>710</td>
<td>1519 1259 944 774 1087 742 1239</td>
<td>2.18 1.85 1.38 1.13 1.59 1.09 1.82</td>
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<tr>
<td>Ryton; R3</td>
<td>682</td>
<td>1487 1123 701 574 825 767 801</td>
<td>1.15 0.85 0.53 0.44 0.63 0.58 0.61</td>
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<tr>
<td>Kallo; K1</td>
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<td>2030 8461 4591 6121 6403 6199 2273</td>
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<tr>
<td>Kallo; K2*</td>
<td>5802</td>
<td>936 3007 2261 2193 2023 2357 1788</td>
<td>0.38 1.23 0.93 0.90 0.83 0.97 0.73</td>
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<td></td>
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</tr>
<tr>
<td>Kallo; K3</td>
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<td>1236 4049 2931 3055 3057 3056 1447</td>
<td>0.43 1.40 1.01 1.06 1.06 1.06 0.50</td>
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</tr>
<tr>
<td>Kallo; K4*</td>
<td>2890</td>
<td>1774 6989 4180 5184 5368 5133 1767</td>
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<td></td>
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<td></td>
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<td></td>
</tr>
<tr>
<td>Kallo; K5</td>
<td>4810</td>
<td>600 1642 1367 1153 1108 1142 1145</td>
<td>0.43 1.18 0.98 0.83 0.80 0.82 0.82</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Kallo; K7</td>
<td>1390</td>
<td>1041 3734 2556 2650 2589 2680 2491</td>
<td>0.42 1.50 1.03 1.06 1.04 1.08 1.00</td>
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<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Le Havre; C1</td>
<td>665</td>
<td>1020 1381 839 727 901 698 610</td>
<td>1.53 2.08 1.26 1.09 1.35 1.05 0.92</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

All sites
No. of piles, $n$ 12 12 12 12 12 12 12
Average $Q_{b,c}/Q_{b,m}$ 1.02 1.58 1.00 0.95 1.05 0.95 0.95
COV 0.74 0.45 0.25 0.24 0.29 0.20 0.70

Excluding Franki piles
No. of piles, $n$ 8 8 8 8 8 8 8
Average $Q_{b,c}/Q_{b,m}$ 1.35 1.68 1.04 0.91 1.06 0.91 1.17
COV 0.55 0.51 0.28 0.30 0.35 0.25 0.61

Excluding Shotton S1, Ryton R3 & Franki piles
No. of piles, $n$ 6 6 6 6 6 6 6
Average $Q_{b,c}/Q_{b,m}$ 1.46 1.96 1.18 1.05 1.21 1.02 1.41
COV 0.57 0.42 0.14 0.14 0.22 0.10 0.47
Figure 7.19 - Mean base capacity ratio for CPT methods

Figure 7.20 - Coefficient of variation in mean base capacity ratio for CPT methods
7.5.7 Summary

The predictive performance of seven CPT-based displacement pile design methods for estimating the shaft, base and total capacity of DCIS piles in sand was assessed using a database of 26 DCIS piles. While the NGI-05 method provided the best overall estimate of DCIS pile total capacity, the shaft capacity prediction for this method exhibited a significant degree of variability (≈ 50 %) and should therefore be used with caution if a significant portion of the total capacity of a DCIS pile is to be derived from shaft resistance. On the other hand, the UWA-05 method provided the best estimate of both base resistance and shaft resistance (when the results of the tests at Kallo are excluded). Poor predictive performance was displayed by the EF-97, Fugro-05 and Van Impe-86 methods and their use for estimating DCIS pile capacity should therefore be discouraged. While the LCPC-82 method provided reasonable estimates of DCIS total pile capacity when the standard shaft coefficients for driven piles are used in place of the specified DCIS coefficients, such a result is a direct consequence of compensating errors as this method cannot account for friction fatigue through the use of a single shaft coefficient (as shown previously in Section 7.4.2.2); its use for DCIS pile design is therefore discouraged.
7.6 LOAD-DISPLACEMENT BEHAVIOUR

7.6.1 Introduction
The results of the instrumented DCIS pile load tests in Chapters 5 and 6, together with the analysis of current CPT-based design methods in Section 7.5, have demonstrated that the shaft, base and total resistances of DCIS piles in sand are comparable to traditional preformed displacement piles. However, given that peak local shear stresses were generally mobilised at considerably larger shaft displacements ($w_s/D_s > 5\%$), it is worthwhile examining the load-displacement behaviour of DCIS piles in more detail. To this end, a database of 105 load tests on DCIS piles in sand has been compiled which is subsequently used in conjunction with a database of 30 high-quality load tests on preformed displacement piles to assess differences in load-displacement behaviour using the simple statistical simulation method proposed by Phoon et al. (2006) and Phoon (2008).

7.6.2 Pile test databases
In order to assess the load-displacement curve characteristics of both DCIS piles and preformed driven displacement piles, it was necessary to compile a series of load tests for each pile type into corresponding pile test databases. The following sections provide details of each database.

7.6.2.1 DCIS pile database
Table 7.8 presents a database of 90 maintained compression load tests (including the seven instrumented DCIS test piles described in Chapters 5 and 6) performed on temporary-cased DCIS piles at various sites in the United Kingdom which were compiled by the author from Keller Foundations files. The ground conditions at each site were typically layered in a similar manner to the instrumented DCIS pile test sites in Chapter 5, with the bases of all DCIS piles founded in granular material. The load tests for the majority of the piles in Table 7.8 were conducted in accordance with the Institution of Civil Engineers Specification for Piling and Embedded Retaining Walls or SPERW (ICE, 2007), the details of which were previously presented in Section 3.6.3, after curing periods of between 9 days and 33 days. The piles were typically tested to maximum loads corresponding to 150\% of the Design Verification Load (DVL) with an unload/reload cycle performed at 100\% DVL. The measured load-displacement curves for the 90 DCIS piles in the database are presented in Appendix D.
In addition to the Keller Foundations piles, the database also includes 15 compression load tests on temporary-cased DCIS piles in sand reported in the literature and details of these piles are summarised in Table 7.9.

