

Review of technical issues relating to foundations and geotechnics for offshore installations in the UKCS

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Review of technical issues relating to foundations and geotechnics for offshore installations in the UKCS

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Foundation design and especially pile design and analysis are currently undergoing an important stage of technical development, with new methodologies and recommendations coming into practice. Detailed guidance on technical issues and best practice recommendations are provided in Parts 1 to 3 of this Review on the critical design issues and topics that need to be addressed in both site investigation and re analysis. Consideration is also given to possible monitoring and strengthening of foundations systems. The Parts also provide lists of relevant publications and useful references to background material and guidance on specific topics.

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PREFACE

This document was prepared for HSE by R J Jardine, Professor of Geomechanics, Imperial College, London in support of the provision of good practice in the area of foundations and geotechnics for offshore installations in the UKCS. It contains the dissemination of recent research results in this area together with examples of application of this research.

Review of Technical Issues relating to Foundations and Geotechnics for Offshore Installations in the UKCS

Introduction

Foundation design and especially pile design and analysis are currently undergoing an important stage of technical development, with new methodologies and recommendations coming into practice. Detailed guidance on technical issues and best practice recommendations are provided in Parts 1 to 3 of this Review on the critical design issues and topics that need to be addressed in both site investigation and re analysis. Consideration is also given to possible monitoring and strengthening of foundations systems. The Parts also provide lists of relevant publications and useful references to background material and guidance on specific topics.

PART 1

Pertinent Technical Issues and Best Practice Recommendations

1.0 Introduction

This Part aims to provide up-to-date guidance on the technical aspects of foundation integrity assessment. Emphasis is placed on fixed, piled, structures, as these dominate in the UK offshore section. Jack-up foundations, gravity base structures and major offshore geo-hazards are also considered; Randolph et al (2005) review other offshore geotechnical issues including deepwater site investigations, ‘suction caisson’ foundations and anchors.

The Part explains how research, field experience and improved understanding are leading to step changes in some aspects of offshore foundation design, and to steady evolution in others. *Explanatory sections are given that lead to check points that the Duty Holders may consider useful when reviewing the integrity of their installations. These are given in italics.* Some significant gaps in current knowledge exist, leading to weaknesses in both theory and practice. These gaps are also identified at appropriate points in the document. One such area is the effect of time on driven pile axial capacity, particularly for sites dominated by sands, or sensitive low OCR clays.

This Report is designed to be a ‘living document’ that can be updated as new information emerges. The document may also be revised to cater for any potential re-drafting of the ‘industry-standard’ API/ISO design recommendations for offshore foundations. The latter might, for example, consider different approaches for pile design in clays. Ten Sections are presented that cover the following main themes.

- Current and best practice regarding design of piled foundations
- Pile design issues in ‘special’ soils
- Pile cyclic loading considerations
- Pile set-up, ageing and re-assessment of existing foundations
- Pile installation problems, including driveability and buckling
- Group action under static and cyclic loads
- Considerations relating to jack up foundations

- Considerations relating to gravity base foundations
- Geo-hazards, including disturbance to foundations caused by well drilling
- Site assessment procedures for a range of applications

Reflecting the main focus of this document, the Sections devoted to piling provide the greatest degree of detailed guidance.

The references cited in the above Sections are detailed at the end of this Part (Part 1). Three further Parts follow. Part 2 lists the definitions of the various abbreviations used in the document, while Part 3 provides the main technical support for the assertions made in the Main Text regarding current practice and research developments. This work has required making a comprehensive review of recent developments in research and practice that may affect offshore pile design. Jardine and Chow (2007) summarised the findings of this review in a conveniently condensed keynote paper that was presented to the September 2007 SUT Conference on Offshore Geotechnics. This 30 page document, which is reproduced in part 3, includes many illustrations and a comprehensive set of further references.

2.0 Current and best practice regarding design of piled foundations

2.1 Overview of driven pile construction and critical design aspects

Driven steel tubular piles provide the most common form of North Sea offshore foundations. The associated manufacture and installation technologies are relatively mature. A review given by Overy (2007) of Shell UK's North Sea piling operations shows a trend for platforms designed since 1996 to employ mid-sized piles (0.660 to 2.134m diameter, with 26 to 87m penetration), for which the rated axial compressive capacities fall between 14 and 100MN. However, diameters greater than 4m have been specified for wind turbine structures in the North Sea, where piles with diameters of up to 2.5m have been driven routinely for oil and gas platforms to depths of 100m, or greater, in a variety of geotechnical settings.

The experience reviewed in Part 3 indicates that pile diameter to wall thickness ratios (D/t) of between 15 and 45 (with an average around 27) are typical in the North Sea,

although more slender ratios have been used elsewhere. Adopting high wall thicknesses may necessitate special stress relieving treatment for the pile welds, making diameters significantly greater than 2m potentially less attractive economically when working with D/t ratios lower than ~ 40 . However, thin wall piles may lead to other problems. For example the primary piles that experienced buckling failures during installation in hard calcareous layers at the Goodwyn field (NW Australia) employed a D/t ratio of 60; Randolph et al (2005). Buckling has taken place during driving in very dense sands in other major projects that may have been exacerbated by chamfered pile tip details and/or complex stepped pile specifications.

Understanding of the ground's reaction to driven pile installation and loading has lagged behind Industry's practical capabilities; design approaches are still in an imperfect state of evolution. In addition to expensive offshore pile installation failures (see for example Alm et al 2004), considerable mismatches have been found in other cases where it has proved possible to check the Industry-standard API/ISO recommendations in tests on large, offshore scale, piles (see for example Clarke 1993, Williams et al 1997, Kolk et al 2005). The informal overview of current practice given in Part 3 indicates that current design practice for clay and silica sand sites in the North Sea remains, in most cases, based on the historical API RP2A recommendations. The latter have undergone only relatively minor changes since 1993, but are due for substantial revision in late 2007. The main changes, which concern the calculation of axial capacity for piles driven in sand, have been prompted by research in several centres and vigorous debate over several years. Alternative geotechnical design frameworks have been proposed that have been applied comprehensively in some sectors (see for example Overy 2007). However, progress is being made cautiously by the API Panel and further evolution of design practice can be expected. Part 3 sets out a detailed description of the main problems of the historical API RP2A recommendations, as well as the key features of the new methods.

2.2 Piles driven in silica sand

Subsections 2.2.1 to 2.2.6 summarise the key points to be considered in relation to piles driven in sand. The assertions made are supported by the more detailed arguments and references cited in Part 3.

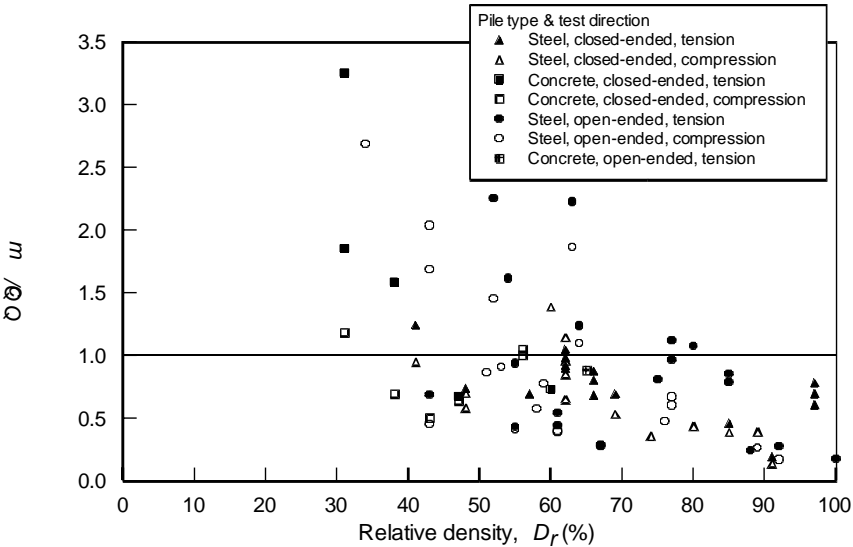
- 2.2.1 It is now generally agreed that the physical models implicit in the API-1993 approach for calculating shaft and base resistances in sand offer a poor representation of the real pile-soil system, and that the most widely used (1993) API-RP2A set of recommendations lead to skewing between calculated and measured pile capacities. API-1993 provides potentially non-conservative results for shaft capacity in loose sands, and in loose-to-medium sands with high length (L) to diameter (D) ratios. Figures 1 and 2 illustrate these skewed trends, reproducing the database comparisons given by Jardine et al (2005) between calculated (Q_c) and measured (Q_m) shaft capacities.
- 2.2.2 Non-conservative bias applies to API-1993 base capacity in loose sands, and to large diameter piles in medium dense sands. In addition, the 1993 Main Text methods do not allow for the acknowledged trend for tension shaft capacity to fall well below that applying under compression loading. The latter difficulty is sometimes addressed in UK practice by applying the pre-1993 RP2A recommendations, taking $K = 0.5$ in tension and 0.7 in compression.
- 2.2.3 Practical cases have been reported from near-shore and river-bridge projects where piles designed to the API-1993 sand method were tested to failure and found to have insufficient capacities for their intended purposes. The 1993 API sand variant can also be overly conservative in many cases, particularly with very dense sands, low L/D ratios or small diameter piles. The EURIPIDES tests in very dense North Sea sands gave medium term capacities far above the API-1993 method predictions; axial capacity was found to grow with time in a way that is not anticipated in the API-1993 recommendations or commentary. The poorly understood effects of time on capacity are currently being investigated under an HSE supported JIP.
- 2.2.4 Statistical studies of API-1993 predictions compared with pile load tests indicate a slightly conservative mean value Q_c/Q_m (calculated/measured capacities) but with large coefficients of variation (around 70%) that sit uncomfortably with existing WSD Factors of Safety or LRFD Resistance Factors. The low incidence of reported offshore piled foundation failures may reflect unaccounted for features of behaviour such as pile capacity growth with time. Other explanations include potentially lower-than-expected service loads, system redundancy or a possibly conservative bias in the conventional

API methods towards the soil conditions encountered in the North Sea and other offshore provinces.

- 2.2.5 Extensive research over the last 20 years has led to improved design methods. Field tests with instrumented piles have been particularly informative; demonstrating that the ‘earth-pressure’ and ‘shallow foundation’ theory incorporated into the historical API approaches does not model field behaviour well. New importance is given to: continuous CPT profiling (even when resistance values q_c exceed the previous upper limit of 50 MPa) as a means of gauging sand state; pile tip position; and pile tip details. A spread of new predictive methods has been developed building from these new insights. Part 3 reviews the debate that has ensued, reporting a database study by Lehane et al (2005) that concluded that the UWA-05 method and the Imperial College ICP-05 (essentially the ‘MTD’ approach of Jardine and Chow 1996) give the best reliability parameters, performing far more satisfactorily than API-1993 and better than two other ‘CPT’ based approaches and two other methods (termed Fugro-05 and NGI-05). These new approaches reduce or eliminate the skewing that results from the API-1993 methods; see Part 3.
- 2.2.6 However, that UWA-05 and ICP-05 apply different weightings to factors relating to open-end conditions and “friction fatigue” and are therefore unlikely to give coincident results when applied to identical piles in the same soil profiles. The 2007 API-RP2A recommendations have been modified cautiously to reflect the lack of universal agreement regarding the recent research. While the new Main Text method for sands retains a modified version of the conventional approach, practitioners are encouraged to consider four CPT based methods set out in the RP2A Commentary, which includes reference to the ICP-05 procedures, along with UWA-05, Fugro-05 and NGI-05. The Main Text API approach no longer contains any recommendations for loose sand. Noting the extensive field experience reported by Overy (2007) and Overy and Sayer (2007) with ICP-05, it is argued in Part 3 that the latter can be used safely to estimate medium term capacities without making the simplifications or modifications recommended by API-2007. Field experience (in the UK Sector or elsewhere) has yet to be reported with the UWA-05 approach, or the Fugro-05 and NGI-05 methods.

2.2.7 With piles in sand, designers may demonstrate that their design requirements are met or exceeded when axial capacities are checked with the ICP-05 and possibly UWA-05 methods. As discussed in Section 6 below, this practice could have additional benefits when considering predictions for pile driveability and also help to avoid problems with refusal and pile buckling.

2.2.8 A higher level of site investigation practice and geotechnical expertise is required to apply the new 'CPT based methods' than the conventional API-93 approach. It may not be inappropriate to suggest that design engineers can show proof of appropriate staff training and evidence of a sufficiently detailed



site investigation, as outlined in Section 11. In cases where the new methods are adopted as the primary design tools, it is suggested that consideration be given to the more stringent WSD Factors of Safety and LRFD Factors discussed by Jardine et al (2005).

Figure 1. Distribution of Q_d/Q_m with respect to relative density D_r ; API (1993) shaft procedure for sands, after Jardine et al (2005).

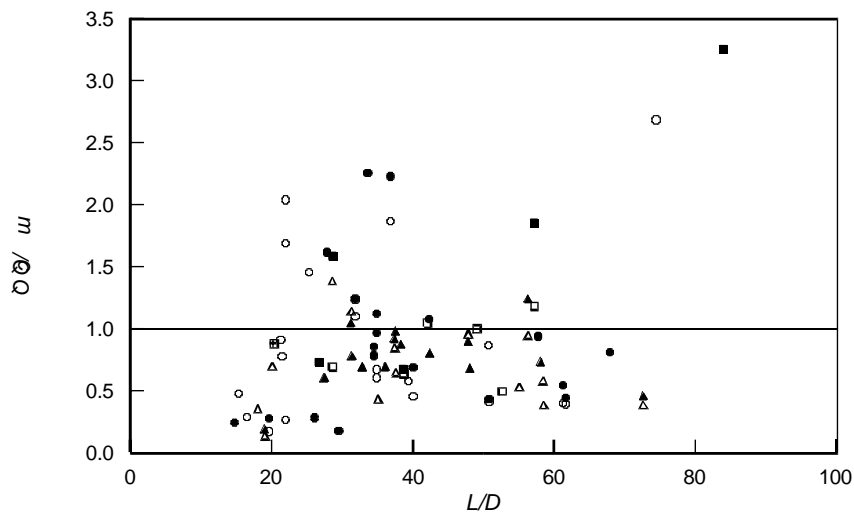


Figure 2. Distribution of Q_c/Q_m with respect to relative density L/D ; API (1993) shaft procedure for sands, after Jardine et al (2005).

2.3 Piles driven in clay

The reviews given in Part 3 also address piles driven in clay. Seven main summary points regarding axial capacity are discussed below:

- 2.3.1 Most North Sea pile designers apply the current (and 2007 revision) Main Text API-RP2A total stress approach in clays. Some retain the historical $\alpha = 0.5$ approach, while others apply an 'effective stress' method similar to that for sands with upper bound 'K' values near to the surface in stiff clays.
- 2.3.2 Statistical database assessments indicate that the Q_c/Q_m parameters associated with the current Main Text API-RP2A are generally more favourable for clay cases than with sand and the mean API Q_c/Q_m values are close to unity. Although the API predictions for end bearing capacity are subject to substantial uncertainty, this is less important in most cases than the shaft component - for which the coefficient of variation is relatively low (at around 35%).
- 2.3.3 Nevertheless, it is argued in Part 3 that the total stress API approach suffers fundamental weaknesses that render it liable to systematic skewing. In particular, it may be non-conservative when dealing with L/D ratios greater than around 60 (because of shaft 'friction fatigue' and brittleness phenomena) and/or clays with overconsolidation ratios (OCRs) less than around 2. As reviewed in Section 3, capacity may also be lower than expected with carbonate clays and those that develop a weak (slickensided) residual

strength interface shear fabric. Sensitive clays may also give difficulties; a limited class of low OCR, sensitive, low plasticity clays exists in which axial capacities can be far less than calculated.

- 2.3.4 Alternative formulations are set out in Part 3 that seek to improve predictive reliability. The ICP-05 approach (formerly MTD-96) set out by Jardine et al (2005) adopts an effective stress framework developed from intensive research with highly instrumented piles. It has been found that shaft failure is governed by an effective stress ‘Coulomb’ law, with $\tau_f = 0.8 \sigma'_{rc} \tan \delta$, where the key parameters are σ'_{rc} the radial effective stress developed as a result of installation and full equalisation (set-up) and the interface-shear friction angle δ . ICP-05 gives rules for calculating base resistance from CPT data. The method can be used conveniently in conjunction with the ICP-05 sand method and applied in layered cases.
- 2.3.5 As set out in Section 7, the ICP approach for calculating σ'_{rc} relies on site investigation parameters that are not routinely measured for API pile design purposes. It also states that design δ values should only be assessed from large displacement ring shear interface tests performed to a prescribed technique. Fast installed piles form a partially developed residual fabric that can give brittle local peak δ values. While the latter often fall well below critical state ϕ' , only a few millimetres of post peak slip may be required to reduce δ to a lower ultimate δ . This brittleness gives the potential for progressive failure down the pile shaft that may be modelled by a “falling branch” T-Z approach, or by the conservative assumption of ultimate δ values applying at all positions on the pile shaft.
- 2.3.6 Jardine et al (2005) report database studies that indicate improved statistics for ICP-05, with Q_c/Q_m values around unity and coefficients of variation around 20% for both base and shaft approaches. Field evidence reported by Overy (2007) for nine North Sea locations indicates ICP-05 capacities that can differ considerably from the API Main Text predictions but fall closer to (typically marginally below) the short-term field axial resistances assessed during pile driving by instrumented pile monitoring. Noting the well known tendency for set-up to develop relatively rapidly, these observations support the method’s application to North Sea installations.

2.3.7 Noting the friction fatigue factors in the ICP approach, Kolk and van de Velde (1996) proposed a variation of the API-93 method that took account for pile length effects, where the average coefficient $\alpha = \tau_f/S_u$ depended on the pile length:

$$\alpha = \tau_f/S_u = 0.55 (\sigma'_{v0}/S_u)^{0.3} (L/40D)^{-0.2}$$

The method can be recast in an effective stress format to take account of variations in δ , but it is not clear how the approach should be applied in layered deposits.

2.3.8 *In summary, assessment of designs developed with the conventional API Main Text method should consider whether non-conservative factors apply that might lead to capacity over-estimates. These include: high L/D ratios, low OCRs and the list of 'special' clays discussed in Section 3. In most of the 'special cases' alternative procedures exist that may be preferred when the necessary site investigation parameters are available (see Section 11). Site monitoring may also be recommended.*

2.3.9 *It appears that designers can realise significant practical advantages by using alternative 'modern' procedures such as ICP-05 (Overy 2007). However, be aware that a higher level of site investigation practice and geotechnical expertise is required than with the conventional API-93 approach. Careful consideration should also be given to the selection of WSD and LRFD design factors; it may be appropriate to suggest that the design engineers can show proof of appropriate staff training and evidence of a sufficiently detailed site investigation.*

2.4 Load-displacement behaviour

Offshore designers have to show that their pile foundations can withstand lateral loads and pile head moments without developing excessive deformations or pile material over-stressing. The standard API P-Y procedures set out in API-RP2A are usually employed for this purpose, and the more conservative 'cyclic' curves are often adopted in North Sea practice when considering piles in clay.

While load-displacement predictions are usually of secondary importance in foundation safety assessment, they can affect the fatigue life calculations for structural components and may be vital to the interpretation of any foundation

movement monitoring (see Jardine and Potts 1988). The offshore Industry's standard practice of applying T-Z and P-Y analyses for single piles and elastic interaction factors for pile groups has changed little in recent years and is generally considered to be marginally conservative. Points to note include:

- 2.4.1 Field measurements have indicated that conventional procedures over-estimate foundation movements under load, particularly for groups involving multiple piles. At the Magnus and Hutton TLP sites axial stiffnesses were four-to-five times greater than expected, while lateral and moment-rotation stiffnesses were under-estimated by factors of 2 to 3.
- 2.4.2 Improved predictions can be made, where necessary, by advanced Finite Element techniques that model: the succession of strata; the effective stress level dependence and extreme stiffness non-linearity shown by offshore soils (ie incorporating a suitable "small-strain" stiffness formulation); a reliable approach for defining the potentially brittle local shaft failure characteristics; and a flexible FE code.
- 2.4.3 The analytical techniques applied to the Magnus and Hutton cases are described by Jardine and Potts (1988) and (1993). These and alternative approaches such as those described by Simpson (1992) or Whittle et al (1993) are now applied routinely in major civil engineering projects and may be applied usefully to offshore foundation analysis; see Jardine et al (2005).
- 2.4.4 *Advanced Finite Element analyses may be proposed if there is concern that insufficient allowance has been made for potential fatigue failures, or when criteria are being set for in-service monitoring of platform safety.*

It also worth considering the recommendation monitoring foundation displacement for any platforms where there is concern over the existing foundations' fitness-for-purpose'. High resolution static measurement systems have been available for many years (see Part 3). Less expensive dynamic accelerometers, or GPS based equipment may be applicable in some cases.

3.0 Pile design issues in 'special' soils

3.1 General

The summary of current practice set out in Part 3 suggests that most designers working on UK Sector North Sea projects do not, at present, make allowances routinely for 'non-standard' soil types apart from soft rocks, such as Chalk and weak Mudstone. However, the review given in Part 3 does highlight some 'special' soil types in which driven piles mobilise substantially lower medium term capacities than might be expected by current approaches. The following sub-sections outline briefly the most significant of these special cases.

3.2 Piles driven in carbonate or mica sands

Carbonate sands are characterised by a high intrinsic compressibility and a strong susceptibility to load cycling. They are broadly defined as having CaCO₃ contents exceeding 50%, although difficulties may also appear at lower carbonate contents. While such sands are unusual in the North Sea they are encountered, for example, offshore Brittany. Severe problems have been encountered with piles driven in carbonate deposits offshore Australia, in the Arabian Gulf, offshore South Africa, Brazil and in the Mediterranean. The difficulties have included pile buckling during driving through hard Calcarenite layers, and very low shaft and base resistances being inferred from driving records in uncemented layers that have indicated inadequate axial capacities. Current silica sands methods (API-1993, API-2007, ICP-05, UWA-05, Fugro-05 or NGI-05) are all liable to be non-conservative in uncemented carbonate sands.

Thompson and Jardine (1998) proposed a simple, approximate and safe approach for estimating axial capacity in calcareous sands that has found application in Mediterranean and other projects. These recommendations are re-stated by Jardine et al (2005), who also report that the axial capacities of piles driven at a 'problem' mica sand site fell around 30% lower than expected by the ICP procedures, at any given age. It is well known that the presence of even small percentages of mica flakes can make principally silica (or other hard mineral) sands considerably far more compressible and collapsible. Schneider et al (2007) argue that different geometrical factors relating to pile end conditions and steeper 'friction fatigue' decay rates apply to silica, mica and carbonate sands.

Enquiries should be made about whether calcareous or micaceous sands have been encountered at any given site and, if so, whether the designers have applied the information in the above cited references to establish safe pile designs. They may

also consider whether the assessment made requires checking by instrumented pile driving monitoring, including any appropriate re-strike tests.

3.3 Problem clays

We consider below three types of ‘clays’ that may not conform to standard design approaches: low plasticity, sensitive, low OCR clays or clay-silts, carbonate deposits and plastic clays containing slickensided shear surfaces. We consider first Karlsrud et al (2005) observations for piles driven in low OCR, low plasticity, clays and clay-silts. Tests by NGI indicated that the axial capacities developed at three onshore research sites (Lierstranda, Onsøy and Pentre) fell well below predictions made with both the API-93 and ICP-05 clay methods. Karlsrud et al (2005) proposed an NGI-05 clay capacity calculation method that defines values of α as functions of plasticity index I_p and S_u/σ'_{v0} ; their curves have a dramatic impact on low OCR, low I_p clays, leading to far lower capacities than API-93. NGI-05 does not include allowance for local brittleness, or any effect of relative pile tip depth (often termed ‘friction fatigue’) that could affect local τ_{ztf} . However, the NGI group’s tests suggest that capacity may grow with time to give higher long term values at the sensitive ‘problem’ clay sites.

Noting the relatively low α values found in the ‘LDP’ pile tests performed at Pentre on low OCR, low plasticity, clay-silt Fugro-McClelland (FM) proposed in 1992 a more conservative API variant which is applied by some designers to low plasticity, low OCR, clays or clay silts. In this “FM-92” method α is taken equal to $0.5 [S_u/\sigma'_{v0}] - 0.16$. Jardine et al (2005) argue that the low capacities measured at Pentre can be explained by the effective stress ICP-05 approach, noting that the latter matches the field tests well, provided the appropriate site specific input parameters are adopted (see Section 11). They also note that pile driving data presented by Overy (2007) for nine UK sites indicated encouraging support for the ICP-05 approach in clays with a range of OCRs and plasticity indices. Ridgway and Jardine (2007) agreed that a ‘problem’ class of low OCR sensitive clays (or clay-silts) does exist, but argued that it encompasses a narrower spread of material types than suggested by NGI, and that the problem clays and clay-silts occupy one corner of the well known CPTU classification diagrams, as illustrated in Figures 3.

Interpretation of pile driving monitoring at Arabian Gulf and Mediterranean sites, where carbonate clays are common, has indicated Soil Resistances to Driving (SRDs) that are substantially lower than expected for ‘standard’ clays with equivalent

undrained shear strength profiles. Positive post-driving set-up trends have also been noted. Some practitioners now apply the “FM-92” method as a local ‘carbonate clay’ approach, while others have recently applied the ICP-05, taking a cautious approach when selecting the key input parameters.

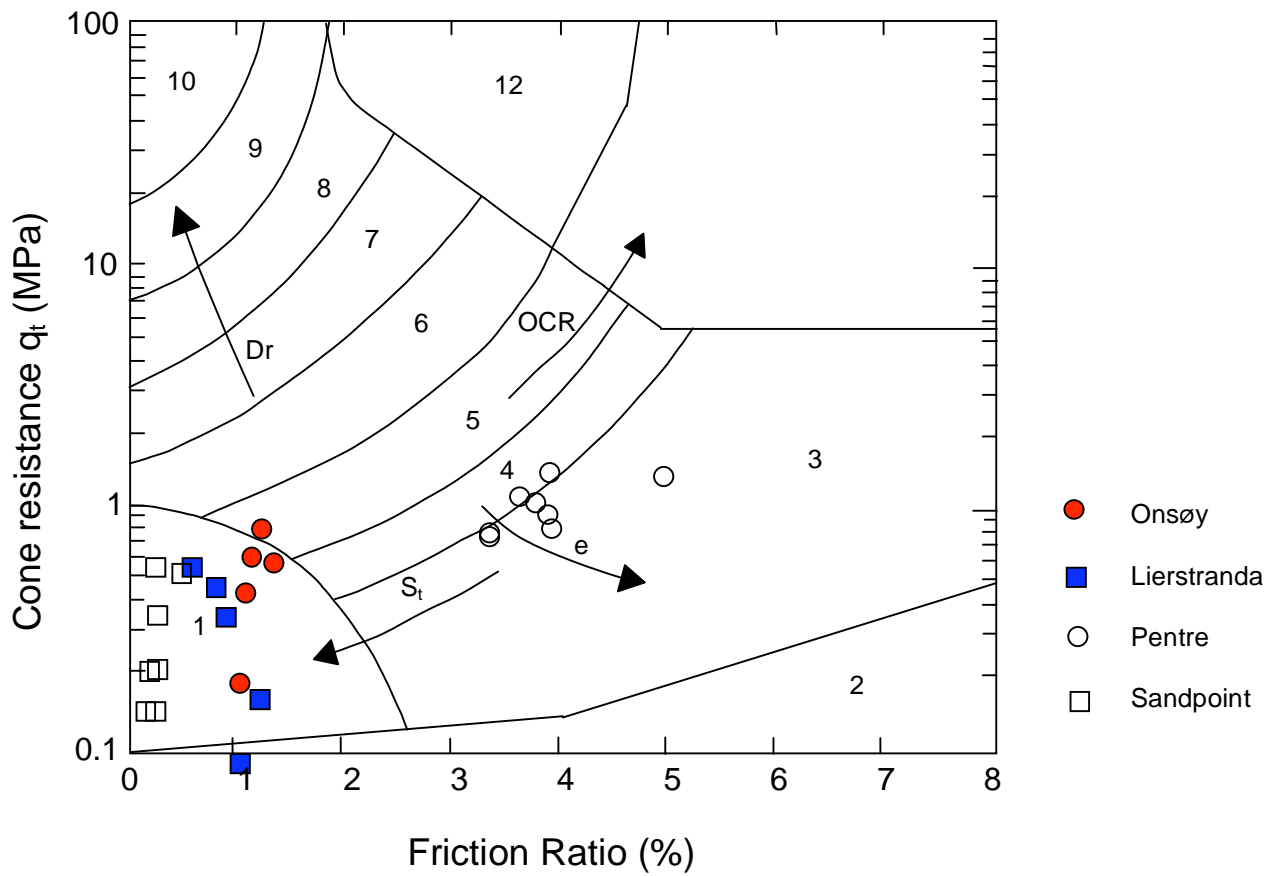
The final class of ‘problem’ clays consists of plastic soils that contain pre-existing slickensided shear surfaces, or those that develop such features readily during handling or routine shear testing. Such materials are likely to have particularly low interface effective shear angles (δ) and develop lower than expected values of α . Use of the ICP-05 effective stress method, supported by appropriate site investigation data, should lead to satisfactory predictions of capacity in such cases, as verified by the WD58A case history from the Gulf of Mexico; Jardine et al (2005).

Relevant conclusions regarding the above ‘problem’ clay soils are:

- 3.3.1 *Lower-than-anticipated axial capacities may be developed in a class of ‘problem’ sensitive low OCR, low plasticity, clays (or clay-silts). Although probably uncommon, these deposits may exist in the North Sea. Further research is required into such cases, which should encompass the potential effects of time on static capacity. We understand that NGI is about to embark on studies of this type.*
- 3.3.2 *In the interim, it is advised to apply caution for sites where low plasticity, low OCR, clays are encountered, especially if the CPT data fall in the ‘problem’ zones identified by Ridgway and Jardine, one of which is reproduced as Figure 3.*
- 3.3.3 *In cases where carbonate clays are encountered the foundation design should be checked with alternative methods such as the FM-92 or ICP-05 clay methods.*
- 3.3.4 *Capacity calculations made for carbonate clays with the ICP-05 should involve a conservative assessment of the design profiles for Yield Stress Ratio (YSR, or apparent OCR) based on the weaker (less cemented) layers present within the profile. The clay sensitivity parameters should likewise be based on a conservative interpretation of the available SI data. Site specific interface shear measurements and unit weight profiles should also be employed.*
- 3.3.5 *The occurrence of plastic clays that contain slickensided shear surfaces, or form these features readily, is indicative of lower capacities being developed*

than expected from API-93. The ICP-05 effective stress procedure is likely to give more representative capacities.

3.3.6 Data from instrumented pile driving records can help to verify the effects that the above potentially problem clays may have on foundation performance. Carefully planned re-strike tests would provide useful information on potential set up effects. Pile capacity is likely to require upgrading in cases where the problematic clays are encountered. Foundation displacement monitoring may also be used to assess the continuing fitness-for-purpose of any foundations where doubt may exist over the approach taken for pile design in potentially problem clays.



Soil Behaviour Type	
1	Sensitive fine grained
2	Organic material
3	Clay
4	Silty clay to clay
5	Clayey silt to silty clay
6	Sandy silt to clayey silt
7	Silty sand to sandy silt
8	Sand to silty sand
9	Sand
10	Gravelly sand to sand
11	Very stiff fine grained (overconsolidated /cemented)
12	Sand to clayey sand (overconsolidated /cemented)

Figure 3: ‘Problem clay sites’ plotted on Soil Behaviour classification chart based on CPT or piezocone data, after Ridgway and Jardine (2007).

3.4 Piling in soft rocks and Chalk

While most UK North Sea Sector platforms are founded on piles driven through sequences of clays and sands, some encounter soft rocks. Routine practice in these deposits is to apply guidance gathered in the same strata through experience in onshore civil engineering, as published by bodies such as the Construction Industry Research Information Association (CIRIA). For example CIRIA C574, which was published in 2002, covers pile behaviour in the Chalk which is a soft rock that is almost entirely composed of silt sized CaCO_3 particles of biological origin; it can exist in both low and high porosity forms. As with carbonate sands and clays, open piles driven in Chalk of high porosity develop very low shaft resistances in medium-term pile load tests while denser layers that include fewer discontinuities achieve better resistances.

Projecting trends on the basis of a modest number of short term tests, CIRIA quote ranges for τ_f between 10 to 20 kPa and 100 to 150 kPa for the two extreme ranges for Chalk and relate maximum base resistances q_b to SPT or CPT measurements made near the tip, quoting a range of $q_b/q_c = 0.75$ to 1.0. CIRIA note that set-up processes may lead to greater long term resistances and trial pile tests are recommended to reduce uncertainty for onshore projects. CIRIA's recent findings supersede their earlier recommendation of an effective stress approach similar to that applied to sands in API-1993. However, further research into the behaviour of offshore piles driven in soft rocks is clearly warranted.

3.4.1 In the absence of other local data evidence should be sought that the latest onshore guidance relating to the relevant soft rock type is applied appropriately. Confirmation of design assumptions through pile driving monitoring, ideally combined with re-strike tests to assess set-up characteristics should also be sought.

4.0 Cyclic loading considerations

4.1 General

The critical loading conditions for many offshore foundations involve storm conditions. However, cyclic loading is rarely addressed explicitly in design, except by adopting marginally softer P-Y relationships for clays. It is often assumed that positive loading rate effects counteract any cyclic degradation of axial capacity, even though research in Norway, the UK and elsewhere has demonstrated that piles

driven in clays and sand can experience significant capacity losses when subjected to high level cyclic loading. The only field scale cyclic tests published to-date involving open tubular piles driven in sand, were obtained in an HSE funded study conducted at Dunkirk, France (Jardine and Standing 2000); methods for assessing cyclic effects in sands and clays are discussed in Part 3.

4.2 Cyclic and static load interactions

Their interpretation cyclic loading tests is aided by cyclic interaction diagram such as that from the Dunkirk sand tests presented in Figure 4; Part 3 presents similar charts for clays. The number of cycles required to bring about failure depends on the combination of the amplitudes (Q_{cyclic}) and average 'static' values of the applied loads. When considering regular cycles, it is common to divide by the static global or local shaft capacity $Q_{max\ static}$ to give the non-dimensional parameters:

$$\begin{aligned} \text{Cyclic amplitude} &= Q_{cyclic}/Q_{max\ static} \\ \text{Mid-cycle, average load} &= Q_{average}/Q_{max\ static} \end{aligned}$$

The rates of loss in capacity increase systematically with $Q_{cyclic}/Q_{max\ static}$ which has a maximum value of around unity (when $Q_{average}/Q_{max\ static} = 0$) under extreme two-way loading, which involves alternating from compression shaft failure, through to tension failure, with each cycle. Failure can be expected in both sands and clays after some tens or hundreds of cycles at even half this cyclic loading level, when $Q_{cyclic}/Q_{max\ static} = 0.5$ and $Q_{average}/Q_{max\ static} = 0$.