### 7.6.2.2 Driven preformed displacement pile database

To enable a comparison of the DCIS test pile curves with preformed driven displacement piles, a database of compression load tests on 16 precast concrete piles and 14 closed-ended steel piles reported in the literature was also compiled; details of these piles are summarised in Table 7.10 and Table 7.11 respectively. The majority of piles form part of the displacement pile database compiled by researchers at the University of Western Australia during the development of the UWA-05 CPT-based design method (see Section 2.5.3 for more details) and had normalised displacements $w/D_b$ of ≈ 10 % or greater, enabling direct assessment of the total pile capacity $Q_{t,0.1D_b}$.

### 7.6.3 Interpretation of capacity

As discussed previously in Section 2.2 of Chapter 2, the total pile capacity is defined in this thesis as the applied head load corresponding to a head displacement equivalent to 10 % of the pile base diameter $D_b$. However, given that the majority of the piles in the DCIS database were typically subjected to proof load tests (whereby the maximum applied load was generally equivalent to 1.5 times the design verification load of the pile), the resulting pile displacement was generally insufficient to enable direct measurement of the total capacity $Q_{t,0.1D_b}$. However, a total of 37 piles were sufficiently displaced to enable the slope tangent capacity $Q_{st}$ to be measured which, as illustrated in Figure 7.21, is defined as the load corresponding to a displacement equal to the initial slope (elastic line) of the load-displacement curve plus the sum of 3.8 mm and $D_b/120$ (Chen and Fang, 2009).

In a similar manner to the assessment of pile capacity from non-failure load tests by Paikowsky and Tolosko (1999) and Dithinde et al. (2011), the load-displacement curves of the remaining piles in each database for which $Q_{st}$ was also not directly measured were hyperbolically extrapolated to the offset line in Figure 7.21 using Equation 7.7 (Chin, 1972).
\[ Q = \frac{w}{a' + b'w} \]  

where \( Q \) is the applied load, \( w \) is the pile head displacement and \( a' \) and \( b' \) are hyperbolic curve-fitting parameters for the load-displacement curve.

The level of extrapolation required for each pile was classified as follows:

- Category A - slope tangent capacity measured and no extrapolation necessary
- Category B - extrapolation necessary in the region between the elastic line and the offset line
- Category C - extrapolation necessary from the elastic line

Based on this classification system, a total of 37 DCIS piles in Table 7.8 and Table 7.9 were classified in Category A, 60 piles in Category B and 8 piles in Category C, with all piles in the driven displacement pile database in Table 7.10 and Table 7.11 classified as Category A.

Figure 7.21 - Slope tangent capacity with hyperbolic extrapolation categories

A summary of the minimum, maximum, average, standard deviation and coefficient of variation of the pile dimensions and load tests results for the DCIS and driven pile databases is given in Table 7.12.
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<th>Slenderness ratio $L/D_b$</th>
<th>Age (days)</th>
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<th>Maximum Settlement $w$ (mm)</th>
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<td>I6</td>
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<td>I7</td>
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<td>GP1</td>
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<td>620</td>
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<td>Gwizdala &amp; Krasinski (2013)</td>
<td>GK1</td>
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<td>2.05</td>
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| Gwizdala      | Gwizdala & Krasinski (2013) | G1    | 508                       | 560                      | 21.90         | 39.11                     | 3642                          | 5.29                              | 0.94                                | 5063                              | B        | 230
Table 7.10 - Closed-ended precast concrete displacement pile database

<table>
<thead>
<tr>
<th>Site</th>
<th>Reference</th>
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<th>Pile Ref</th>
<th>Shaft diameter $D_s$ (mm)</th>
<th>Base diameter $D_b$ (mm)</th>
<th>Length $L$ (m)</th>
<th>Slenderness ratio $L/D_b$</th>
<th>Maximum Test Load (kN)</th>
<th>Maximum Settlement $w$ (mm)</th>
<th>Normalised Settlement $w/D_b$ (%)</th>
<th>Slope-tangent capacity $Q_{st}$ (kN)</th>
<th>Category</th>
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<td>Baghdad</td>
<td>Altaee et al. (1992)</td>
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<td>Nevels &amp; Snethen (1994)</td>
<td>Sand</td>
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<td>610</td>
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<td>3528</td>
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<td>11.39</td>
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<td>Sand</td>
<td>E</td>
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<td>280</td>
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<td>14.36</td>
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<td>Fittja Straits</td>
<td>Axelsson (2000)</td>
<td>Sand</td>
<td>D</td>
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<td>235</td>
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<td>15.02</td>
<td>306</td>
<td>A</td>
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<td>Vesic (1970)</td>
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<td>H2</td>
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<td>406</td>
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<td>2497</td>
<td>A</td>
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<tr>
<td>Tickfaw River</td>
<td>Titi &amp; Farsakh (1999)</td>
<td>Sand</td>
<td>TP1</td>
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<td>610</td>
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<td>4141</td>
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</table>
Table 7.11 - Closed-ended steel displacement pile database

| Site            | Reference                              | Soil type | Pile Ref | Shaft diameter $D_s$ (mm) | Base diameter $D_b$ (mm) | Length $L$ (m) | Slenderness ratio $L/D_b$ | Maximum Test Load (kN) | Maximum Settlement $w$ (mm) | Normalised Settlement $w/D_b$ (%) | Slope-tangent capacity $Q_{st}$ (kN) | Category |
|-----------------|----------------------------------------|-----------|----------|---------------------------|--------------------------|---------------|---------------------------|--------------------------|----------------------------------|----------------------------------|----------|
| Cimarron River  | Nevels & Sneathen (1994)               | Sand      | P1       | 660                       | 660                      | 19.00         | 28.79                     | 3584                     | 80.29                            | 12.17                            | 2622     | A        |
| Hoogzand        | Beringen et al. (1979)                 | Sand      | II       | 356                       | 356                      | 6.80          | 19.10                     | 3119                     | 63.41                            | 17.81                            | 2666     | A        |
| Hsin Ta         | Yen et al. (1989)                      | Sand      | TP4      | 609                       | 609                      | 34.30         | 56.32                     | 4260                     | 79.20                            | 13.00                            | 3671     | A        |
| Hsin Ta         | Yen et al. (1989)                      | Sand      | TP6      | 609                       | 609                      | 34.30         | 25.32                     | 4445                     | 21.65                            | 3.56                             | 3378     | A        |
| Hunter’s Point  | Briaud et al. (1989b)                  | Sand      | S        | 273                       | 273                      | 7.80          | 28.57                     | 489                      | 38.04                            | 13.93                            | 411      | A        |
| Ogeechee River  | Vesic (1970)                           | Sand      | H11      | 457                       | 457                      | 3.00          | 6.56                      | 924                      | 126.90                           | 27.77                            | 466      | A        |
| Ogeechee River  | Vesic (1970)                           | Sand      | H14      | 457                       | 457                      | 12.00         | 26.26                     | 3531                     | 131.91                           | 28.86                            | 2707     | A        |
| Ogeechee River  | Vesic (1970)                           | Sand      | H15      | 457                       | 457                      | 15.00         | 32.82                     | 3806                     | 60.43                            | 13.22                            | 3128     | A        |
| Pigeon Creek    | Paik et al. (2003)                     | Sand      | 1        | 356                       | 356                      | 6.90          | 19.38                     | 1761                     | 151.81                           | 42.64                            | 1218     | A        |
| Lock & Dam 26   | Briaud et al. (1989a)                  | Sand      | 3-1      | 305                       | 305                      | 14.20         | 46.56                     | 1302                     | 76.32                            | 25.02                            | 921      | A        |
| Lock & Dam 26   | Briaud et al. (1989a)                  | Sand      | 3-4      | 356                       | 356                      | 14.40         | 40.45                     | 1114                     | 32.39                            | 9.10                             | 855      | A        |
| Lock & Dam 26   | Briaud et al. (1989a)                  | Sand      | 3-7      | 406                       | 406                      | 14.60         | 35.96                     | 1800                     | 74.34                            | 18.31                            | 1162     | A        |
Table 7.12 - Summary statistics for pile test databases

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<tr>
<th></th>
<th>Shaft diameter</th>
<th>Base diameter</th>
<th>Length</th>
<th>Slenderness ratio</th>
<th>Normalised Settlement</th>
<th>Slope-tangent capacity</th>
<th>Capacity ratio</th>
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<tr>
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<td>$D_s$ (mm)</td>
<td>$D_b$ (mm)</td>
<td>$L$ (m)</td>
<td>$L/D_b$ (%)</td>
<td>$w/D_b$ (%)</td>
<td>$Q_{st}$ (kN)</td>
<td>$Q_{st}/Q_{t,0.1Db}^*$</td>
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<td>285</td>
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<td>0.72</td>
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<td>5.50</td>
<td>14.47</td>
<td>2.05</td>
<td>448</td>
<td>0.36</td>
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<td>645</td>
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<td>86.37</td>
<td>10.67</td>
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7.6.4 Comparison of measured load-displacement curves for DCIS and driven piles

Prior to performing the statistical simulation of the load-displacement curves, the Category A DCIS piles in Table 7.8 and Table 7.9 were normalised by their corresponding total capacity $Q_{t,0.1Db}$ (either measured or hyperbolically extrapolated from $Q_{st}$) and plotted against normalised pile head displacement $w/D_b$ in Figure 7.22(a), while Figure 7.22(b) shows the equivalent normalised load-displacement curves for the driven pile database in Table 7.10 and Table 7.11. An overall comparison of the measured DCIS ($n = 37$) and preformed displacement pile curves ($n = 30$) is illustrated in Figure 7.22(c). Visual inspection of Figure 7.22(a), (b) and (c) suggests that preformed driven piles exhibit a steeper normalised load-displacement response during the initial stages of loading, mobilising ≈ 80 % of the total capacity at failure $Q_{t,0.1Db}$ at normalised displacements $w/D_b$ of ≈ 2.5 %, with the DCIS piles generally mobilising capacity at a slower rate in comparison ($w/D_b$ ≈ 4 %). Furthermore, the mean capacity ratio $Q_{st}/Q_{t,0.1Db}$ for DCIS piles is ≈ 0.63 in comparison to a mean $Q_{st}/Q_{t,0.1Db}$ of ≈ 83 % for the driven displacement piles (see Table 7.12 for more details). Based on these observations, it is likely that the load-displacement behaviour of DCIS piles is somewhat different to a preformed pile of equivalent dimensions.
Figure 7.22 - Normalised load-displacement curves for (a) DCIS piles Category A (b) preformed driven displacement piles and (c) overall comparison
7.6.5 Load-displacement curve simulation

Given the uncertainties associated with the excessive level of extrapolation necessary to reach $Q_{t,0.1Db}$ for the load-displacement curves of the piles in Categories B and C, it was decided to use the statistical method by Phoon et al. (2006) to simulate normalised load-displacement curves for DCIS and preformed driven displacement piles. This method involves normalising the measured load-displacement curves by the corresponding value of $Q_{st}$ and fitting the hyperbolic parameters coefficients $a = a'Q_{st}$ and $b = b'Q_{st}$ to the resulting response using Equation 7.8.

$$\frac{Q}{Q_{st}} = \frac{w}{(a'Q_{st}) + (b'Q_{st})w} = \frac{w}{a + bw} \quad 7.8$$

Figure 7.23 shows the resulting hyperbolic parameters $a$ and $b$ obtained for the 37 Category A DCIS piles and the 30 displacement piles using Equation 7.8; strong negative correlation is evident for both pile types which is expressed using the Pearson correlation coefficient $\rho_{ab}$ as follows:

$$\rho_{ab} = \frac{\sum_{i=1}^{n} (a_i - \bar{a})(b_i - \bar{b})}{\sqrt{\sum_{i=1}^{n} (a_i - \bar{a})^2 \sum_{i=1}^{n} (b_i - \bar{b})^2}} \quad 7.9$$

where $a_i$ and $b_i$ are the individual pairs of hyperbolic parameters and $\bar{a}$ and $\bar{b}$ are the sample means. Pearson coefficients of -0.740, -0.668 and -0.733 were obtained for the Category A DCIS piles ($n = 37$), the DCIS overall database ($n = 105$) and the driven pile database ($n = 30$) respectively.