The maximum value of $Q_{cyclic}/Q_{max\ static}$ that can be imposed under one way (tension or compression) loading conditions is 0.5, which applies when $Q_{average}/Q_{max\ static}$ is also 0.5. Increasing $Q_{average}/Q_{max\ static}$ accelerates the rate of degradation for any given cyclic amplitude ratio. Similarly, degradation generally accelerates with $Q_{cyclic}/Q_{max\ static}$ for piles loaded to a fixed factor of safety. However, low level cycling within the stable region of the interaction diagram may even be beneficial and piles appear to recover their capacities with time after cyclic loading, provided that they do not develop significant pile-soil slip displacements during cycling.

- 4.2.1 *High level cycling can degrade capacities in both clays and sands. In general, sensitive low plasticity clays and compressible carbonate/mica sands (or chalks) are likely to experience more rapid degradation than other, more stable, geomaterials. The potential impact grows with the normalised cyclic amplitudes and may affect lightweight structures more than heavier platforms.*
- 4.2.2 *It is not wise to rely on significant base resistance under cyclic loading because in most cases the shaft movements required to mobilise the base capacity lead to extreme two-way shaft loading; the same process applies during pile driving and is one of the causes of the 'friction fatigue' contribution to the h/R^* effects discussed earlier.*
- 4.2.3 *Cyclic degradation studies may be required in critical cases. Field displacement monitoring during storm loading may also be used to assess fitness-for-purpose under cyclic loading.*

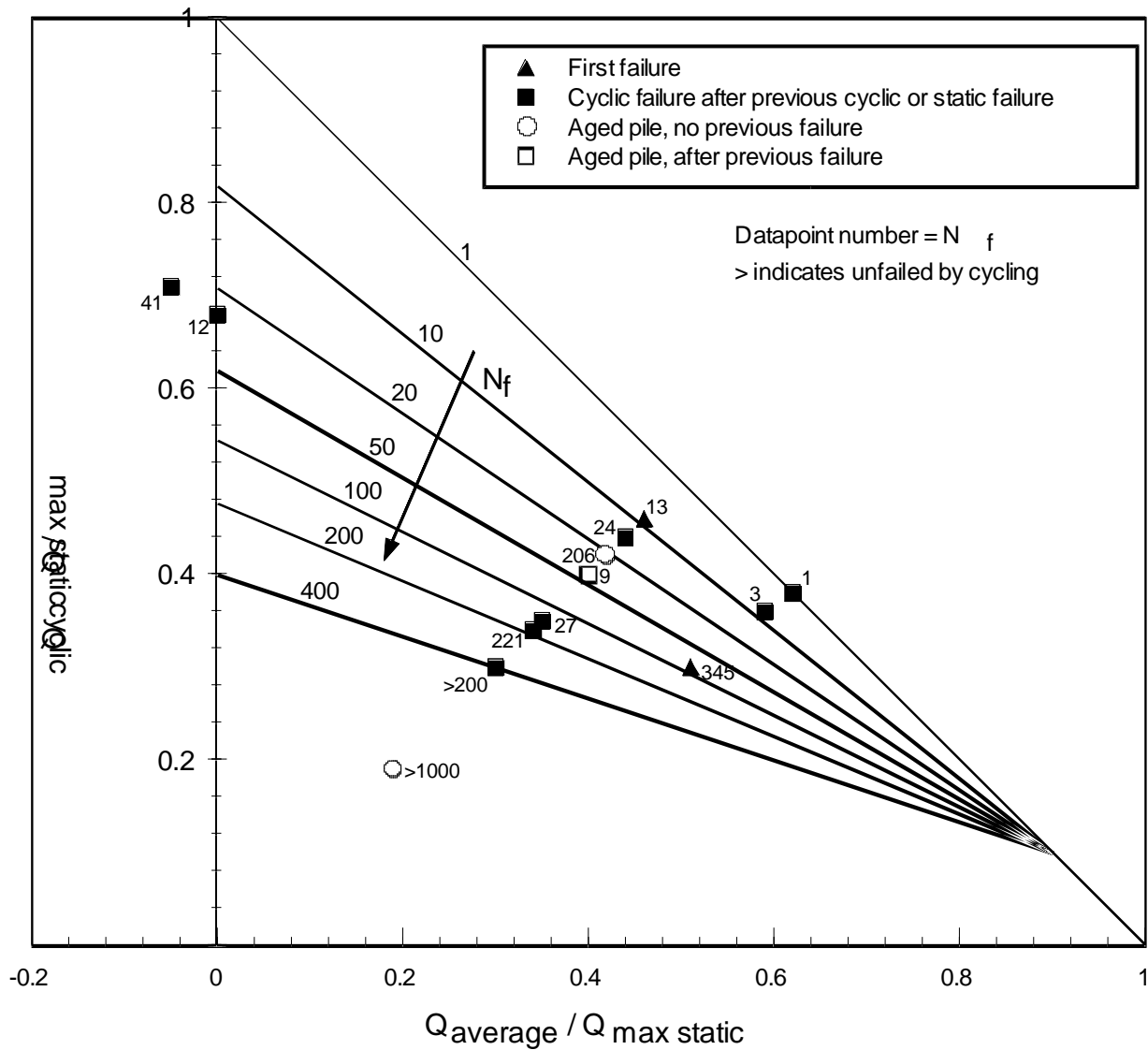


Figure 4. Cyclic interaction diagram from one way tension loading tests on 19m long, 457mm diameter piles, in dense sand at Dunkirk, after Jardine and Standing (2000).

5.0 Pile set-up, ageing and re-assessment of existing foundations

5.1 General

It is well known that piles driven in clay can develop increases in shaft capacity with time; it is generally less well appreciated that piles driven in sand may experience equally significant gains over the months or years that follow driving. While the pore-water pressure diffusion and re-consolidation mechanism has been explored most intensively for clay soils, changes in capacity can continue at times beyond those required for pore pressure equalisation in both clays and sands. It has been

suggested that the latter long term trends may involve local chemical changes, the effects of creep on the stress fields developed around the piles, or the possible effects of pre-shearing on the interface dilation characteristics.

5.2 Piles in clay

Jardine et al (2005) discuss the changes in shaft capacity in clays that occur during the pore pressure equalisation period in clays. Reporting field tests with highly instrumented closed-ended jacked piles, they note that low OCR, sensitive soils display the largest gains in capacity with time. Insensitive high OCR clays indicate smaller changes and can experience a marginal reduction in capacity over the first few days or weeks after installation.

Instrumented monitoring data (most of which is unpublished) from North Sea pile driving operations invariably indicates gains in resistance, or positive set-up, taking place in clays of all OCRs during pile driving pauses, which usually extend from a few hours up to several days. The large scale LDPT field tests on tubular piles indicated considerably larger long term static shaft resistances than those interpreted during installation from driving instrumentation (Randolph 1993). An average 'set-up' factor of around 5 was found by static testing 44 days after driving (when pore pressure dissipation was complete) in the low OCR Pentre clay-silt. The factor applying to the high OCR Tilbrook case (130 days after driving, without full pore-pressure equalisation) was lower, at around 2.

Trends for capacity to continue growing well after the completion of all 'driving' pore pressure equalisation have been reported by NGI at two of the low sensitive OCR 'problem' clay-silt and clay sites (Lierstranda and Onsøy) discussed above in Section 3. NGI interpreted these as being due to a 'pre-shear' effect analogous to that seen in Direct Simple Shear (DSS) laboratory tests.

Chow et al (1997) and Jardine et al (2006) consider how creep might allow a relaxation of the circumferential stresses developed around piles driven in high ϕ' , low compressibility, cyclically susceptible, sands that could lead to time dependent gains in radial effective stresses. The same mechanism may apply to the low-plasticity, low OCR, 'problem' clays, in which capacities may recover over months or years from exceptionally low short term values towards the levels conventionally expected for more 'typical' clays.

Pellew (2002) also noted marked capacity gains in re-tests conducted on steel piles driven in insensitive, high-OCR, London clay at times of up to 19 years after installation. His investigations indicated that Sulphate Reducing Bacteria (SRB) played a key role in catalysing chemical reactions that disrupted the weak residual strength shear surfaces that had developed around the pile perimeter during pile driving and prior load testing to failure.

5.3 Piles in sand

As mentioned above, it is now clear that ageing processes apply to piles driven in sand that can lead to very substantial gains in shaft capacity with time. The weakening by creep of circumferential arching mechanisms, and potentially stronger interface dilation, are considered the most likely sources for such effects. While the capacities of piles that are not taken to failure can double or treble within a year of driving, prior-testing disrupts the growth of capacity with time and multiple tests on single piles can lead to highly confusing results; Jardine et al (2006). Current design methods aim to match short-to-medium term capacities and it is not yet clear how rapidly shaft capacity may change in the first few weeks after driving. The ageing processes may be accelerated by low level cyclic loading. HSE are participants in a current JIP investigation by a team from Imperial College London and INGP (Grenoble) that is using highly instrumented calibration chamber tests to explore the stress conditions associated with piles installed in sand, and how these may vary with time. Considerable implications are likely to follow (regarding for example the re-use of existing foundations) if the ageing processes can be understood sufficiently well to be exploited confidently in practice.

5.4 Axial capacities deduced from driving monitoring are likely to provide lower bound estimates of the long term static capacities of piles driven in sands and clays. While capacity growth with time may be expected in some soil types, field evidence of this trend is essential when evaluating critical cases. Limited penetration instrumented re-strike tests, conducted at selected intervals after installation, may provide such evidence - provided the tests are well planned, conducted and interpreted. Progress with research will be highlighted in future updates of this document.

6.0 Pile installation problems, including driveability and buckling

6.1 General

Pile driving difficulties can be encountered due to: (i) the piles being dented or otherwise damaged during offshore handling, (ii) hammers performing poorly, (iii) the soil conditions leading to either harder or easier driving than expected, or (iv) encountering rock layers.

Instrumented pile monitoring is often performed routinely to provide a pile design quality assurance check and this should be encouraged. Soil Resistances to Driving (SRDs) are notoriously difficult to predict accurately. However, they may be checked during and after driving by dynamic analysis and stress wave matching procedures. Applying the improved methods for pile static capacity assessment described in Part 3 should allow dynamic SRD estimates to be improved, so aiding design driveability analysis, hammer selection and driving data interpretation. Overy (2007) shows that applying the ICP procedures led to moderately conservative estimates for the 'static' components of the SRDs measured at nine North Sea sites, faring considerably better than the conventional SRD prediction procedures.

Piles have been driven successfully in even the hardest North Sea clay tills; cases of pile refusal usually concern rocks, cemented soils or dense sands. Both unexpectedly large free-fall penetrations, and primary pile buckling failures, have been encountered during driving in calcareous sand deposits (Randolph et al 2005). Where SRDs are much lower than expected, further analysis may indicate worse than expected ground conditions, such as the presence of 'problem' clays or sands. Driving refusals and catastrophic buckling have been experienced at North Sea locations, and in other provinces, where thick layers of very dense silica sands have been encountered at depth (eg Alm et al 2004). Investigation of these problems has led to an appreciation by the industry of: (i) the need to invest in appropriate site investigations, including site specific CPT tests with appropriate capacity cones and the use of geophysics to identify potential local changes in stratigraphy, (ii) the benefits of adopting sufficient pile wall thicknesses (with low D/t ratios) and (iii) the importance of avoiding pile make-up and driving details that can encourage buckling and early refusal.

6.2 Potential remedial actions

Pre-drilling, jetting and vibration have been applied to ease conditions when pile driving proves harder than expected. Model studies reported by Jaime et al (1991) suggest that the negative effects of pre-boring can be very significant in clays, while Overy and Sayer (2007) describe how drill-driving greatly reduced (in a deliberately engineered operation) the shaft capacities developed in dense sands. In general, such remedial measures downgrade axial capacity and careful investigations employing advanced numerical methods, centrifuge studies, or field trials, are needed to be certain of their potential impact on static capacity. If feasible, applying more powerful hammers may provide a better solution to refusal, provided the piles remain intact and can withstand the fatigue loading.

An alternative approach to early refusal is to undertake detailed analyses of any pile driving instrumentation records and/or a re-analysis of the pile characteristics on the basis of new site investigation data, possibly applying more advanced analytical methods. The results may indicate that the piles are fit for purpose at shallower-than-intended penetrations. However, down-pile surveys may be required to demonstrate that pile buckling has not taken place, and it is necessary to demonstrate that the pile wall thickness is sufficient at the mud-line to carry the design lateral and moment loading. If welded shear studding has not been provided over the lengths of the pile that fall within the pile sleeve, the usual pile-to-sleeve grouting procedures may prove unfeasible. Expensive 'piggy-back' solutions involving additional structural elements and new piles may be warranted in particularly difficult cases, as described by Alm et al (2004).

6.3 Conclusions to note:

6.3.1 *Pile driving SRDs are difficult to predict in advance. Applying recent developments in pile capacity assessment should improve the reliability of SRD predictions.*

6.3.2 *In cases where the SRDs fall far below the expected values, the foundation capacities may prove to be inadequate. Re-strike tests may be used to gauge whether set-up processes are at work that could lead to the desired capacities developing within an appropriate time scale. In cases where this cannot be demonstrated, the driving of insert piles, or other remedial measures, may*

have to be considered. In non-critical cases, platform in-service displacement monitoring may provide a means of checking fitness-for-purpose.

6.3.3 The risks of pile early refusal during driving may be reduced by applying improved site investigation procedures and SRD estimation methods, as well as by avoiding pile details that are now known to be associated with buckling failures.

6.3.4 Conventional methods for easing hard driving may cause degradation of capacity, and this should be acknowledged and assessed. Checks should be made for pile buckling when refusal is met with high capacity hammers. Fatigue life, shear capacity and pile-sleeve connection details may also require checking in cases where refusal has been met.

7.0 Group action under static and cyclic loading

7.1 Load-displacement behaviour

Field measurements of foundation response to both static and storm loading indicate that the standard practice of applying T-Z and P-Y analyses for single piles and elastic interaction factors for pile groups can over-predict displacements considerably. Fully non-linear analyses that recognise the departure from elasticity at even very small strains are required to reproduce field behaviour; Jardine and Potts (1988), (1993). The implications of over-predicting displacements are generally benign for static and fatigue structural analysis, but could lead to unwarranted complacency when movements are monitored to assess foundation safety.

7.2 Pile group effects on static and cyclic capacities

Field tests with concrete piles driven in clays have identified negative aspects of pile group interaction that are rarely considered in offshore design. Lehane et al (2003), (2004) report HSE funded studies that showed the interacting shear stress fields developed around the piles lead to:

- Group static capacities significantly less than the sum of the individual piles, and failures that did not involve the equivalent caisson mechanism that is usually considered in offshore analysis.

- A greater-than-expected susceptibility of the pile group to cyclic loading, with more severe degradation developing under given loading levels than with single piles.

The exacerbating effects of group action may be important in offshore pile group design, which frequently employs closer group spacing ratios (s/D , where s is the pile to pile spacing) than equivalent onshore pile designs. Offshore groups installed in clays may therefore be subject to potentially significant negative capacity interactions in clays; Jardine et al (2005). Chow (1995) showed that in sands the interactions can be more positive, leading to overall gains in shaft capacity. However, the mobilisable base resistances were found to reduce very significantly as each neighbouring pile was driven.

7.3 There is the potential for downgrading of static and cyclic capacity in cases where designers specify pile groups involving s/D ratios lower than 4.

8.0 Recent developments relating to jack up foundations

8.1 Introduction

The installation of mobile offshore jack-up units involves the foundations (commonly referred to as spudcans) being subjected to elevated pre-load, which is designed to assure stability during operating conditions by penetrating the spud-can to a sufficient depth of embedment to cope with the design storm conditions. Pre-load levels and penetrations are, however, limited by the physical capabilities of the jack-up unit. Foundation assessments for jack-up units are commonly undertaken in accordance with guidance in SNAME T&RB 5-5A (2002), DNV (1992, 2001), ABS (2001), Lloyds (2001), RINA (1996) or BV (1996). The most commonly applied guidance in UK territorial waters is given in SNAME T&RB 5-5A; an ISO Standard (ISO 19905) is being developed for the site-specific assessment of mobile offshore units, based on the same criteria, with a target publication date of May 2010.

Spud can details are usually generic and are rarely designed to suit for the ground conditions at particular locations. Foundation behaviour, during both pre-loading and subsequent service life, varies from location-to-location and in some cases may

involve undesirable characteristics. Foundation failures can arise from one, or a combination of causes, as considered below.

8.2 Typical modes of foundation failures

8.2.1 Punch-through

Punch-through refers to the potentially dangerous situation where a stronger soil layer overlies a weaker stratum and even a small additional spudcan penetration (below a critical depth) may lead to a significant reduction in bearing capacity. Uncontrolled rapid leg penetrations may occur resulting in severe damage to the jack-up legs, or at worst, catastrophic failure. Notable punch-through incidents include:

- Gulf Saint Vincent (South Australia): drilled in 1996 by the Maersk Victory. Punch-through occurred during preloading operations resulting in extensive damage to all three of the jack-up's legs. No fatalities were reported.
- North Sea (Norwegian Sector): drilled in 1995 by the West Omicron. Punch-through occurred where one leg penetrated by 1.5m.
- Gulf of Mexico: drilled in 2000 by the Glomar Adriatic 3. The port leg punched through by 8.5m causing the jack-up to hit the adjacent platform and resulted in significant damage to both structures.
- Natuna (Indonesia): drilled in 2004 by the Atwood Beacon. Punch-through occurred during preloading operations resulting in the bow leg being snapped off. The other legs had to be cut off.
- Indonesia: drilled in 2007 by the West Larissa. Punch-through occurred when positioning at a jacket resulting in leg damage.

8.2.2 Scour

Scour may partially remove the soil from below the spudcan, resulting in a reduction of the ultimate bearing capacity of the foundation and any seabed fixity. This is normally a gradual process and the effects of the reduced bearing capacity may not be apparent until during storm loading when (rapid) downward movement of the leg may occur. The effects of scour are potentially more severe when it occurs at a location where a potential for punch-through exists.