It is also apparent in Figure 7.23(a) that the values of $a$ and $b$ for the DCIS piles tend to be clustered around the centre of the graph, while the data for the driven piles tend to be more concentrated in the bottom-right region. Bearing in mind that the measured capacity ratio $Q_{st}/Q_{t,0.1Db}$ was $\approx 20\%$ greater for the driven piles in comparison to the DCIS piles, the data in Figure 7.23 suggests that the hyperbolic curve-fitting parameters for DCIS piles differ somewhat from traditional preformed displacement piles. The relationship between $a$ and $b$ for the overall DCIS database of 105 piles is shown in Figure 7.23(b) where considerably greater scatter is apparent in comparison to Figure 7.23(a), due to the level of extrapolation required, as well as the effects of creep displacements during load holds and loading cycles.
The statistical software package Minitab was used to assess the probability distributions of \( a \) and \( b \) for the Category A DCIS piles, the overall DCIS pile database and the preformed driven displacement piles using histograms and lognormal probability plots which are illustrated in Figure 7.24 and Figure 7.25 respectively. It is evident that the \( a \) parameter follows a lognormal distribution for the two pile types (including the two DCIS subsets), with P-values obtained using the Anderson-Darling goodness-of-fit test greater than the specified 5 % level of significance. On the other hand, the \( b \) parameters for all scenarios are normally-distributed with Anderson-Darling P-values < 0.05 requiring the rejection of the null-hypothesis of lognormality. A similar outcome was observed for bored piles in cohesionless soils by Dithinde et al. (2011) who proceeded to assume a lognormal distribution of \( b \) for practical purposes during the subsequent load-displacement curve simulation. The assumption of a log-normal distribution was therefore adopted for the \( b \) parameters in Figure 7.24 and Figure 7.25.
Figure 7.24 - Histograms of hyperbolic parameters for (a) DCIS piles Cat. A (b) DCIS piles overall and (c) driven piles
Figure 7.25 - Lognormal distribution probability plots for (a) DCIS piles Cat. A (b) DCIS piles overall and (c) driven piles
7.6.5.1 Simulation procedure

A total of 10,000 load-displacement curves were simulated for DCIS piles and driven displacement piles using the following procedure described by Phoon et al. (2006) and Dithinde et al. (2011):

1. The mean $m$ and standard deviation $s$ of the hyperbolic parameters $a$ and $b$ (given in Figure 7.24a and b) are used to calculate their equivalent lognormal mean $\lambda$ and standard deviation $\xi$ using the following expressions:

   $$\lambda = \ln m - 0.5\xi^2$$

   $$\xi = \sqrt{\ln\left[1 + (s/m)^2\right]}$$  \hspace{1cm} 7.10

2. A dataset of $n = 10,000$ uncorrelated standard normal random variables $Z_a$ and $Z_b$ are simulated using the Random Number Generation function in Microsoft Excel.

3. The values of $Z_a$ and $Z_b$ are transformed into random variables $X_a$ and $X_b$ respectively using the expressions:

   $$X_a = Z_b$$

   $$X_b = Z_a\rho_{X_aX_b} + Z_b\sqrt{1 - \rho_{X_aX_b}^2}$$  \hspace{1cm} 7.11

where $\rho_{X_aX_b}$ is the Pearson correlation coefficient for $X_a$ and $X_b$ which is back-figured using the expression:

$$\rho_{ab} = \frac{\exp\left(\xi_a\xi_b\rho_{X_aX_b}\right) - 1}{\sqrt{\exp\left(\xi_a^2\right) - 1}\sqrt{\exp\left(\xi_b^2\right) - 1}}$$  \hspace{1cm} 7.12

4. The random variables $X_a$ and $X_b$ are transformed into equivalent lognormal hyperbolic parameters $a$ and $b$ by:

   $$a = \exp(\xi_a X_a + \lambda_a)$$

   $$b = \exp(\xi_b X_b + \lambda_b)$$  \hspace{1cm} 7.13

5. The normalised load-displacement curves are generated using Equation 7.8.

The above simulation procedure was performed for the DCIS Category A piles, the overall DCIS database and the driven pile database, the results of which are presented below.
7.6.6 Simulation results

Figure 7.26(a) illustrates a comparison of the average curves for DCIS piles and the driven preformed displacement piles obtained using $n = 10,000$ simulations, while the variability of the curves within one standard deviation above and below the average curve for the two pile types is shown in Figure 7.26(b). It is apparent that:

- In line with the observations of measured pile behaviour in Figure 7.22, a stiffer average normalised load-displacement response is evident during the initial stages of loading of the driven piles in comparison to the simulated DCIS pile response.
- The slope tangent capacity $Q_{st}$ is mobilised at a normalised displacement $w_b/D_b$ of $\approx 3\%$ for both pile types.
- The reduced variability in driven piles curve in comparison to the DCIS curve is attributed to the minimal level of extrapolation (and hence uncertainty) required for the piles in Table 7.10 and Table 7.11.
Figure 7.27 shows a comparison of the simulated curves obtained using the data from the DCIS Category A piles with the overall DCIS pile database; the curves are in good agreement during the initial stages of loading up to $w_b/D_b \approx 2\%$, although, slight discrepancies are evident at greater normalised displacements.

Finally, the influence of the number of simulations $n$ on the resulting load-displacement curves was assessed by performing additional simulations using $n = 100$ and $n = 1,000$. The resulting curves are compared with $n = 10,000$ in Figure 7.28 where differences in the simulated load-displacement response exist for $n = 100$ and $n = 1,000$, particular for $w_b/D_b > 4\%$. However, identical responses were evident for $n = 1,000$ and $n = 10,000$. 