A recent notable scour incident occurred in the southern North Sea with the GSF Monarch in 2002. Whilst on location scour occurred at the bow leg resulting in a loss of trim at the working air gap. Subsequent jacking operations to level the unit resulted in buckling of braces in the stern legs due primarily to the eccentricity in the foundation reaction acting on the spudcans.

8.2.3 Spudcan-footprint interaction

Spudcan-footprint interaction takes place when spudcans are positioned close to, or partially overlap, footprints derived from previous jack-up installations. The slope at the footprint perimeter together with differences in resistance between the disturbed and undisturbed soil, may cause the spudcans to slide towards the footprint. The resulting leg displacements could cause severe damage to the structure and, at worst, could lead to catastrophic failure. The situation could also be complicated by the proximity of a fixed structure. Similar considerations arise where any hard sloping stratum forms the foundation for the spudcans. The Ensco 101 suffered leg damage at a North Sea location in 2000, where a seabed footprint was identified as a contributory factor.

Crater infilling solutions to footprint problems have been proposed and attempted. However, advanced numerical analyses reported by Jardine et al (2002) and Grammatikopolou et al (2007) indicate that these solutions may not be advisable when considering clay foundations.

8.2.4 Spudcan-pile interaction

When a jack-up unit is installed in close proximity to a piled structure, soil displacements caused by spudcan penetration and extraction apply lateral loads to the nearby piles. The amount of soil displacement will depend on the soil type, spudcan-pile spacing, spudcan size and penetration. Guidance on assessing the consequences of the induced pile loading is given by Siciliano et al (1990), Stewart (2005) and Xie et al (2007).

8.2.5 Shallow gas

As with fixed structures, the presence of gas in foundation soils may result in hazards such as reduced bearing capacity, unpredictable foundation behaviour (due to seabed depressions or gas accumulations under the spudcans) and complications

with shallow drilling operations, including blowouts. The latter involve an uncontrolled discharge of gas and/or oil through the drill string to the surface, which can then subsequently ignite and explode. Notable jack-up related blowouts include:

- Hasbah Platform (Persian Gulf): drilled in 1980 by the Ron Tappmeyer. The well blew out for 8 days and cost the lives of 19 men.
- Nigerian Coast (exact location unknown): drilled in 1989 by the Santa Fe Al Baz. A shallow gas blow out ignited causing the death of the five crew members. The rig subsequently sank.
- Temsah Platform (Mediterranean Sea, Egypt): drilled in 2004 by the Adriatic IV. The well blew out for over a week, but with no casualties. The vessel sank and the platform was damaged beyond repair.
- Platform A, Eugene Island Block 273 (Gulf of Mexico): drilled in 2001 by the Ensko 51. The fire resulting from the blow out extensively damaged the rig.

No jack-up related blowouts have been recorded in the North Sea, although other types of mobile offshore units have been affected (such as the ODECO Ocean Odyssey semi-sub at the Shearwater Field in 1998). Shallow gas is particularly hazardous when located above the primary casing shoe level, or the conductor pipe shoe level. At these depths, the wells are not drilled under blow out prevention (BOP) control. Seabed cratering could develop around the well in a blow out and undermine the jack-up foundations. Further discussion on shallow gas is given in Section 10.4.

8.2.6 Seafloor Instability

Seafloor instability commonly results in large-scale mass movement, in the form of mudslides or slope failures. Such instability, which is often associated with deltaic deposits, may be manifested as either continued foundation settlements or large-scale failure of the soil mass. Sections 10.1 and 10.2 provide further discussion on geological faults and landslides.

8.2.7 Liquefaction

Liquefaction occurs under cyclic loading, typically in loose saturated sand, when excess pore pressures build up and reduce the effective stresses and shear resistances to very low values. Liquefaction foundation failures can result from

severe storms or earthquakes and can potentially cause large differential lateral and vertical displacements of the spudcans or large-scale failure of the soil mass.

8.3 Accident statistics for jack-up units

Dier et al (2004) present an HSE funded study into current knowledge regarding jack-up safety, finding that one third of jack-up accidents were associated with foundation problems. Figure 5 summarises their analysis of 51 international foundation led incidents. Punch-through failures represent 53% of all foundation accidents. Uneven seabed/scour/footprint interaction, was the next most likely cause, covering 15% of all incidents. Of the six North Sea incidents considered, five were due to seabed instability/scour/footprint interaction, and the sixth punch-through. According Dier et al, there had been no jack-up foundation fatality in the North Sea up to 2004. Jack et al (2007) reviewed more recent trends in international jack-up accident statistics. While advances in several disciplines significantly reduced incident rates and fatalities between the mid-fifties and 2000, they found that improvements have not continued in recent years. They highlight the impact on accident statistics of severe Gulf of Mexico hurricanes and the effects of increasing jack-up water depth capabilities on the reserve strength of jack-up legs. They also note that the international jack-up fleet is likely to expand greatly in future years and that shortages of skilled and experienced personnel may impact on safety.

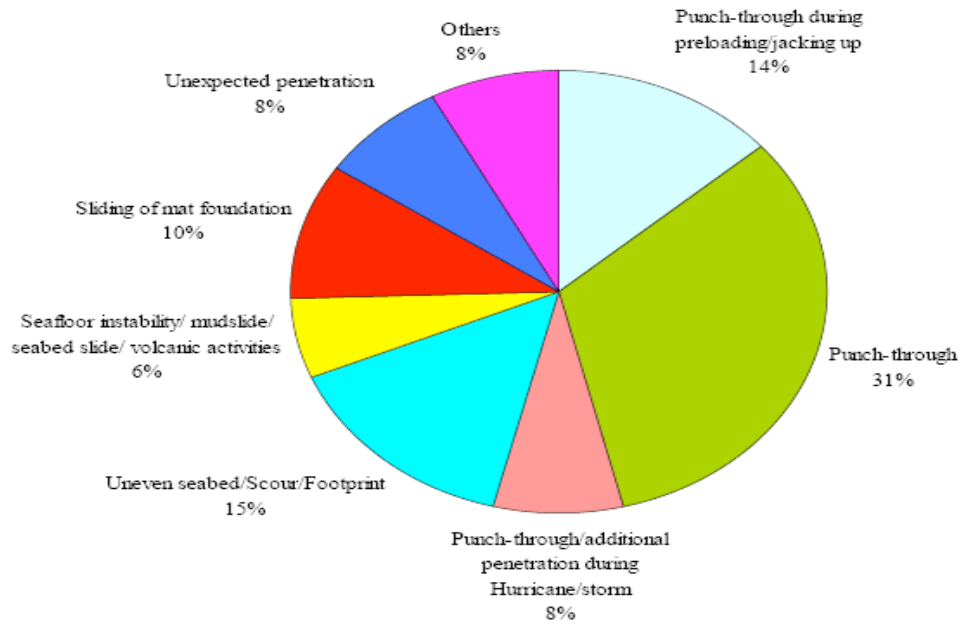


Figure 5. Case histories classified according to the cause of failure, after Dier et al (2004).

Jack et al. (2007) present a recent analysis of over 1,250 international jack-up related losses and severe accidents between 1956 and 2006. Figure 6 presents a summary of the cost impacts due to structural, foundation and jacking system incidents (excluding hurricanes) identified by their study. The trends since 1990 are presented in Figure 7.

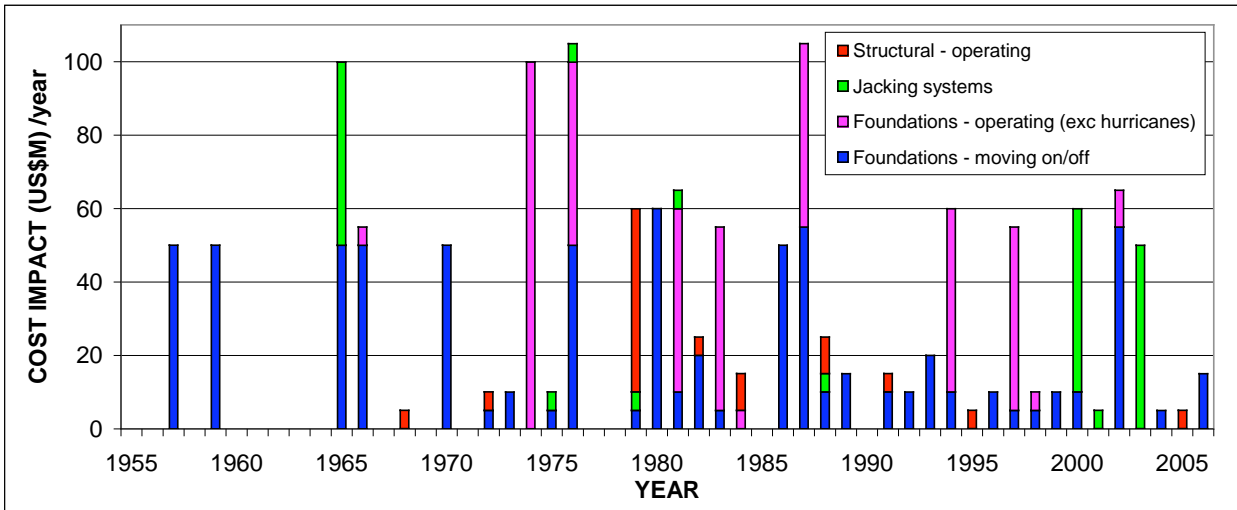


Figure 6. Cost impacts of Jack-up system failures 1955 – 2006, after Jack et al (2007).

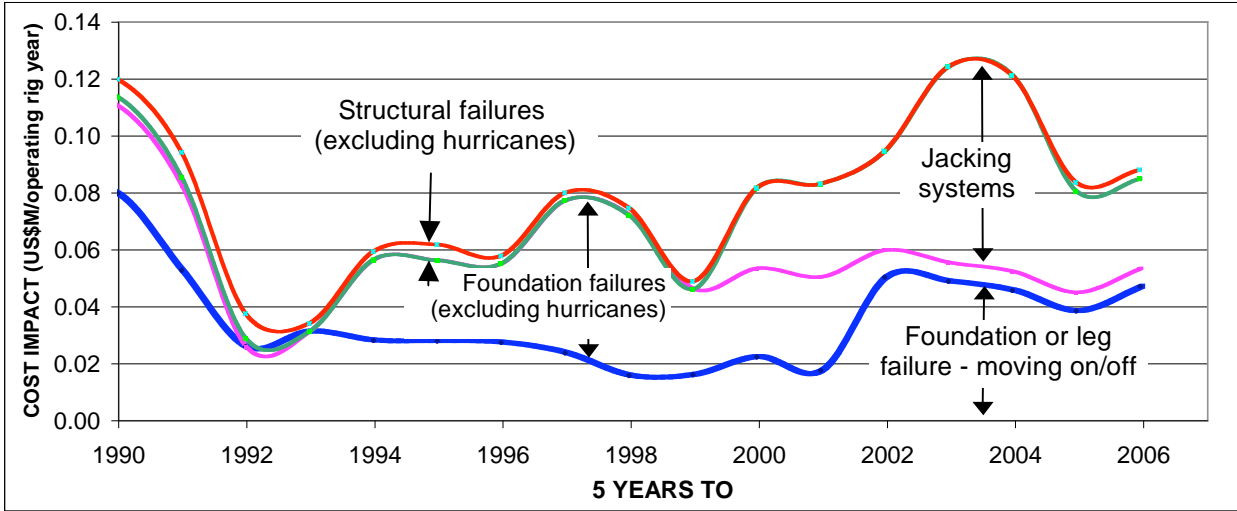


Figure 7. Cost impacts of structural, foundation and jacking system risks 1995 – 2006, after Jack et al (2007).

Figure 7 indicates an increasing trend in cost impact due to foundation failures going on location, which were predominantly related to punch-through failures. Jack et al. (2007), suggest causes for this disappointing trend. One factor is the deployment of the newer jack-up units in greater water depths. Whilst their legs work well under normal operating conditions, they have less reserve strength to resist the greater loads experienced during a punch-through. In addition, these newer rigs tend to be more complex to jack up and down, and require personnel to be trained and experienced in the measurement and control of Rack Phase Differences (RPDs).

Figures 6 and 7 do however highlight that the other modes of foundation failure, which have historically had significant cost impact, appear to have been mitigated in recent years.

8.4 Reliability of foundation assessment

Meyer et al. (2003) summarise an HSE funded study into the foundation reliability afforded to jack-ups in the UKCS area of the North Sea by SNAME T&RB 5-5A. They assumed that pre-loading had been imposed up to the levels calculated by applying the SNAME T&RB 5-5A recommendations with the 50-year extreme loading case and specified load and resistance factors. The foundation performances of the jack-up spud cans installed to the required depths were then assessed under the more extreme 10,000 year joint probability loading condition, but with 'neutral' load and resistance factors set to 1.0. The key conclusion was that general compliance with the T&RB5-5A recommendations should provide sufficient pre-load levels to cope with the un-factored 10,000 year event, without developing overall foundation failure. Preload capacity utilisations (defined as the ratio of the expected maximum vertical load to the pre-load) ranged between 0.91 and 1.00. Foundation bearing capacity utilisations (defined as the ratio of the expected maximum load to the bearing capacity calculated under the expected multi-dimensional loading conditions) were all greater than 1.0, but the leg displacements expected under the expected short duration extreme loading events were considered to be both tolerable (according to the Stage 3 procedures incorporated in SNAME T&RB 5-5A) and leading to a stable expansion of the bearing capacity envelope. ISO 19905, which is to be largely based on SNAME T&RB 5-5A, should provide a similar level of foundation reliability.

8.5 Recent advances in analysis and practice

Recent advances in the analysis and assessment of jack-up foundations have focussed on foundation stiffness, and on how interacting load components affect foundation 'yielding' or failure. The latter involves defining three (or more) dimensional 'general loading yield surfaces' in place of conventional bearing capacity approaches for the assessment of jack-up and Gravity Base offshore foundations, as described below in Section 9.0. Advances made through environmental and rig motion monitoring, and back analysis of hurricane events in the Gulf of Mexico, have

allowed higher soil shear moduli to be applied to model foundation stiffness and fixity (Noble Denton, 2002; Templeton et al, 2005; Templeton, 2007; Cassidy, 2007). Advances have also been made in the selection of bearing capacity factors for circular conical foundations in clay (Houlsby and Martin, 2003) and sand (Cassidy and Houlsby, 2002). However, these factors still require field validation through comparisons with recorded penetrations and conventional predictions made using the SNAME T&RB 5-5A approach. The latter applies conventional strip footing bearing capacity factors together with approximate depth and shape factors.

8.6 *It is recommending that the Duty Holder and/or his consultants appointed for location assessment/approval show proof of appropriate foundation assessment for each particular location, including details of:*

- *Expected spudcan pre-load penetration depth and any potential for rapid footing penetration;*
- *Specific consideration of the potential for punch-through capacity to be affected by sub-layering within the soil deposits, where appropriate;*
- *Foundation loads derived from the storm survival assessment, highlighting any expected foundation over-utilisation, where relevant;*
- *Assessments of relevant potential foundation hazards, as discussed in Section 8.2;*
- *The experience of the person who undertook the assessment, showing suitable competence.*

In cases where new methods of analysis have been adopted, it could be suggested that consideration be given to checks involving existing assessment procedures.

9.0 Recent developments regarding Gravity Base Structure (GBS) foundations

Relatively few GBS platforms have been installed within the UK North Sea Sector, although they are common offshore Norway and Australia and elsewhere. Shallow GBS foundations are generally complex, often including water-tight skirts to boost sliding resistance and under-drainage systems to aid installation and deal with long-

term soil consolidation and gas accumulation processes. They must withstand large environmental loads as well as self weight loads, and horizontal sliding (or combined loading) modes are generally more critical than with piles. Cyclic loading is also considered far more closely; large storm-induced settlements have been recorded on some platforms. More sophisticated site investigation procedures are required (see Section 11) and comprehensive geotechnical and other instrumentation systems are often specified that give invaluable advance warnings of potential problems; Tjelta et al (1992), (2007).

Classical design practice (DNV 1995 or API 2000) includes conventional bearing capacity formulations with factors for inclined and eccentric loading. However, the suitability of these methods may be questioned in cases where the moment and horizontal load components are far more important than in most onshore structures, where vertical loads predominate. The alternative three (or more) dimensional 'general loading yield surface' approaches suggested by Butterfield (1979) have undergone rapid development and are now being applied; Randolph et al (2005). As with jack-up platform foundations, the API and ISO recommendations for shallow foundations are currently under review and seem likely to move towards the general loading surface approaches and away from conventional bearing capacity methods.

Limit equilibrium procedures are also applied in GBS foundation design that can account for important features such as shear strength anisotropy in critical layers. These have also been adapted to consider additional aspects such as strain compatibility and progressive failure, cyclic loading effects and deep 'skirt-pile' configurations; see for example Tjelta et al (1988) and Andersen et al (1988). It is also now common to apply advanced Finite Element analysis and physical model testing to GBS foundations design; Hight et al (1988), Randolph et al (2005), The latter allow designers to check simpler calculations and identify potentially different failure modes, as well as providing predictions for soil stresses and movements and structural forces.

The key points for consideration are :

- *A suitable spread of site investigations and design studies has been undertaken under the supervision of suitably qualified GBS specialists.*
- *Suitable skirts and under-drainage systems are provided to cope with soil consolidation, cyclic loading and any gas migration processes.*

- *A well-designed geotechnical monitoring and data interpretation system is provided in conjunction with measures to allow any potentially necessary future interventions.*

10.0 Geo-hazards, including disturbance to foundations caused by well drilling

Geotechnical and geological conditions on the sea bed can lead to sets of 'geo-hazards' that can impact on the safety of offshore facilities. Features such as scattered boulders, iceberg scour scars, relict pock-mark depressions, jack-up foundation craters, rough landslide morphology or buried valleys can cause difficulties when attempting to install fixed facilities. They are best identified by geophysical surveys and either avoided, removed or addressed fully in design. Other potential geo-hazards are discussed below under five sub-headings.