Figure 7.28 - Influence of simulation size on normalised load-displacement curves
7.6.7 Implications for DCIS pile design

Despite having similar shaft, base and total capacities, the total resistance of DCIS piles tends to mobilise at a slower rate in comparison to preformed displacement piles which is attributed to the larger displacements required to mobilise peak local shear stresses $\tau_{sf}$ for DCIS piles due to their increased shaft roughness created by in-situ concreting. The main implication of this for DCIS pile design is that for a given applied load, DCIS piles may potentially displace further than a preformed driven pile due to partial mobilisation of the shaft resistance.
8.1 INTRODUCTION

Driven cast-in-situ (DCIS) piles are typically classified as large displacement piles, despite certain aspects of the construction process being relevant to replacement pile types due to the casting of the concrete in-situ and the resulting rough shaft interface. This thesis provides an investigation of the behaviour of seven DCIS piles during installation, curing and maintained compression load testing in layered soil and sand using the methodology illustrated in Figure 2.30. The DCIS test piles were instrumented with strain gauges in order to assess the development of residual loads during curing, as well the variation in shaft and base resistance during loading. Databases of DCIS piles were subsequently compiled to examine the predictive performance of seven CPT-based displacement pile design methods in estimating the shaft, base and total capacity of DCIS piles in sand and the load-settlement performance of DCIS piles in comparison to traditional preformed closed-ended displacement piles. This chapter summarises the key findings of the thesis and provides recommendations for future work.

8.2 RESEARCH FINDINGS

The key research findings are summarised in chronological order of pile history i.e. installation, curing and maintained compression load testing, followed by the assessment of the CPT-based methods in estimating DCIS pile capacity in sand and the load-displacement behaviour. Prior to doing so however, general conclusions regarding the experiences associated with instrumenting and interpreting strains in cast-in-situ piles are presented.

8.2.1 Instrumentation and strain interpretation of cast-in-situ piles

1. Installation of the steel reinforcement cage after pile construction and concreting proved very difficult at the layered soil sites, with significant damage to the instrumentation occurring near the base of the test pile at Erith.

2. The unload/reload cycles during early maintained compression load tests led to difficulties in interpreting the pile stiffness, primarily due to the development of plastic strains, as well as reductions in local shear stresses upon reloading due to cyclic shearing at the pile-soil interface. Such load cycling should therefore be avoided when conducting future load tests on instrumented DCIS piles for research purposes.
3. Significant creep strains developed within the concrete during load holds and must be removed during the subsequent strain interpretation process in order to derive the elastic stiffness of the pile $E_p$ for converting the measured elastic strains to loads.

### 8.2.2 Installation

The DCIS piling rigs were fitted with instrumentation which enabled the installation resistance $q_{cp}$ to be derived using the Danish pile driving formula. The relationship between installation resistance and base resistance predicted by three popular CPT-based design methods was subsequently investigated using the corresponding CPT $q_c$ profile adjacent to each test pile location. The following conclusions on DCIS pile behaviour during installation are summarised:

1. The installation resistance $q_{cp}$ shows excellent agreement with the base resistance $q_{b,0.1D_b}$ predicted by the UWA-05 method using the Dutch average cone resistance. The profile predicted by the ICP-05 method (using the arithmetic average over a distance of 1.5$D_b$ above and below the pile base) also compares favourably with $q_{cp}$ in uniform sand, but less so in layered soil. On the other hand, the base resistance profile determined by the EF-97 method (derived using the geometric average) over-predicts $q_{cp}$ significantly in both uniform sand and layered soil.

2. Analyses of the strain response during subsequent maintained compression load testing suggested that increases in shaft diameter occurred after casting of the instrumented DCIS piles within the soft clay/peat deposits at the layered sites in Chapter 5.

### 8.2.3 Curing

The variations in strain and temperature were continuously monitored within three instrumented DCIS test piles after casting in order to investigate the development of residual loads during curing. The results have shown the following:

1. The processes which occur within the concrete and soil after casting i.e. initial set and hydration, curing, and installation/consolidation effects can be distinguished clearly from strain and temperature measurements.

2. The initial set period (< 24 hours) after casting is characterised by increases in hydration temperatures and compressive strains throughout the pile due to the differences in the coefficient of thermal expansion $\alpha$ of the concrete and steel strain gauges.
3. The time required to reach peak hydration temperature $T_{peak}$ is a near-linear function of pile shaft diameter $D_s$ which is expressed using Equation 7.1.

4. The compressive strains subsequently reduce and become tensile after hydration temperatures peak and begin to stabilise during the curing phase; this tensile strain response phase is not a result of swelling of the concrete due to the absorption of additional moisture from the surrounding saturated soil as suggested in the literature, but a consequence of pile restraint during drying shrinkage.

5. The compressive strains which slowly developed within the lower sections of the instrumented piles $\approx$ 80 hours after casting in the layered soil were attributed to the formation of residual loads due to consolidation settlement of soft clay/peat layers. In contrast, negligible changes in strain were observed during the curing period of the instrumented test pile installed in uniform sand.

6. While the residual load profiles tended to vary considerably with interpretation method, the method proposed by Siegel and McGillivray (2009), i.e. Method 3, provides the most sensible distribution of residual load in layered soil.

7. Based on the results for Pile S1 at Shotton in Chapter 6, residual loads for DCIS piles in uniform sand are considered negligible and can therefore be ignored.

8. The quantity of residual load in a soft cohesive layer (applicable to DCIS piles) can be reasonably estimated using standard design equations for negative skin friction, e.g. $\tau_{s,res} = 0.3\sigma'_v$ (Meyerhof 1976).

### 8.2.4 Maintained compression loading

Maintained compression load tests were designed and performed on a total of 7 instrumented DCIS piles in layered and sand deposits to a minimum pile head displacement equivalent to 10 % of the pile base diameter $D_b$, with instrumentation enabling the shaft and base resistances to be accurately measured during loading. The following conclusions regarding DCIS pile behaviour have been reached:

1. Peak local shear stresses $\tau_{s,loc}$ tended to mobilise at shaft displacements of between 10 mm and 50 mm during loading.

2. The normalised average shear stresses $\tau_{s,avg}/q_c$ mobilised along the pile shafts in sand at Ryton-on-Dunsmore were greater than the typical range of between 0.008 and 0.01 for closed-ended steel displacement piles reported by Gavin and Lehane (2003).

3. A clear reduction in normalised local shear stress $\tau_{s,loc}/q_c$ and normalised radial effective stress at failure $\sigma'_{r0}/q_c$ with normalised distance from the pile base $h/D_s$ was
evident for the instrumented DCIS test piles. This phenomenon, known as friction fatigue, is a well-known characteristic of preformed displacement piles and its existence for DCIS piles implies that radial stresses induced during driven installation of the steel tube are not erased upon concreting of the pile/withdrawal of the tube. In line with studies of friction fatigue on preformed closed-ended displacement piles, the degradation in normalised local shear stress at failure \( \tau_{sf}/q_c \) for DCIS piles exhibits strong correlation with the number of cycles \( N \) experienced during both installation and subsequent load testing (if load cycles were performed). Equation 7.5 provides the best-fit relationship for the reduction in \( \sigma'_{if} \) with \( h/D_s \) for DCIS piles in sand.

4. Based on the close agreement between the measured normalised radial effective stress at failure \( \sigma'_{rd}/q_c \) and the normalised radial stress after installation and equalisation \( \sigma'_{rc}/q_c \) predicted by the UWA-05 method for preformed displacement piles, it may be conjectured that the dilation-related increase in radial stress during loading \( \Delta \sigma'_{rd} \) represents a small proportion of the overall radial effective stress at failure \( \sigma'_{if} \) for the instrumented DCIS test piles.

5. The shaft coefficients \( \alpha_s = q_c/\tau_{sf} \) for DCIS piles recommended by the LCPC-82 method tend to over-predict the measured values of \( \alpha_s \) significantly, with closer agreement obtained with the measured data using the driven coefficients for \( q_c \) values ranging from 5 MPa to 12 MPa.

6. The normalised base resistances at failure \( q_{b,0.1Db}/q_{c,avg} \) for the instrumented DCIS tests piles show excellent agreement with the UWA-05 design equation \( q_{b,0.1Db}/q_{c,avg} = 0.6 \) for closed-ended driven displacement piles and further justifies the use of the Dutch \( q_c \) averaging technique by the UWA-05 method to account for the influence of partial embedment on the base resistance of the displacement piles founded in the layered strata.

7. The gradual degradation in base stiffness \( E_{b,eq} \) with increasing normalised base displacement \( w_b/D_b \) during loading is in contrast with a typical yield displacement \( w_{byd}/D_b = 1.5 \% \) advocated by Gavin and Lehane (2007) using \textit{jacked} closed-ended steel displacement piles. The bearing capacity factors \( N_{q,0.1Db} \) back-figured from the measured base resistance at failure \( q_{b,0.1Db} \) are also in keeping with the range of \( N_q \) values reported for preformed closed-ended displacement piles in the literature.
8.2.5 Predictive performance of CPT-based displacement pile design methods

An examination of the predictive performance of seven CPT-based displacement pile design methods for estimating DCIS pile capacity, using a database of 26 piles with adjacent CPT $q_c$ profiles, revealed the following:

1. The NGI-05 method provides the best estimate of total capacity, while the simplified EF-97 and Van Impe-86 methods show poor predictive performance.
2. The UWA-05 method provides the best estimate of shaft capacity, but the coefficient of variation of 36% indicates considerable variability.
3. In terms of base capacity, all methods (with the exception of Fugro-05) yield mean $Q_{c,b}/Q_{m,b}$ values of between 0.95 and 1.02. However, the EF-97 and Van-Impe-86 methods have COVs in excess of 70% which indicates poor predictive performance.
4. The mean capacity ratios for base and total resistance are in close agreement with the results of similar studies for preformed closed-ended displacement piles by Schneider et al. (2008) and Xu et al. (2008).
5. While vastly-improved estimates of pile capacity are achieved for LCPC-82 method when driven pile coefficients (i.e. category 2A) are used in place of DCIS pile coefficients (i.e. category 1B), such a result is a direct consequence of compensating errors regarding the estimation of the local shear stress profile along the pile shaft (as the method cannot account for friction fatigue); its use is therefore discouraged.

8.2.6 DCIS load-displacement behaviour

A database of compression load tests on 105 DCIS piles in granular soil was compiled by the author to assess the load-displacement characteristics of DCIS piles. These curves were subsequently compared with a database of 30 high-quality load tests on preformed displacement piles reported in the literature using statistical simulation techniques. It was concluded that:

1. Despite having similar shaft, base and total capacities, the total resistance of DCIS piles tends to mobilise at a reduced rate in comparison to preformed displacement piles; such behaviour is attributed to the larger displacements required to mobilise peak local shear stresses $\tau_{sf}$ for DCIS piles.
2. The slope tangent capacity $Q_s$ represents a greater proportion of the total capacity at failure $Q_{t,0.1Db}$ for preformed displacement piles ($\approx 82\%$) in comparison to DCIS piles ($Q_s/Q_{t,0.1Db} \approx 0.65$).
8.3 IMPLICATIONS FOR DCIS PILE DESIGN IN SAND

Based on the research findings summarised previously, the following implications for DCIS pile design in sand are presented:

- The use of the installation resistance during driving may provide an initial, albeit crude estimate of the base resistance at failure $q_{b,0.1Db}$ of a DCIS pile during a subsequent maintained compression load test, as well as useful information regarding the variation in the in the ground profile across a site during piling operations.
- When estimating the shaft resistance of DCIS piles in sand, the reduction in normalised radial effective stress at failure $\sigma'_{rd}/q_c$ with increasing normalised distance from the pile base $h/D_s$ (i.e. friction fatigue) should be accounted for using Equation 7.5.
- The base resistance of a DCIS pile in sand behaves in a near-identical manner to a preformed driven displacement pile, with the normalised base resistance at failure $q_{b,0.1Db}/q_{c,avg}$ given by Equation 2.16.
- Recent CPT-based displacement design methods provide reasonable predictions of the shaft, base and total resistance of DCIS piles in sand.
- The total resistance of a DCIS pile tends to mobilise at a slower rate in comparison to preformed driven displacement piles (due to the increased interface roughness of DCIS piles created by in-situ concreting), implying that DCIS piles may exhibit greater levels of displacement for a given applied load.