10.1 Active geological faults

Faults may generate steady creeping displacements, or seismic shocks. Tectonic conditions are relatively stable in the North Sea, but wherever such faults are identified from geological analysis, consideration should be given to re-location, or to adopting resistant design. *Small earthquakes do take place in the North Sea and seismic monitoring databases exist from which assessments may be made of earthquake hazard levels in particular areas, and the levels of shaking that might have to be addressed in critical cases.*

10.2 Landslides.

Existing landslides can often be recognised from geophysical surveys that can show the seabed morphology (sidescan sonar) and identify any distortion to the in-situ sediments (high resolution reflective surveys). Seafloor conditions are often practically flat and geotechnically stable in the much of the UK continental shelf area, where oil and gas production has tended to concentrate. However, areas of slope instability are encountered in some of the deeper water fields that are now being considered on the sloping shelf margins. The most dramatic examples are found in Norwegian deepwater developments such as the Ormen Lange Field, which sits in

the backscarp area of the massive Storegga slide. As set out by Brynn et al (2002), special geotechnical investigations and analyses are required to assess whether such landslides pose hazards for the developments, or to society at large through the possible development of highly destructive tsunami events. Smaller scale mudslides involving sloping areas of soft soils in relatively shallow water are known to have been triggered by hurricanes in the Gulf of Mexico, causing substantial damage to offshore structures in recent years. While comparable events seem unlikely in the North Sea, considering the potential for landslides is an important part of any geo-hazard risk assessment.

10.3 Diapirism

Geological conditions can exist that force domes of either weak mud or creeping salt to emerge from considerable depths below the seabed up towards the surface. These processes, known as diapirism, can lead to considerable geo-hazards. Salt diapirs are relatively slow moving, but they can cause slope features to form that become unstable, as for example along the Sigsbee escarpment in the Gulf of Mexico. Mud diapirs lead to areas of very weak seabed conditions and the mud produced from them can also cause considerable difficulties in some offshore provinces. Problems associated with diapirism are uncommon in the North Sea.

10.4 Shallow gas

Shallow gas poses a far more serious hazard than diapirism in the UK Sector. Gas seeps, possibly rising from deeper reservoir areas or shallower source bodies, can collect in sand strata capped by clays, and may find pathways to the surface that fracture through the clays if sufficient gas pressures develop. Drilling into such deposits can lead to catastrophic 'blow-outs' that can result in the total loss of platforms or drilling units. Shallow gas can often be identified through high resolution reflective geophysics. In-situ testing tools such as the BAT probe designed by NGI can be employed to assess gas concentrations in layers where shallow gas is considered a possible hazard. Other features may be associated with prior or current gas, water or mud expulsion to the surface. These include sizeable 'pock-mark' craters and cracks in the sea bed. *The presence of active features is indicative of possible shallow gas and such areas do not provide good locations for fixed structures.*

10.5 Potential effects of pile disturbance by drill-drive, adjacent well drilling and other activities

Drilling or other sub-surface operations can affect pile capacity in unintended ways. As noted in Section 6, pre-drilling, jetting and vibration applied to ease pile driving downgrade axial capacity. Other forms of disturbance can affect service life. Hobbs and Senner (1997) reviewed potential effects of drilling and conductor installation on foundation Safety. Problems with 'running sands' are perhaps the most common source of well drilling disturbance, although other problems can arise including the 'packing-off' in collapsing clay wells and gas seeps along imperfectly sealed conductions. Schroeder et al (2007) report how hydraulic fracturing associated with 'packed-off' conditions in wells drilled in collapsing clay reduced the axial capacity of nearby conductors dramatically at one site, and may have posed a hazard to nearby foundation piles. Their advanced FE analyses indicated that while the influence on the piles' axial capacities may have been modest, the associated ground movements could have been more significant, and a full soil-structure interaction was carried out to assess the status of the particular fixed facility.

Observations were made for this critical case of the ground's response to drilling disturbance through new sampled borings, with laboratory testing and in-situ probing and pore-pressure measurements. Foundation behaviour was also checked by monitoring ground movements. *Similar steps should be considered in any potentially critical case.*

11.0 Site assessment procedures for a range of applications

11.1 General

Site investigations should be designed to suit the particular foundation project, the probable geomaterials present and the regional setting. For example, deepwater project studies often involve a limited programme of borings and soundings because of their practical difficulties and costs. In the same way, the investigation scope will change if the project area is subject to any significant geohazards, as outlined in Section 10. The API/ISO group for offshore fixed structures is starting work in 2007

on a proposed international guidance document covering site investigation and testing procedures for fixed offshore platform projects.

Site assessment procedures are well developed in the North Sea, where a range of good quality facilities are available. Different approaches and techniques are generally adopted for piled platforms, gravity base units, jack up platforms (or Mobile Offshore Drilling Units, MODUs), suction anchors, well templates or sub-sea completion units. Desk studies and geophysical surveys are crucial in assessing the regional geology and local stratigraphy. 3-D geophysical data are often available from the exploration stage of any oil or gas project, and these can provide invaluable initial assessments, along with any shallow exploration drilling data that might be available or exploration rig site-survey data. Higher quality 2-D data-sets, or very high quality information from Autonomous Underwater Vehicles (AUVs) will provide better resolution data for use in geotechnical assessments.

The sub-sections below concentrate principally on the geotechnical assessment activities required to design piles driven in North Sea sands and clays, reflecting the main focus of the document. However, brief additional comments are offered on other relevant geotechnical foundation investigation issues.

11.2 Site Investigations techniques

The site investigation process starts with a desk review of available data, including inspection of geophysical profiling and the possible commissioning of new survey data. As reviewed in Part 3, one or more boreholes would normally be required for a new platform; severe problems have arisen in cases where this practice has not been followed. The number of borings will depend on the scale and complexity of the platform and the geotechnical setting. The borings usually involve a mix of high quality sampling and CPT profiling. CPT profiles can be established to modest depths by using independent units mounted on the seabed, but boreholes are required to undertake CPT probes to the full depths of most offshore piles. Ideally, separate boreholes will be made for sampling and CPT testing, particularly when the design approach relies on a continuous CPT trace, as with the 'modern' approaches set out in the 2007 API-RP2A recommendations. However, in cases involving small lightweight unmanned structures it may be common to have a single borehole, especially when the regional conditions are relatively well understood. In many

investigations, piezocones are employed that gather additional information concerning the ground's pore-water pressure response.

Standard 'WISON' or other down hole techniques are applied to advance CPT devices by up to 3m per stroke, with drilling out taking place between each push. The standard jacking equipment allows cones to be advanced in soils with CPT resistances q_c of up to 90 MPa, provided high capacity cones are employed. Smaller diameter cones with still higher ratings may be used in very dense sands and cemented soils, although the results may be treated with caution in pile capacity assessment.

Sampling usually involves hydraulically pushed, thin walled, smooth stainless steel tubes, typically with a length of 900mm and a diameter of 72mm. In soft soil deposits a fixed piston may be deployed to reduce disturbance and a sharp cutting edge (less than 10° taper) may also be specified to advantage. Rotary coring is the preferred approach with soft rocks such as chalk; percussively taken samples are likely to be massively disturbed. SPT testing is almost never preformed offshore in the UK North Sea Sector, but is performed in some other regions.

11.3 Soil testing required for pile design

Investigations for piled fixed structures should establish the detailed geotechnical profile, down to a depth exceeding the possible pile penetrations. The profile should include full sample descriptions, unit weight determinations, water contents, Atterberg limits, and some particle size distributions. It is common for index strength tests to be performed in clays with Torvanes or pocket penetrometers. Chemical tests or microscope analyses may also be made in cases where 'special' soils involving carbonate, mica or other contents may be suspected. Effective stress triaxial or shear box tests are also commonly undertaken.

In order to apply the standard API-93 pile design procedures requires sufficient testing to define:

- A secure profile of Unconsolidated Undrained triaxial shear strength (S_u) measurements made on good quality samples, with remoulded S_u values being determined to aid driveability studies. These are often supplemented by CPT data.

- Profiles of relative density from CPT or other in-situ tests. The upper limits to relative density stated in the RP2A Main Text method give little incentive to continue CPT jack strokes once q_c values exceed 50MPa, so tests are often terminated prematurely once this 'max-out' value is reached.

Additional parameters are required when considering the new CPT based approaches for sands. Firstly, it is more important to develop a full CPT q_c profile including the measurement of any values in excess of 50 MPa in very dense sands, as described above. Recording such data can also help in avoiding pile refusal and buckling problems during installation. It is also important to assess the pile-soil interface shear resistance. Ideally this should be through site specific laboratory interface ring-shear tests. Soil grading curves may also be used when such data are not available.

Considering clays, the following additional data are required in order to apply the ICP-05 approach:

- A good assessment of sensitivity, ideally involving both remoulded and peak S_u measurements as well as intact and reconstituted oedometer tests.
- Reliable estimates of Yield Stress Ratio. The latter is best obtained through a holistic interpretation of geology, oedometer test yield points, high quality triaxial S_u/σ'_{v0} data and profiles from CPT or other in-situ test types.

These data are not always available, even for well established test sites, making database assessments of the ICP-05 clay approach harder to achieve than is the case for the sand method. Part 3 discusses YSR assessment from undrained triaxial tests, which can be difficult in clays that bifurcate during shear, or are susceptible to sampling disturbance. Jardine et al (2005) recommend that CAU triaxial compression procedures should be followed if possible when applying the ICP for design in clays, along with high quality push sampling.

In cases where cyclic loading analyses are required, undrained triaxial or simple shear tests may be specified that cover an appropriate range of cyclic loading levels to allow assessments to be made of potential degradation effects in both sands and

clays. In the same way, additional information from locally instrumented ‘small-strain’ triaxial tests, bender-element shear wave velocity measurements, or resonant column experiments is required if non-linear finite element analyses are to be undertaken to provide accurate load-displacement behaviour predictions, including dynamic foundation response. Part 3 provides further references to these more advanced geotechnical tests.

11.4 Soil testing for Gravity Base Structure (GBS) foundations

GBS site investigations generally employ similar field techniques and profiling measurements to piled foundations. However, different patterns of laboratory testing are usually specified. Greater consideration is given to establishing the following additional features:

- The potential anisotropy of undrained shear strength is usually investigated by performing Triaxial Compression (TC), Triaxial Extension (TE) and Direct Simple Shear (DSS) tests on clay specimens that have been consolidated under K_0 conditions to a suitable range of initial conditions. This may also be carried out for any sand layers that could be brought to failure under undrained conditions under storm loading.
- Cyclic TC, TE and DSS tests may be performed on the critical layers to establish parameters for an assessment of potential degradation and storm induced ground movements.
- A greater emphasis is placed on measuring soil shear stiffness and compressibility, along with the geomaterials’ permeability and consolidation characteristics. These aspects are critical to assessing the platform’s in-service, long term, settlement behaviour and dynamic response characteristics.

While GBS structures remain uncommon in the UK North Sea Sector, the site investigations and geotechnical analyses undertaken to aid their safe design are generally more advanced and comprehensive than those required for piled structures.

11.5 Soil testing for Jack-up foundations

Detailed site-specific geotechnical and geophysical information is essential for the assessment of jack up foundations. Guidance provided for jack-ups by SNAME T&RB 5-5A and Noble Denton Guideline 0016 is summarised below.

11.5.1 Geophysical surveys:

Four types of geophysical survey are routinely undertaken for jack-up site assessments. *Bathymetric surveys*: are required covering an area of approximately 1 km square centred on the proposed location. Survey line spacing should be typically no greater than 100 metres x 250 metres. *Seabed surface surveys*: are performed over similar plan areas utilising side scan sonar, swathe bathymetry and high resolution echo sounder techniques. The resolution should be sufficient to identify obstructions and seabed features. *Magnetometer surveying* is also required to identify any buried pipelines, cables or other metallic debris located at or below the seabed. Seabed surface surveys can become out-of-date rapidly, particularly in areas of construction/drilling activity or areas with mobile sediments. As a general rule, re-surveying is required after six months. *Sub-bottom profiling surveys*: reveal the general near-surface geological structure of the sea bed by identifying reflectors that may pick out the sequence of sediments and possibly identify changes in soil characteristics. Their interpretation requires essential correlation of the seismic data with nearby geotechnical borehole data. Shallow seismic data may also reveal any areas containing shallow gas. The seismic acquisition equipment applied for jack-up surveys should be capable of providing detailed information to a depth of at least 30m below seabed, or the anticipated footing penetration plus 1.5 times the footing diameter.

11.5.2 Geotechnical investigations:

Site specific geotechnical data, generally involving one or more sampled and tested boreholes, are required for each jack-up location. In certain circumstances a borehole may not be required if there is sufficient historical data and/or geophysical tie lines to boreholes in close proximity to the proposed jack-up location. The number of boreholes required at a site should account for the lateral variability of the soil conditions, regional experience and the geophysical investigation. When a single borehole is made, the preferred location is at the intended centre of the leg pattern. The geotechnical investigation should comprise a minimum of one borehole to a

depth equal to 30 metres, or the anticipated penetration plus 1.5 times the spudcan diameter, whichever is greater. The investigations should allow the geotechnical properties of all layers to be known with confidence. As with fixed structures a combination of in-situ CPT profiling and laboratory tests on good quality samples is often preferred. For spudcan penetration analyses the site investigation report should include as a minimum:

- Profiles of undrained shear strength (S_u) with depth for clay strata
- Effective stress shear strength parameters for sands
- Ideally continuous Piezocone penetration test (PCPT) records for all strata
- Appropriate soil classification tests including Atterberg limits, water contents, particle size distributions, unit weights, relative densities, sensitivity, etc.
- Over-consolidation ratio (OCR) profiles for fine-grained soils, particularly where foundation fixity is an issue.

In cases where more comprehensive cyclic or fixity analyses are required additional laboratory testing may be specified to determine the cyclic/dynamic behaviour and non-linear shear stiffness characteristics of the foundation soils.

11.5.3 Evidence of a sufficiently detailed site-specific site investigation, as outlined above, and that a competent geotechnical person has been responsible for interpreting the data and determining the key design soil parameters, is required..

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PART 2

Abbreviations and symbols adopted

Abbreviations

API	American Petroleum Institute
AUV	Autonomous Underwater Vehicles
CAU	Consolidated Anisotropically, sheared Undrained (triaxial and DSS tests)
CIRIA	Construction Industry Research Information Association
CPT	Cone Penetration Test
DNV	Det Norske Veritas
DSS	Direct Simple Shear test, (usually CAU, see above)
FE	Finite Element
FM	Fugro McClelland
GBS	Gravity Base Structures
GCG	Geotechnical Consulting Group
EPSRC	Engineering and Physical Sciences Research Council
HSE	Health and Safety Executive
ICE	Institution of Civil Engineers
ICP	Imperial College Pile
ICON	Imperial College Consultants
INPG	The Polytechnical University of Grenoble, France
ISO	International Standards Organisation
MTD	Marine Technology Directorate (now defunct)
NGI	Norwegian Geotechnical Institute
OCR	Over Consolidation Ratio (ratio of maximum past σ'_v to current value)
P-Y	Horizontal local-lateral displacement relationship
SRD	Soil Resistance to Driving
SUT	Society for Underwater Technology (UK group which convenes a committee on Offshore Site Investigations and Foundation Behaviour and organises Offshore Geotechnics Conferences)
TC	Triaxial compression test (usually CAU, see above)

TE	Triaxial extension test (usually CAU, see above)
T-Z	Shaft vertical shear stress-local vertical displacement relationship
TPD	Technical Policy Document
UWA	University of Western Australia
UU	Unconsolidated Undrained triaxial test
YSR	Yield Stress Ratio, apparent OCR (see above)

Symbols

D	Pile outside diameter
I_p	Soil plasticity Index
K	Earth pressure coefficient = σ'_r / σ'_{v0}
L	Pile embedded length
q_c	CPT cone resistance
q_b	Pile end-bearing resistance
Q_c	Calculated pile axial capacity (applied to shaft, base or total)
Q_m	Measured pile axial capacities (applied to shaft, base or total)
$Q_{cyclic}/Q_{max\ static}$	Normalised cyclic shaft load amplitude
$Q_{average}/Q_{max\ static}$	Normalised average shaft load under cyclic loading
s	Pile group centre-to-centre spacing
S_u	Undrained shear strength
t	Pile wall thickness
α	Pile shaft adhesion factor = τ_f/S_u
δ	Pile-to-soil effective stress interface shear friction angle
ϕ'	Effective angle of soil shearing resistance
σ'_r	Radial effective stress
σ'_v	Vertical effective stress
σ'_{v0}	Undisturbed, free field vertical effective stress
τ_f	Vertical shear stress on shaft at failure

PART 3

Keynote paper by Jardine and Chow (2007) summarizing recent developments.

Some Recent Developments in Offshore Pile Design, RJ Jardine, FC Chow; Proceedings of the 6th International Site Investigation and Geotechnics Conference: Confronting New Challenges and Sharing Knowledge, 11 – 13 September 2007, London UK

SOME RECENT DEVELOPMENTS IN OFFSHORE PILE DESIGN

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Abstract

This paper reviews the research and debates that have led to substantial changes being made in 2007 to the API-RP2A recommendations for assessing offshore driven pile axial capacity. The reasons for the conventional MainText method's large scatter, strong skewing and significant biases are explored, with particular emphasis on piles driven in sand. Recent alternative design frameworks are reviewed critically and conclusions are drawn regarding their practical application. Comments are also made on predicting load-displacement behaviour, assessing the impact of load cycling, group interaction effects and aspects of foundation disturbance by drilling.

1. Introduction

The technologies associated with the manufacture and installation of offshore piles are relatively mature; very large piles may now be driven routinely in a wide range of water depths and geotechnical settings. However, the understanding of the ground's reaction to driven pile installation and loading has lagged behind the impressive developments made by the offshore construction industry, as design approaches are still in an imperfect state of evolution. Severe problems have arisen during pile installation in some major projects¹. Considerable mismatches have been found in other cases where it has proved possible to check Industry-standard design expectations by static tests on large offshore scale piles^{2, 3, 4}.

Research in several centres has emphasised the scientific weaknesses of the industry-standard American Petroleum Institute (API) RP2A⁵ methodologies, which have remained practically unchanged between 1993 and 2007. While most practitioners have continued to use the conventional methods, alternative geotechnical design frameworks have been proposed that have been applied comprehensively in some sectors⁶. Vigorous debate has taken place over several years, prompted by industry-sponsored reports, academic papers, conference proceedings and meetings of the relevant API/International Organization for Standardization (ISO) review panels. Important changes are included in the 2007 API-RP2A recommendations for piles driven in sand that will also affect the ISO documents and industrial practice. However, progress is being made cautiously and further evolution of design practice can be expected.

This paper offers one perspective on some of the issues raised in the recent debates, referring to background research and highlighting physical aspects of pile behaviour that are important to practice. Particular emphasis is placed

on the question of axial capacity, as this is arguably the most important issue and has proved to be the focus for most discussion. Consideration is also given to the assessment of load displacement behaviour. While movement prediction and control is emphasised more strongly in onshore foundation projects, as reflected in the recent review by Mandolini *et al.*⁷, offshore engineers may become more concerned with making better fatigue life predictions for critical structures. Assessing limits for acceptable displacements also could become more important as a means of monitoring platform safety in critical cases.

It is useful to consider at the outset the ranges of pile sizes specified by offshore engineers. Piles with diameters of ~5m have been driven for offshore wind turbines. Smaller diameters of 3 to 4m have been specified for such structures in the North Sea, where piles with diameters of up to 2.5m have been driven routinely to depths of 100m or greater to support large fixed oil and gas platforms. In this offshore domain, pile diameter to wall thickness ratios, D/t of between 15 and 45 are typical (with an average around 27), although more slender ratios have been used elsewhere. For example, the primary piles that experienced buckling failures during installation at the Goodwyn field (North West Australia) employed a D/t ratio of 60⁸. Adopting high wall thicknesses may necessitate special stress relieving treatment for the pile welds, making diameters much greater than 2m potentially less attractive economically when working with D/t ratios lower than ~40⁹.

The review given by Overy⁶ on Shell UK's North Sea piling operations shows a trend for platforms designed (since 1996) in this now mature province is to employ mid-sized piles (0.660 to 2.134m diameter, with 26 to 87m penetration), although their rated axial compressive capacities remain substantial (14 to 100MN).

2. Uncertainty in Main Text API-RP2A Offshore Pile Axial Capacity Calculation Procedures

While pile load tests are usually prohibitively expensive to perform offshore, pile testing to failure is frequently carried out on smaller onshore piles. Tests are usually conducted within a few days or weeks of driving as part of project quality assurance or for research purposes. Substantial databases can be assembled to test existing predictive approaches and suggest ways in which they can be improved. Such studies have been crucial to the evolution of the API and ISO recommendations for driven offshore piles, leading to detailed guidance for pile design in (primarily siliceous) sands and clays of various strengths.

The interpretation of the databases can be hampered by complexities in the site geotechnical profiles, missing site investigation data, variations in pile testing methods or uncertainty in the history and accuracy of reported measurements. Applying quality filters leads to data sets containing typically tens, rather than hundreds, of tests being available. Simple statistics can be applied to check for the average trends, degrees of scatter expressed as coefficients of variation (CoV) and possibly skewed biases associated with particular design approaches. However, it is becoming clear that many factors can affect axial capacity, and the data sets are generally too small to allow reliable statistical regression investigations that can cover a sufficiently comprehensive range of influential variables. Advances are typically made by developing hypotheses from theory, model experiments or reports of specific field behaviour, and then testing these against the available test databases.

Considering the API-RP2A⁵ recommendations for pile capacity calculations, database studies have shown that the uncertainties, expressed as CoVs of Q_d/Q_m (the ratio of calculated to measured pile capacities) are greater in sands than in clays^{10,11,12}. Table 1 summarises the statistics reported in Jardine *et al.*¹² for the API-RP2A⁵ recommendations (termed API-93 from here onwards), covering a filtered database of 81 piles driven in sand and 68 piles in clay. Open-ended piles comprise just over 40% of the combined data set, while sufficient information existed to decouple the base and shaft capacity components of compressive capacity in just over half of the cases. So for example, the subset of tests involving end bearing measurements for open-ended piles driven in sites dominated by sand amounts to 20 mainly uncoordinated tests.

Smith *et al.*¹³ showed that the reliability factors associated with CoVs of the order cited in Table 1 are poor when

Table 1: Summary of statistical analysis of API-93 pile axial capacity approach tested against a database of 149 piles¹²

	Mean Q_d/Q_m	CoV in Q_d/Q_m
Shaft Resistance: Sand	0.87	0.60
Base Resistance: Sand	0.83	0.73
Shaft Resistance: Clay	0.99	0.33
Base Resistance: Clay	1.06	0.98

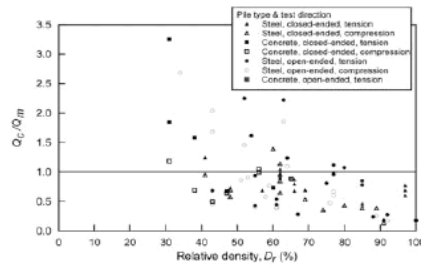
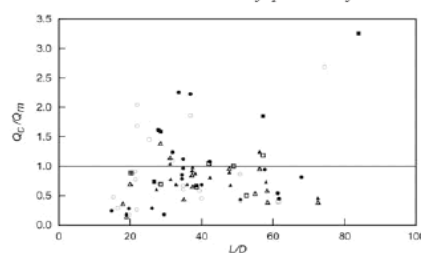


Figure 1: Distribution of Q_d/Q_m with respect to relative density, D_r ; API-93 shaft procedure for sands¹²

combined with currently recommended working stress design (WSD) factors of safety (FoS) or load and resistance factor design (LRFD) parameters. It appears hard to justify the status quo on the basis of pile test database assessments, particularly in the case of piles driven in sand. Nevertheless, some practitioners argue that reports of inadequate offshore pile foundation performance in service are rare, suggesting that current practice is in fact fit-for-purpose. The latter observation could be rationalised if one or more previously unrecognised factors or biases that led systematically to field conditions being more benign than expected could be identified. Potential explanations might include any tendency for environmental loading to be less severe in service than expected, or for long-term (or in-storm) pile capacity to be higher in the field than anticipated in design.

The database review by Chow¹¹, updated by Jardine *et al.*¹², identified several significant biases and skews in capacity assessments made with API-93. Examples are shown in the scatter plots presented in Figures 1 to 6. The clearest single trend was for shaft capacity predictions for sand to be skewed with respect to relative density (Figure 1), being systematically conservative in very dense sands and vice versa with loose deposits. Another bias was evident with regard to pile length to diameter ratio, L/D , in Figure 2. A tendency for Q_d/Q_m to trend upwards with L/D was evident, in addition to a global under-prediction (by 17%) of average shaft capacity, making the method conservative for relatively short piles and potentially non-conservative for slender (high L/D) piles.

Figure 2: Distribution of Q_d/Q_m with respect to length to diameter ratio, L/D ; API-93 shaft procedure for sands¹²



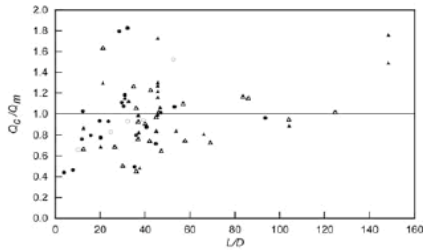


Figure 3: Distribution of Q_c/Q_m with respect to length to diameter ratio, L/D ; API-93 shaft procedure for clays²²

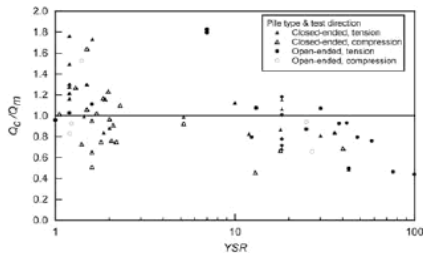


Figure 4: Distribution of Q_c/Q_m with respect to yield stress ratio (YSR); API-93 shaft procedure for clays²²

A similar trend with L/D was seen for shaft capacity in clays (Figure 3) but without any shift in mean Q_c/Q_m . API-93 also showed bias with respect to apparent over-consolidation ratio (OCR) (termed yield stress ratio, or YSR, here to include allowance for any features of cementing or ageing that add to simple stress history factors), apparently showing a tendency to over-predict capacity in near normally consolidated (low OCR or YSR) clays; see Figure 4. The Norwegian Geotechnical Institute (NGI) databases study for piles in clay¹⁴ identified an additional potential bias in Q_c/Q_m for shaft resistance in low plasticity, low OCR clays that was not identified by Chow¹¹ or Aldridge¹⁵ in earlier database investigations.

Problems were also noted in the API-93 predictions for pile end bearing in sands. Figure 5 confirms that the tendency for shaft Q_c/Q_m to fall with relative density also applies to end bearing, while the Figure 6 indicates an apparently strong trend for base capacity Q_c/Q_m to rise with pile diameter. Other studies have led to broadly similar conclusions on most of the aspects highlighted in this section for piles driven in sand^{4, 16, 17}, although controversy exists over the tendency reported in Figure 6 for closed- and open-ended piles to show similar trends with respect to pile diameter (this feature will be discussed later).

It is important to note that any underlying trend for systematic skewing with respect to a key soil or pile parameter renders the average and CoV Q_c/Q_m parameters for that method sensitive to changes in the composition of the test databases. Given the sparse database populations, adding

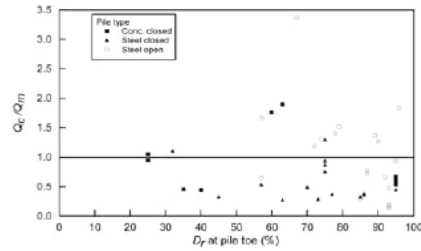


Figure 5: Distribution of Q_c/Q_m with respect to relative density, D_r ; API-93 base capacity procedure for sands²²

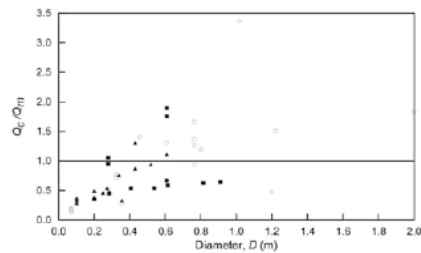


Figure 6: Distribution of Q_c/Q_m with respect to pile diameter, D ; API-93 base capacity procedure for sands²²

just a few new tests on slender piles driven in loose sand pushes the average Q_c/Q_m value for the API shaft method up significantly, and vice versa. In a similar way, excluding three 'problem clay' sites eliminates the 'plasticity index' bias noted by Karlsrud *et al.*¹⁴ for low YSR clays in the NGI's data set of 49 tests.

3. Developments Regarding Axial Capacity Assessment

Recognition of the poor reliability of the API-93 procedures has led to research at several centres over an extended period. The authors and their colleagues contributed through a long campaign of field experiments with the highly instrumented Imperial College Piles (ICPs), combined with various laboratory and theoretical studies^{11, 18, 19, 20}. After producing a series of interim discussion papers that described the implications of the research findings for design^{21, 22, 23}, Jardine and Chow²⁴ produced a comprehensive report setting out a practical alternative approach that became known as the UK's former Marine Technology Directorate (MTD)-96 method. This report used a substantial database of pile load tests to demonstrate that their approaches eliminated the biases and skewing of the API methods discussed previously. Jardine *et al.*¹² reported on the method's application and development over the decade that followed, updating it – with the benefit of field experience – with new research findings and an expanded test database. Given that the MTD ceased to operate in the late 1990s, the updated approach was termed the ICP-05 method.

MTD-96, which has been adopted as the primary design tool in some sectors⁶, has also been subject to considerable debate and commentary in recent years^{25, 26, 27}. It is worthwhile revisiting some of the key aspects of the method's development and putting these in context before considering some of the critiques made and derivatives, or alternatives, proposed. The latter are now impacting on practice, have influenced the 2007 revisions to API-93 and will, in turn, be recognised in the ISO and Eurocode recommendations for offshore driven piles.

4. ICP Method Development for Sand

The MTD-96 and ICP-05 approaches for piles driven in sand will be considered first and then the approaches for clays will be discussed, to make some links to inter-related issues at earlier stages where it will be helpful.

4.1 The direct use of cone penetration test (CPT) data in sands

Sand state has a crucial role in defining the capacity of pile driven sands²⁸. A key aspect of the ICP is the direct use of cone penetration test (CPT) q_c values to characterise *in situ* changes in sand state with depth. While this had been implicit in civil engineering practice in several European countries for some time²⁹, the ICP field tests performed at Labenne and Dunkirk showed for the first time that the local radial effective stresses, σ'_{rc} , and pile base resistances rose and fell synchronously with q_c ^{11, 30}. Incorporating direct links with q_c practically eliminated any skewing in database shaft and base capacity predictions with respect to relative density.

4.2 The importance of the relative pile tip depth

The second most important aspect of the ICP is allowing for a marked reduction in the radial effective stress, σ'_{rc} , developed in any particular soil horizon as the pile tip advances below that horizon. The dependence of σ'_{rc} on the relative depth of the pile tip, h , as defined in Figure 7, was observed in the early pile tests conducted in London Clay

at Canons Park^{18, 19} and later confirmed at other clay sites. However, the reduction of σ'_{rc} with h was found to be most marked in sands, with broadly similar trends being seen in both loose and dense deposits^{11, 30}. This feature, illustrated in Figure 8, explains the skewing with respect to L/D shown by all classical methods (including API-93 see Figure 2) and also the paradox of the 'critical depth' reported by Vesic³¹ and others. Below the 'critical depth', the average shear resistance acting over a pile shaft remains apparently constant with increasing depth in uniform sands, rather than increasing steadily as expected by the conventional simple analogy with earth pressure theory.

The associated trend for local shaft shear resistance, τ_{zs} , to decline with pile penetration during driving had been recognised earlier from pile installation records in clays and described as a 'friction fatigue' process by Heerema³² for North Sea clays. Toolan *et al.*³³ made some allowance for such a process in their 'sliding triangle' approach for sands, but this was not tested against a large database or incorporated into routine pile design practice. Bond¹⁹ and Lehane²⁰ investigated whether cavity expansion and strain path analyses could offer insights, as well as recognised the importance of load cycling during installation. Chow's¹¹ schematic summary of possible sources is reproduced in Figure 9; she noted that the free surface effects (a) could play a role with closed-ended piles down to perhaps $20D$ (in clay, less in sand), while driving 'whip' (b) might affect the upper four diameters. However, the effects illustrated as (c) and (d) were considered the most important. The tendency for the radial stresses set up by installation to be highly concentrated close to the pile tip and decay rapidly with vertical distance, h , above the tip level as a result of (c) the geometry of the steady flow system around the tip and (d) the cyclic loading imposed by jacking (or driving) was clear in the Labenne and Dunkirk tests.

Very interesting additional data were gathered in an experi-

Figure 7: Definition of relative pile tip depth parameter, h , a key controlling parameter for radial effective stresses acting on pile shaft?

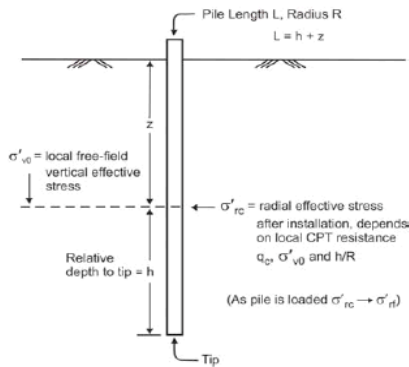
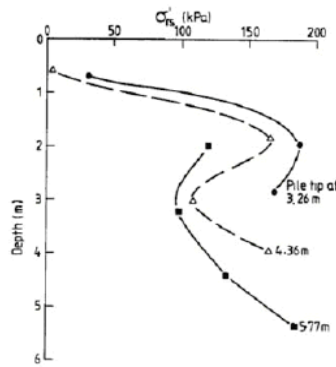


Figure 8: Profiles of radial effective stress developed on shaft of ICP tests at Dunkirk showing combined influences of variations with depth of CPT profile q_c and pile tip depth h/R



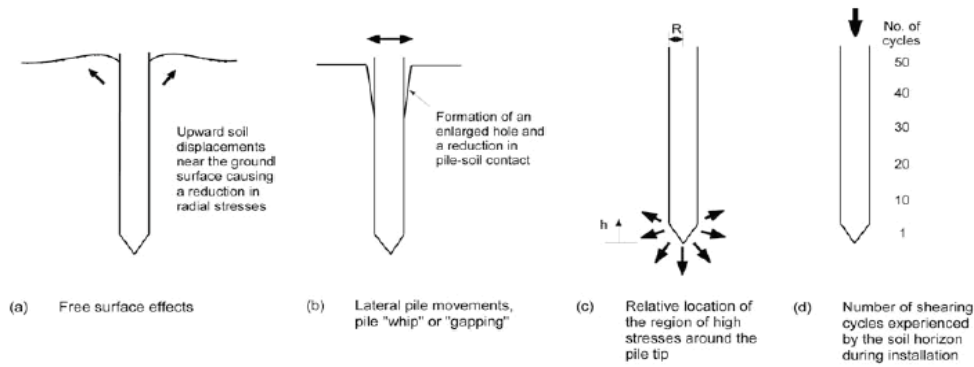


Figure 9: Potential factors leading to decay of radial effective stress acting on shaft with relative pile tip depth, h/R

ment conducted by Chow³⁴ at Dunkirk, in which four levels of radial and shear stress cells mounted on a pre-installed ICP pile, DK2, were used to monitor the effect of installing another 100mm-diameter pile, DK2b, at a centre-to-centre spacing of 450mm from the ICP. Figure 10 illustrates some key results, where h_b is the height of the tip of DK2b above any given instrument level on DK2. The instrument traces show that the most important changes in the stresses developed (at any particular depth) within the surrounding mass take place as the pile tip passes from 20 pile radii above the level in question, to 10 pile radii below. However, the transducers mounted on penetrating ICP piles showed that the local σ'_{rc} and τ_{rc} values stresses continued to decline on the pile surface after larger values of h/R . These continuing changes can be seen as evidence of local cyclic action, as sketched in Figure 9(d). As discussed by Jardine^{35,36}, degradation is likely to be concentrated in the interface zone close to the pile shaft, with σ'_{rc} falling as the number of cycles, N , applied (during jacking or driving) increases.