8.4 RECOMMENDATIONS FOR FUTURE RESEARCH

Whilst this thesis provides a significant contribution to the knowledge of DCIS pile behaviour during installation, curing and maintained load testing in both layered soils (with granular sockets) and uniform sand, the following recommendations for future research are proposed:

- The base of the steel installation tube should be instrumented with a load cell to investigate more directly the relationship between the base resistance and installation resistance derived by the piling rig instrumentation during driving.
- To further verify the development of friction fatigue for DCIS piles, the shaft of the steel installation tube should be instrumented with total stress cells in order to measure the changes in radial stresses during driving.
• The database of studies of residual loads in cast-in-situ piles remains rather limited. Future studies involving instrumented cast-in-situ pile types (including DCIS piles) should therefore investigate residual loads during curing.

• The installation of driven DCIS anchor piles to provide sufficient reaction for the loading frame during maintained compression load testing of the instrumented DCIS piles inevitably led to disruption of the surrounding soil (and hence CPT $q_c$ profile), particularly for Pile S1 at Shotton. It is therefore desirable for future instrumented DCIS pile tests that CPT profiling be conducted after anchor pile installation but prior to installation of the test pile.

• The bases of the instrumented DCIS piles in this thesis were founded in granular soil and therefore behaved in a predominantly end-bearing manner during maintained compression loading. The behaviour of DCIS piles in predominantly cohesive soils should therefore be investigated.

• Changes in DCIS pile shaft diameter after casting in soft layers should be investigated by integrity testing.

• Given the dearth of published case histories involving DCIS piles, it is essential that further tests be conducted on instrumented DCIS piles in order to improve correlations for design.
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Titi, H.H. and Abu-Farsakh, M.Y. 1999. *Evaluation of bearing capacity of piles from cone penetration test data*. Louisiana Transportation Research Center, Baton Rouge, LA.


APPENDIX A  DERIVATION OF INSTALLATION RESISTANCE USING RIG INSTRUMENTATION
A.1 INTRODUCTION
This section describes in detail the derivation of the installation resistance \( q_{cp} \) of the DCIS tube based on measurements by the DCIS rig instrumentation. The rig instrumentation consists of two sensors, placed a specified distance apart, vertically above the hammer casing. During driving, the instrumentation records the time taken for the hammer rod to transit the distance between the two sensors. The permanent displacement or ‘set’ of the tube is also measured by a laser which is mounted at the top of the rig mast. The instrumentation uses these measurements to derive a dynamic driving resistance at the base of the tube \( q_{cp} \) using the Danish pile driving formula.

A.2 DERIVATION OF HAMMER DROP HEIGHT AND IMPACT ENERGY
Figure A.1 shows a diagram (in elevation view) of the instrumentation used to derive the hammer drop height. Positions \( A_{\text{rig}} \) and \( B_{\text{rig}} \) represent the two sensor locations, while the point of hammer impact is position \( C_{\text{rig}} \). The distance between the two sensors is designated \( c_{\text{rig}} \), and \( a_{\text{rig}} \) represents the distance between the lower sensor \( B_{\text{rig}} \) and the impact point \( C_{\text{rig}} \).

![Figure A.1 - Schematic of instrumentation for measuring hammer velocity](image)

The time \( t_{\text{hammer}} \) taken for the hammer rod to travel the distance between \( A_{\text{rig}} \) and \( B_{\text{rig}} \) is recorded. As the distance between the sensors is known (i.e. distance \( c_{\text{rig}} \)), the average velocity \( u_{\text{avg}} \) between \( A_{\text{rig}} \) and \( B_{\text{rig}} \) can be obtained using Equation A.1:
\[ u_{\text{avg}} = \frac{c_{\text{rig}}}{t_{\text{hammer}}} \]  

However, as the hammer must travel an additional distance \(a_{\text{rig}}\) (typically 50 mm to 150 mm, depending on the piling rig used) before impact occurs, the velocity at impact \(v_{\text{impact}}\) will be slightly larger than \(u_{\text{avg}}\). Using simple mechanics, \(v_{\text{impact}}\) is calculated as follows:

\[ v_{\text{impact}} = \sqrt{u_{\text{avg}}^2 + 2gs_{\text{rig}}} \]  

\[ s_{\text{rig}} = a_{\text{rig}} + \frac{c_{\text{rig}}}{2} \]  

Where \(s_{\text{rig}}\) is the distance travelled by the hammer and \(g = 9.81 \text{ m/s}^2\).

Substituting Equation A.1 and A.3 into A.2 yields in the following expression:

\[ v_{\text{impact}} = \sqrt{\left(\frac{c_{\text{rig}}}{t_{\text{hammer}}}\right)^2 + g\left(2a_{\text{rig}} + c_{\text{rig}}\right)} \]  

Having derived the hammer velocity at impact \(v_{\text{impact}}\), the hammer drop height \(h_{\text{hammer}}\) is back-calculated using the Law of Conservation of Energy given by Equation A.5.

\[ \frac{1}{2}mv_{\text{impact}}^2 = mgh_{\text{hammer}} \]  

\[ h = \frac{v_{\text{impact}}^2}{2g} \]  

The energy at impact \(E_{\text{hammer}}\) is then determined using Equation A.7 which represents the potential energy of the hammer.

\[ E_{\text{hammer}} = mgh_{\text{hammer}} \]
A.3 CALCULATION OF INSTALLATION RESISTANCE

The driving resistance $q_{cp}$ computed by the rig instrumentation software is based on the Danish pile driving formula which is given in Equation A.8. The input parameters for the formula are hammer energy at impact $E_{hammer}$, tube displacement or ‘set’ $s$, hammer transfer efficiency $e_{transfer}$ and elastic tube compression $c_e$. Unlike other pile driving formulae, the Danish formula provides an expression, given in Equation A.8, for the elastic compression of the driving tube which is based on the tube length $L$ and cross-sectional area $A_p$, and the elastic modulus of steel $E_{steel} \approx 209$ GPa.

\[
q_{cp} = \frac{e_{transfer} E_{hammer}}{s + 0.5 c_e} / A_p \tag{A.8}
\]

\[
c_e = \sqrt{\frac{2 e_{transfer} E_{hammer} L}{A_p E_{steel}}} \tag{A.9}
\]