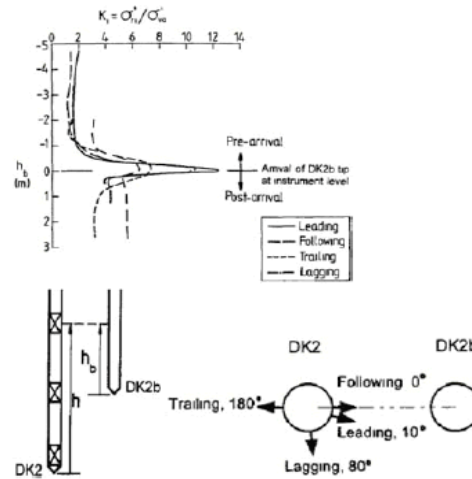
Small displacement cyclic loading tests conducted with the ICP showed that losses in local shear capacity should be expected under the extreme cycles associated with jacked or driven installation, in both clays and sands. This posed the question as to whether the absolute number, N , of jack strokes, or pile blows, would have to be addressed in assessing capacity. Thousands of blows are usually required to install a driven pile, while the ICP piles were usually installed with less than 100 jack strokes. However, a database study reported by Lehane and Jardine²³ indicated that σ'_{rc} expressions fitted to the jacked ICP dataset from Labenne offered reasonable predictions for driven closed-ended piles, suggesting that such variations in the absolute number of cycles would not render the ICP results inapplicable. It appeared possible to consider both the geometrical spreading and the cyclic degradation as varying principally with h/R for closed-ended piles, although this had to be subject to further database testing. Concern over the effects of N on the pile stress regime led Chow¹¹ to include a systematic study into the effects of N into her campaign of ICP tests conducted in the Pentre clay-silt.

Lehane²⁰ proposed from his ICP tests in loose dune sand at Labenne a simple expression for the radial effective stress developed at any particular depth. Chow¹¹ adopted the same form and, taking into account her ICP and open-ended experiments in dense sand at Dunkirk, proposed the modified equation 1 for more general application (where P_a is the atmospheric pressure):

$$\sigma'_{rc} = 0.029 q_c (\sigma'_{v0}/P_a)^{0.13} (h/R^*)^{0.38} \quad (1)$$

With closed-ended piles, R^* is simply the pile radius. Figure 8 presents σ'_r data from one of Chow's closed-ended ICPs at Dunkirk, considering three stages in its installation. The stresses developed are generally several times higher than those found at similar depths at Labenne and fluctuate with

Figure 10: Stress changes experienced at four levels on an ICP pile, DK2, as a second pile, DK2b, is installed offset at a centre-to-centre distance of 4.5 diameters. Note (i) in the plots given for each cluster, h_b is the depth of pile DK2b's tip below the cluster in question on DK2 and (ii) σ'_{v0} is the free field vertical effective stress



depth, reflecting changes in the local CPT q_c profile and the relative pile tip depth, h/R . Chow showed that equation 1 captures these two strong influences, as well as a weak dependence on the local vertical effective stress, σ'_{v0} .

Although a minimum value has to be specified to h/R^* in equation 1 (taken as 8), the ICP approach does not include any upper limit (such as that in API-93) on σ'_{rc} or τ_{rz} . Very high values can be developed close to the tip of piles driven in dense sands, and a new importance should now be given to undertaking high capacity CPT tests in such deposits, rather than simply reporting 'maxed-out' values when q_c exceeds 40 to 50MPa; values exceeding 120MPa have been recorded in some sands³⁷. The classical API assumption that $\sigma'_{rc} = K \sigma'_{v0}$, where K is independent of sand state, gives a unique bilinear distribution above and below the water table that is independent of sand state and pile tip depth, contradicting completely the available field data.

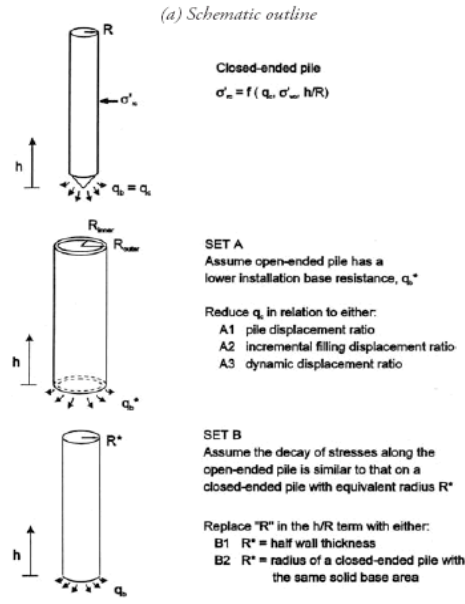
4.3 Application of σ'_{rc} expression to open-ended piles

The ICP piles from which the σ'_{rc} expression was derived were all closed-ended. In considering how the results might be linked to open-ended pile behaviour, Chow¹¹ acknowledged that the complex conditions around either pile type remained open to conjecture and sought to establish a simple approximate link that could be tested subsequently against a larger database of full-scale tests. She considered the five possibilities outlined in Figure 11, testing each against compression and tension loading data from an 11 m-long open-ended strain-gauged pile driven at the Dunkirk site, near to where her ICP tests were performed. The two approaches that were found most promising were

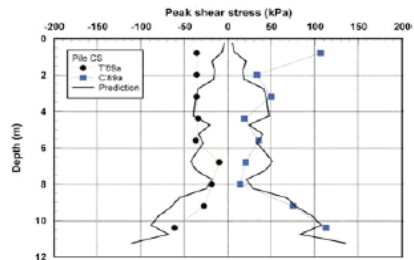
- A2, in which the scalar factor placed before the q_c term in equation 1 was varied according to the pile incremental filling ratio (IFR) recorded during pile driving. This approach reduces the radial effective stresses close to the open pile tip to values below those expected for a closed-ended pile, but maintains a proportionally similar rate of decay of σ'_{rc} for any given depth with h/R as the pile tip advances to greater depth
- B2, in which the effective stresses developed at points close to the tip were similar to those of a closed-ended pile, but the rate of decay with h/R was steeper. Strain path analysis of constant volume open tube penetration, reported by Chin³⁸, suggested substituting for R in equation 1 with an equivalent radius, R^* , equal to the radius of a closed pile of the same solid area.

Both proposals led to reasonable matches with the Dunkirk open-ended pile observations. Other possibilities could include modifying both the scalar q_c factor and the rate of decay with h/R , but insufficient data were available to justify this level of complexity. While the average stresses developed across the base of an open tube are highly likely to be affected by the IFR, those acting close to the annular area will be much higher, with the bearing pressures on the annulus of the order of $-0.7q_c$. Noting also the difficulty of predicting IFR during design in advance of driving, Chow considered option B2 the most suitable for further

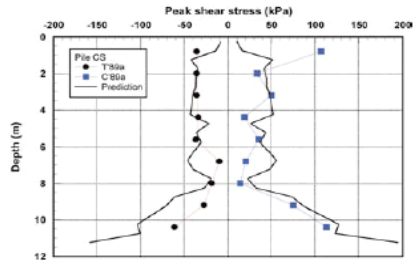
Figure 11: Approaches considered by Chow¹¹ for generalisation of ICP radial effective stress expression to open-ended pile conditions



(b) Shear stress distribution predicted by Method A2: q_c factored by incremental filling displacement ratio



(c) Shear stress distribution predicted by Method B2: R^* computed from the solid base area



investigation through database studies, adopting equation 2 to define R^* in equation 1:

$$R^* = (R_{outer}^2 - R_{inner}^2)^{0.5} \quad (2)$$

Naturally, R^*/R decreases with D/t (tending to $2[t/D]^{0.5}$ when D/t is large), so this expression leads to the radial effective stresses and pile capacities available to an open pile of given length and diameter reducing systematically with its wall thickness, unlike conventional pile design methods.

As shown later, database studies show that this expression, combined with the formulation for pile loading response outlined below, leads to a method that is equally applicable to open or closed ends and very small to very large piles. No significant skewing is evident in relation to pile length, L , diameter, D , or wall thickness ratio, D/t .

4.4 Pile loading to failure in sands

Lehane's tests at Labenne provided the first field evidence of the interface shear behaviour of a closed-ended displacement pile in (loose dune) sand. Chow's work at Dunkirk led to similar conclusions in dense sand of marine origin. The assumption made in API-93 of a simple Coulomb failure law governing failure was proven to apply in both tension and compression tests. In the latter, the limiting local value of τ_{rz} is linked to σ'_{rc} by equation 3:

$$\tau_f = (\sigma'_{rc} + \Delta\sigma'_{rd}) \tan \delta_{cv} \quad (3)$$

Here, δ_{cv} is the constant volume (critical state) interface shear parameter that is measurable in simple laboratory tests³⁹. In contradiction to API-93, δ_{cv} was considered in MTD-96 to reduce as the sand's grain size (represented by d_{50}) increases, but be independent of its initial relative density. As explained later, a slightly modified relationship was suggested in ICP-05.

A further key difference is that σ'_{rc} can change as the pile is loaded. The 'dilatant' component $\Delta\sigma'_{rd}$ could be estimated with reasonable accuracy through a simple elastic cavity expansion expression⁴⁰, and Chow showed that substituting the peak-to-trough pile roughness for Δr and a shear stiffness, G , estimated from CPT q_c data taken from calibration chamber tests led to a match with the available experiments. The formulation leads to a dependency on pile diameter; the $\Delta\sigma'_{rd}$ term provided up to 40% of the 101mm-diameter ICP piles' shaft capacity, but was likely to contribute less than 5% of that developed by a 2m-diameter pile.

$$\Delta\sigma'_{rd} = 2G \Delta r / R \quad (4)$$

Recently, Lehane *et al.*⁴¹ have used model tests and constant normal stiffness (CNS) shear tests to investigate the same phenomena further, where $CNS = \Delta\sigma'_{rd}/\Delta r = 4G/D$. These tests rely on establishing the same soil-interface conditions in the laboratory as in the field (including pile roughness and possibly modified soil grading) and assume that both the *in situ* soil shear stiffness, G , and pile diameter, D , are known at the time of testing.

A modified version of equation 3 applies to tension tests in MTD-96 and ICP-05, giving lower values of τ_f and reflecting the sum of two processes described by De Nicola and Randolph⁴²: the general reduction of the stress field around

the pile that is caused by tension loading; and elastic Poisson effects caused by the pile shaft stretching or compressing under load. Principal stress rotation effects, which can be of major importance in sands, also contribute and the shaft capacities of large diameter piles are predicted to be around 20 and 30% less in tension than in compression for closed- and open-ended conditions respectively.

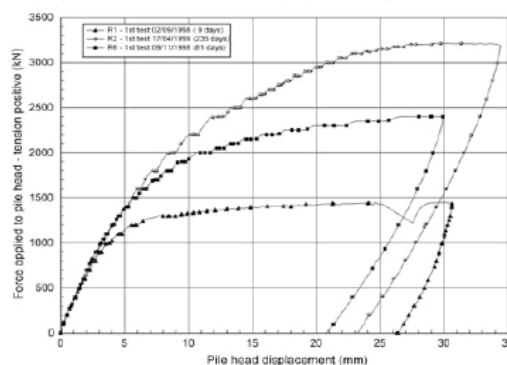
4.5 Effects of time for piles driven in sand

One further key feature of the MTD-96 recommendations for piles driven in sands was the recognition that ageing processes develop *in situ*, which lead to very substantial gains in shaft capacity with time. These were discovered in the course of Chow's research, on open-ended piles driven by the French CLAROM group at Dunkirk, and explored further by assembling a database of similar instances reported in the literature and held in consultants' files. The results and discussion on the possible mechanics of the ageing processes were presented by Chow *et al.*⁴³, who concluded that (i) circumferential compressive (and tensile radial) creep acted to weaken an arching mechanism that developed around the pile shaft during installation and (ii) interface dilation could become more important during first-time loading.

A further field investigation was conducted at Dunkirk in the late 1990s on six open piles (456mm diameter, around 19m penetration) driven in dense marine sand⁴⁴. The key results are set out in Figures 12 and 13⁴⁵. It is immediately clear that shaft capacity is a moving target that has to be associated, as with the strength of concrete cubes, with a particular age after driving. The capacities of piles that are not taken to failure can double or treble within a year of driving, but pre-testing disrupts the growth of capacity with time and multiple tests can lead to highly confusing results⁴⁵.

The ICP is thought to give a short-to-medium term capacity prediction matching tests conducted around 10 days after driving, although it is not yet clear how rapidly shaft capacity may change in first few weeks after driving. Most pile design methods aim to match load tests conducted on fresh piles, typically within a few days of driving. However,

Figure 12: Overall load-displacement curves from first time tension tests to failure on 457mm diameter, -19m penetration, open steel piles tested 9, 81 and 235 days after driving at Dunkirk⁴⁵



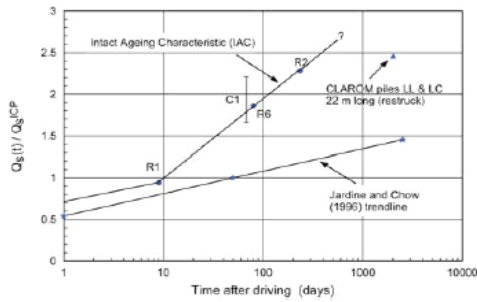


Figure 13: First-time capacities of Dunkirk piles, normalised by ICP-05 capacity predictions, plotted against age after driving⁴⁵

time effects and the negative influence of multiple re-tests on the same pile are very important factors that add to the experimental scatter and confuse pile load test interpretation. Jardine *et al.*⁴⁵ showed that the ageing processes can be accelerated by low level cyclic loading and suggested that the governing mechanisms of both processes are closely inter-related. It is important to note that aged piles experience sharp losses in shaft capacity on unloading after failure.

ICP-05 includes reference to this recent work, and research is continuing on this topic at several centres, including a joint study by Imperial College and Institut National Polytechnique Grenoble (INGP). Considerable implications are likely to follow (regarding, for example, the re-use of existing foundations) if the ageing processes can be understood sufficiently well to be exploited confidently in practice.

4.6 End bearing relationships for sands

The main finding regarding pile end bearing from the ICP field research was that the bearing pressure, q_b , that could be mobilised at a tolerable displacement (taken here as a settlement of $D/10$) was closely linked to the CPT end resistance, q_c . The conventional practice of linking q_b to σ'_{v0} via a bearing capacity factor, N_p , does not recognise the contained nature of the end bearing failure mechanism and is completely unable to capture the sensitivity of end bearing pressure to the variations in sand state observed in the ICP field tests. Severe problems also arise in attempting to apply a more appropriate model, such as cavity expansion, because of the complexities of particle crushing, plastic yielding (shearing and dilation), pressure dependency and the effects of the non-linear stiffness of the soil mass that contains the failure. Chow's assessment of the limited database of full-scale measurements indicated a strong and unexpected trend for q_b/q_c to fall with diameter, D , although few reliable data existed for driven piles with diameters greater than 600mm and none for closed-ended piles larger than 1.2m in diameter.

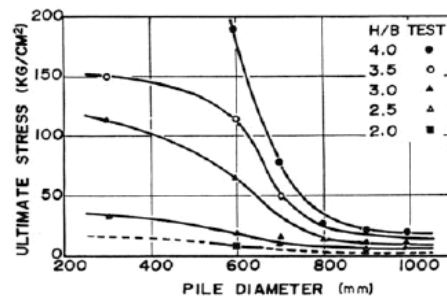
MTD-96 included equation 5 as a simple provisional recommendation for closed-ended piles, where the standard $D_{CPT} = 36\text{mm}$. The lower limit to q_b/q_c was revised from 0.2 to 0.3 in ICP-05 corresponding to a 'cut-off' diameter of 904mm.

$$q_b = q_c [1 - 0.5 \log (D/D_{CPT})] \tag{5}$$

Following from Bustamante and Gianeselli⁴⁶, a simple scheme of averaging q_c over 1.5 pile diameters above and below the pile toe was recommended in MTD-96 and ICP-05. However, it is recognised that the approach taken to select appropriate q_c values from often variable CPT traces can have a considerable influence on the end bearing calculations. Different trends with diameter can be inferred if other averaging schemes are adopted; e.g. that described by Xu and Lehane⁴⁷. The closed-end bearing ICP expressions have proved controversial, even though the bearing capacity factors applying to shallow foundations on silica sands are known to be strongly dependent on foundation diameter, as shown by De Beer⁴⁸, Tatsuoka *et al.*⁴⁹ and others. Recent finite element (FE) analyses by Yamamoto⁵⁰ using the MIT-S1 constitutive model⁵¹ can replicate this trend, which is most probably related to the dependency of the peak ϕ' angle on stress level. Although equivalent numerical modelling has not yet been performed for deep pile foundations, similar scale effects could be expected for pile end bearing where confining stresses are high. Field measurements by De Beer *et al.*⁵², Meyerhof⁵³ and the BCP Committee⁵⁴ support the existence of such scale effects.