The hammer transfer efficiency $e_{transfer}$ is the ratio of energy in the driving tube to energy at impact, and accounts for energy losses due to noise, heat etc. A transfer efficiency for hydraulic hammer of 80 % is assumed. The permanent set of the tube $s$ is calculated by obtaining the difference in total displacement (measured by the laser instrumentation at the top of the rig mast) between the blow in question and the previous blow.
APPENDIX B  DYNAMIC CAPWAP TESTS DURING INSTALLATION
B.1 DYNAMIC PILE TESTS

A series of dynamic tests were conducted on the steel installation tube during driving at two of the test sites (Dagenham and Shotton) using a Pile Driving Analyser (PDA) to assess the distribution of installation resistance into the corresponding shaft and base resistance components. The PDA enables simultaneous measurements of the force and velocity of the stress wave induced in the tube after hammer impact using a pair of strain gauges and accelerometers attached to the outer shaft of the tube at a height $h$ above the base corresponding to the subsequent DCIS pile length $L$. The software package Case Pile Wave Analysis Programme (CAPWAP) is subsequently used to derive the shaft, base and total resistances for a given hammer blow from the measured force and velocity data using a rigorous numerical signal-matching process with a set of input parameters (i.e. pile dimensions, material stiffness and dynamic soil properties) determined by the operator.

CAPWAP analyses were performed by ESG on behalf of NUI Galway/Keller Foundations on several blows recorded by the PDA during installation of the instrumented DCIS test piles D1 at Dagenham and S1 at Shotton. The analysed blows, which were chosen by the author, corresponded to the initial, intermediate and latter stages of driving and were all within cohesionless layers. The resulting shaft, base and total resistance derived by CAPWAP for these blows are summarised in Table B.1 where it is evident that the majority of the installation resistance was generated by the base of the tube rather than through shaft resistance. Examples of the measured force and velocity traces during installation of Pile S1 at Shotton and Pile D1 at Dagenham are shown in Figure ?(a) and (b) respectively.

<table>
<thead>
<tr>
<th>Site</th>
<th>Pile</th>
<th>Blow no.</th>
<th>Tube embedment (m)</th>
<th>Shaft resistance (kN)</th>
<th>Base resistance (kN)</th>
<th>Total resistance (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dagenham</td>
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<td>167</td>
<td>1.60</td>
<td>43</td>
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<td>550</td>
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<td></td>
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<td>102</td>
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Table B.1 – Shaft, base and total resistance during installation derived by CAPWAP
Figure B.1 – Measured CAPWAP force and velocity traces for (a) Shotton S1 and (b) Dagenham D1
APPENDIX C  LOAD TEST SCHEDULES FOR INSTRUMENTED DCIS PILES
## C.1 PONTARDDULAISS

Table C.1 – Loading schedule - Pile P1 - Pontarddulais

<table>
<thead>
<tr>
<th>Cycle 1</th>
<th>Cycle 2</th>
<th>Cycle 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load (kN)</td>
<td>% DVL</td>
<td>Load (kN)</td>
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## C.2 DAGENHAM

Table C.2 - Loading schedule - Pile D1 - Dagenham

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### C.3 ERITH

Table C.3 - Loading schedule - Pile E3 - Erith

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### C.4 SHOTTON

Table C.4 - Loading schedule - Pile S1 - Shotton

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C.2
**C.5 RYTON-ON-DUNSMORE**

Table C.5 - Loading schedules - Piles R1, R2 and R3 - Ryton-on-Dunsmore

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APPENDIX D  DCIS PILE DATABASE LOAD-DISPLACEMENT CURVES
Figure D.1 - Avonmouth - Pile TP1
Figure D.2 - Avonmouth - Pile P3792
Figure D.3 - Avonmouth II - Pile P2605
Figure D.4 - Avonmouth II - Pile P3046
Figure D.5 - Avonmouth II - P2862
Figure D.6 - Avonmouth II - Pile P5156
Figure D.19 - Barking II - Pile P18
Figure D.20 - Barking II - Pile P39
Figure D.21 - Barking III - Pile P53
Figure D.22 - Barking III - Pile P180
Figure D.23 - Barking IV - Pile P82
Figure D.24 - Barking IV - Pile P36
Figure D.25 - Barking IV - Pile P410

Figure D.26 - Barking IV - Pile P285

Figure D.27 - Barking IV - Pile P505

Figure D.28 - Barking IV - Pile P376

Figure D.29 - Barking IV - Pile TPA

Figure D.30 - Barking IV - Pile TPB
Figure D.49 - Dagenham - Pile S2193

Figure D.50 - Dagenham - Pile S1404

Figure D.51 - Dagenham - Pile S766

Figure D.52 - Dagenham - Pile S1492

Figure D.53 - Dagenham - S782

Figure D.54 - Dagenham - Pile S400
Figure D.55 - Dagenham - Pile S80
Figure D.56 - Dagenham - Pile S52
Figure D.57 - Dagenham - Pile D1
Figure D.58 - Dagenham II - Pile RS265
Figure D.59 - Dagenham II - Pile RS55
Figure D.60 - Dagenham II - Pile S1997
Figure D.61 - Dagenham III - Pile PF06

Figure D.62 - Dagenham III - Pile BO28

Figure D.63 - Dagenham III - DS401

Figure D.64 - Erith - Pile E3

Figure D.65 - Erith - Pile E2

Figure D.66 - Erith - Pile E1
Figure D.67 - Erith - Pile S523
Figure D.68 - Erith - Pile S494
Figure D.69 - Pontarddulais - Pile P276
Figure D.70 - Pontarddulais - Pile P163
Figure D.71 - Pontarddulais - Pile P621
Figure D.72 - Pontarddulais - Pile P243
Figure D.79 - Shotton - Pile CB319

Figure D.80 - Shotton - Pile CB145

Figure D.81 - Shotton - Pile CB021

Figure D.82 - Shotton - Pile CB151

Figure D.83 - Shotton - Pile SY068

Figure D.84 - Shotton - Pile SY091
Figure D.85 - Shotton - Pile SY186

Figure D.86 - Shotton - Pile VCT02

Figure D.87 - Shotton - Pile S1

Figure D.88 - Teignmouth - Pile P149

Figure D.89 - Tilbury - Pile P83

Figure D.90 - Tilbury - Pile TP1