While the question of closed-ended piles remains open to further investigation, research summarised by Hight *et al.*⁵⁵ indicated that open-ended piles' end bearing is strongly affected by diameter. Experiments in which cores were pushed up statically through steel pipes, such as those reported by Kishida and Iseimoto⁵⁶ (as shown in Figure 14), proved that plug capacities depend inversely on inner diameter, as well as increase with plug height. These features are thought to be related to the arching mechanisms developed close to the pile tip and the role of constrained interface shear on the inner circumference, where diameter-related effects apply and are analogous to the shaft resistance discussed previously. Similar processes will be active during installation, leading to the known diameter dependence of the pile plug IFR. The soil plug's stiffness to axial loading must be less than that of a closed structural pile and is likely to become softer (in terms of $q_{mobilised}/q_c$ against s/D) as the pile diameter increases.

Figure 14: Pile plug capacity as a function of internal diameter, plug height and sand density; after Kishida and Iseimoto⁵⁶



Reviewing the available information, Chow concluded that plugged open-ended piles should be regarded as delivering half of the end bearing available to closed-ended piles at a pile head settlement of $D/10$, simply by applying equation 5 divided by 2. The lower bound mobilisable q_b/q_c value applying to plugged piles with $D > 1.2\text{m}$ was revised upwards from 0.10 to 0.15 in ICP-05, recognising also that significantly higher base capacities could be available if settlements were sufficiently large to generate 'plunging failures'. A simple criterion was given regarding the possible trend for plugging to break down fully during static loading as pile diameter increases, or sand relative density drops. Taking the local q_c acting over the annular area of the pile tip provided a conservative lower bound to unplugged capacity. As shown later, these provisional rules for end bearing correspond reasonably well with the available end bearing field measurements. There is no evidence at present to suggest that time affects end bearing significantly.

As with the shaft resistance formulation, ICP-05 relies heavily on full site-specific CPT profiles being obtained and adopts a completely different predictive framework to API-93. End bearing is predicted to be far more sensitive to variations in sand states; there is no upper limit q_b and very high values can be obtained in very dense sands. As well as implying a wider spread of possible static capacity predictions, ICP-05 can also have important implications when considering pile driveability at very dense sand sites.

5. Alternative Approaches for Sand and Critiques of the ICP Methodology

The widely known shortcomings of API-93 in sand have stimulated research at several centres. Considered in this section is the development of the three other methods addressed in the Commentary sections of the 2007 API-RP2A edition, discussing their relationship with the ICP and possible advantages or drawbacks. Other approaches that are not covered in detail include those set out by Gavin and Lehane^{57,58} and Foray and Colliat⁵⁹.

One of the most significant spurs to progress was the EURIPIDES Joint Industry Project (JIP) described by Kolk *et al.*⁶⁰ that involved eight tests on a 760mm-diameter open pile driven (in 1995) to penetrations of 30.5, 38.7 and 47m in very dense marine sand in two installations at Eemshaven, the Netherlands. Bearing in mind the time effects found by Chow *et al.*⁴³, a single re-test was attempted on the pile that had been driven to 47m, tested and then aged for 1.5 years. The EURIPIDES results bear out all of the main features described previously regarding the shortcomings of API-93, which under-predicted the medium term capacities very significantly (mean $Q_c/Q_m = 0.58$, range 0.43 to 0.89) and gave distributions of shaft and base components that failed to match the trends seen in the field.

Checks by several independent workers showed that MTD-96 (which is identical to ICP-05 in this case) gave far better predictions, capturing most of the observed results regarding pile penetration depths, sand state variations and loading sign (compression or tension)^{4, 16, 26, 59}. Jardine *et al.*¹² sum-

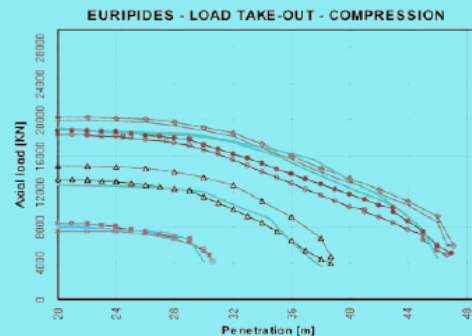


Figure 15: Independent Class A predictions made by Chow and Overy for load-depth transfer in four EURIPIDES compression tests at three pile penetrations, compared with field data trends from experiments (thick lines) reported by Kolk *et al.*⁴

marise Class A predictions, (made before the test results were available) that gave a marginally conservative mean $Q_c/Q_m = 0.97$ and a range of 0.78 to 1.12 for the eight medium term tests. Figure 15 presents the load-depth profiles assessed separately by Chow and Overy for the compression tests, showing both good predictions of field behaviour and relatively minor 'operator sensitivity' in the predicted results.

A final point to note on EURIPIDES was that the single tension re-test attempted after a 1.5 year pause confirmed that ageing effects had led to a substantial increase in the pile's shaft capacity. Similar re-tests conducted at the Jamuna Bridge site, where piles had fallen far short of their design capacities in short-term tests, showed equally impressive gains with time, confirming the trends outlined in MTD-96 and ICP-05.

Other work in the Netherlands included a critical review of the ICP and other design methods for piles in sand. As reported by Centre for Civil Engineering Research and Codes (CUR)²⁵ and summarised by Kolk *et al.*⁴, a modified version of the ICP was recommended for offshore practice that has become known as the Fugro-04 method. The main features are:

- A similar but simplified framework for shaft capacity. The dilatant term in equation 3 was dropped and it was assumed that a unique value of soil-steel interface effective friction, (with δ being taken as 29°) applied to all their sands. The lower bound value for h/R^* was also reduced to 4, equation 2 was retained to define R^* .
- Equations 1 and 3 could then be combined into the form $\tau_{rz} = A q_c (\sigma'_{v0}/P_a)^b (h/R^*)^c$ (6)
- Concerned that the onshore ICP pile load test database might not reflect offshore conditions sufficiently well, the coefficients A, b and c in equation 6 were fitted to a reduced data set of larger, mainly open-ended, piles, considering 18 tension and 21 compression medium term tests performed at 11 sites. Their best fit values for A, b and c were 0.08, 0.05 and -0.90 in compression and

0.045, 0.15 and -0.85 in tension. These values led to greater difference between compression and tension than the ICP, a weaker influence of the vertical effective stress and a steeper decay of σ'_{vc} with h/R^* . No specific allowance was made for the effects of time on shaft capacity.

- Stating that the ICP end bearing expressions were overly conservative, Fugro-04 adopted the different form of end bearing expression given in equation 7.

$$q_b = 8.5 (q_c P_d)^{0.5} (R^*/R)^{0.5} \quad (7)$$

Kolk *et al.*⁴ fitted their selected field data by assuming a non-linear relationship with q_c and D/t , rather than a diameter dependence.

- Considering the overall database of 37 tests, Kolk *et al.*⁴ reported mean and \pm CoV values for Q_c/Q_m of 0.92 ± 0.88 for API-93; 1.14 ± 0.45 for MTD-96^a and 1.03 ± 0.44 for Fugro-04. They concluded that the ICP might be non-conservative for long open-ended piles.

Kolk *et al.*'s⁴ recommendation that δ should be taken as 29° in all cases came from observations that repeated cyclic interface shear tests which were continued to large numbers of cycles (or total shear displacements) led to an ultimate angle δ that was less dependent on initial mean grain size, d_{50} , than the interface shear box test trend reported in the MTD-96 document. They also noted that the EURIPIDES pile, on extraction, had a reduced exterior maximum surface roughness. Figure 16 shows the MTD-96 curve³⁹, the Fugro-04 recommendation²⁵ and the flatter trend found in interface shear tests conducted for Shell UK on North Sea sand samples. A subsequent series of interface ring shear tests conducted at Imperial College on a range of single sized sands has verified that the interface shear zone gradually becomes filled with crushed fine sand and silt, while the average surface roughness may decline very slightly. While the new results give final δ values between 26 and 30° , it is noted that the system applying near the shafts of piles is less constrained than in the thin laboratory specimens. Fine material generated from a coarse sand or gravel could disperse into the surrounding soil mass⁶¹, pushing new coarse grains into contact with the pile surface and renewing the initial shearing conditions. Taking the lower values of either the MTD-96 curve or site-specific ring shear tests is recommended in ICP-05 as the safest way to proceed. Further research is currently underway at Imperial College, including similar tests on concrete interfaces.

Kolk *et al.*'s⁴ findings were discussed by the API-ISO pile design task group. Concerns were raised by the authors over the inclusion of six tests on the Jamuna Bridge piles driven in micaceous sands, as well as two on mixed carbonate sands/clays from Ras Tanajib, as these soils behave differently to silica sands in shear and compression^{12,62}. A further factor was the inclusion of multiple tests on the same piles, which can give misleading results. As noted earlier, it can be hard to identify how different variables affect behaviour in complex problems by statistical treatment of small data-

^a As noted earlier, MTD-96 has slightly different lower bound end bearing values to ICP-05.

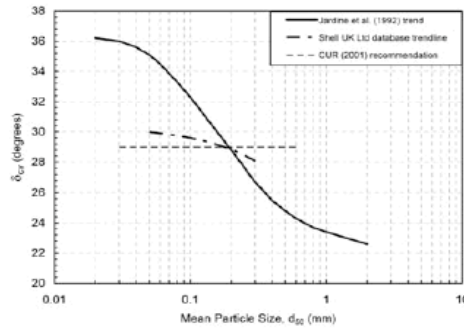


Figure 16: Illustrative trends for interface shear angle variation with d_{50} values¹²

sets. The results can be very sensitive; subtracting just one doubtful EURIPIDES case from the Fugro-04 set led to a 4% fall in the mean Q_c/Q_m values, while the other factors raised could skew the trends with respect to parameters such as h/R^* . Nevertheless, Kolk *et al.*'s⁴ work emphatically reinforces the conclusions drawn earlier regarding the unreliability of API-93 and contributes to the debate on how to improve design recommendations.

In parallel with the EURIPIDES tests and the CUR²⁵/Fugro-04 study, a group from NGI developed another 'CPT' based approach for piles in sand, which is termed NGI-05 here¹⁶. The method adopts the 'sliding triangle' framework proposed by Toolan *et al.*³³, rather than the form given in equation 1 to give 'friction fatigue', with a series of terms derived from analysis of a database of full scale load tests. The key points regarding shaft friction are as follows:

- In place of equations 1 and 3, or 6, they adopt for local shaft resistance the form:

$$\tau_{rz} = z/z_{tip} (P_d) \cdot F_{Dr} \cdot F_{sig} \cdot F_{tip} \cdot F_{load} \cdot F_{mat} \quad (8)$$

$$\text{where } F_{Dr} = 2.1 (D_r - 0.1)^{1.7} \quad (9)$$

$$\text{and } F_{sig} = (\sigma'_{v0}/P_d)^{0.25} \quad (10)$$

$$\text{with } D_r = 0.4 \cdot \ln \{q_c / [22 \cdot (\sigma'_{v0} P_d)^{0.5}]\} \quad (11)$$

- In the above, z = depth to the point in question and z_{tip} = the depth to tip. Pile end conditions have a substantial influence, with $F_{tip} = 1$ for open and 1.6 for closed ends. As with ICP-05, a reduction of 20 to 30% applies to tension loading: $F_{load} = 1$ for tension and 1.3 for compression. A surprising feature is that pile material makes a difference with $F_{mat} = 1$ for steel and 1.2 for concrete, presumably through the development of significantly higher interface δ angles for concrete. The latter feature is not supported by recent interface shear tests carried out at Imperial College on both types of material.
- In common with the ICP and Fugro approaches, equations 8 to 11 result in local shaft resistance being sensitive to D_r , relatively insensitive to σ'_{v0} and strongly affected by h/R .
- Pile end bearing is linked directly to the CPT for closed or plugged conditions with:

$$\text{for closed ends: } q_b = 0.8 q_c / (1 + D_r^2) \quad (12)$$

$$\text{for plugged open piles: } q_b = 0.7 q_c / (1 + 3D_r^2) \quad (13)$$

- Clausen *et al.*¹⁶ gave further details describing the conditions for coring open piles and presented statistics showing how their method compared with others for a database of 28 'high quality' tests on steel piles where CPT profiles were available.
- Their eight tension tests' mean and \pm CoV values for Q_c/Q_m were 0.57 ± 0.33 for API-93; 0.96 ± 0.15 for MTD-96; 0.95 ± 0.20 for NGI-05 and 0.79 ± 0.43 for Fugro-04.
- The 20 compression tests yielded mean and \pm CoV Q_c/Q_m values of 0.67 ± 0.39 for API-93; 0.85 ± 0.27 for MTD-96; 0.95 ± 0.23 for NGI-05 and 0.94 ± 0.39 for Fugro-04.
- As found by Kolk *et al.*⁴, the API-93 method gives the worst fit by far. NGI-05 best predicts the database to which it was fitted, giving comfortably low CoVs. The MTD-96 predictions are marginally more conservative than NGI-05's, although updating to include the end bearing expressions in ICP-05 might redress this balance slightly. The Fugro-04 method did less well when tested against a different database.
- Even though the pile test databases had substantial commonality, the CoVs found by NGI were far smaller than those assessed by Fugro. While the ICP parameters were close to those found independently by the authors, the API-93 CoVs were significantly lower. Clausen *et al.*¹⁶ concluded that the MTD-96 approach worked well for piles in dense sand, but considered it to be potentially conservative in looser deposits.

Conflicting evidence from the ICP-05, Fugro-04 and NGI-05 database studies led the API-ISO pile design group to ask a group led by Professor Lehane at the University of Western Australia (UWA) to make an independent assessment of which method would be most suitable to replace the poorly performing Main Text API-93 approach. This group's work, which started in September 2004, involved first setting up an independent high quality database. Applying quality criteria to filter an initial set of around 200 tests left 77 silica sand tests with no calcareous or micaceous cases. The UWA set, which had substantial commonality with the slightly larger ICP-05 database^b, had two to three times more entries than the filtered Fugro and NGI data sets. Lehane *et al.*¹⁷ systematised their approach to data interpretation, capacity calculation and criteria for comparing methods.

5.1 UWA-05 shaft capacity approach

In addition to testing the API-93, NGI-05, Fugro-04 and ICP-05 procedures, Lehane *et al.*¹⁷ proposed a new, UWA-05 procedure. Their elaboration of the ICP approach followed from Gavin and Lehane's^{57, 58} review of the choices made by Chow to account for differences between open- and closed-ended pile penetration. Gavin and Lehane preferred to apply her Set A hypotheses, as set out in Figure 11, and considered that equation 1 could be applied with

^b Although the UWA database includes more sites with full CPT profiles.

$R^* = R$ for open-ended piles, provided that an effective base resistance, q_b , was substituted, which depended on q_c and fell linearly with IFR. IFR tends to rise with increasing pile diameter, D . In common with Chow's¹¹ application of Set A2, they found a good fit to the Dunkirk open-ended driven pile tests and showed compatibility with open-ended pile driven data from seven other sites. As noted earlier, the stresses developed by a coring pile are likely to approach $-0.7 q_c$ on the annulus, and the substantial reduction implied by adopting a factor relating to IFR, or a related effective area ratio, is unlikely to prove realistic at locations close to the annular base.

White⁶³ proposed that local shaft capacity should be considered through a slightly different form involving the effective area ratio, $A_{r,eff}$, defined in Equation 14, while keeping a similar term for the influence of the pile tip relative depth, b , as shown in equation 15.

$$A_{r,eff} = 1 - IFR [D_i/D]^2 \quad (14)$$

where D_i is the inner diameter.

$$\tau_{rc} = a [A_{r,eff}]^b q_c (b/D)^c \quad (15)$$

White used a simplified cavity expansion analysis to argue that the effective area ratio influence can be expressed as a power law. His argument considered the more limited radial expansion applied to the soil mass by a pipe pile, rather than a closed pile. In common with MTD-96, NGI-05 and Fugro-04, he assumed that 'friction fatigue' depends on a normalised relative tip depth (here b/D), rather than b or blow count, N , as argued by White and Lehane²⁷. Unlike the other methods considered previously, the vertical effective stress, σ'_{z0} , plays no role. The powerful effects of time and significant role of interface dilatancy on shaft capacity were not addressed, and it was assumed implicitly (as in Fugro-04) that the same δ angles apply to all sands and piles. The last two points were addressed by White *et al.*⁶⁴, who used a similar expression to consider the equalised radial effective stress. Equation 6 gives their general form, which was adopted for the UWA approach. White *et al.*⁶⁴ considered this to be applicable up to the D/t ratios applying to offshore piles, but not to slender suction caisson walls.

$$\sigma'_{rc} = a [A_{r,eff}]^b q_c (b/D)^c \quad (15^*)$$

Their simplified model of the tip penetration process assumed plastic cavity expansion and did not model fully the strain reversals that may be anticipated from equivalent strain path analyses of clays. The analysis led to a prediction that partial cavity expansion could be modelled by the power law shown in equation 15*, with the exponent b being $(K_p - 1)/K_p$ or $2(K_p - 1)/K_p$ considering cylindrical and spherical geometries, respectively. These expressions match the b value of 0.3 adopted for the UWA design method if respective ϕ' values of about 25° and 11° are substituted. Far higher ϕ' values are likely to apply in the field, especially to dense sands, so the analysis must be considered as indicative of possible trends rather than being reliable quantitatively.

White *et al.*⁶⁴ made an interesting prediction for the stress regime that might result after a cylindrical cavity unloading

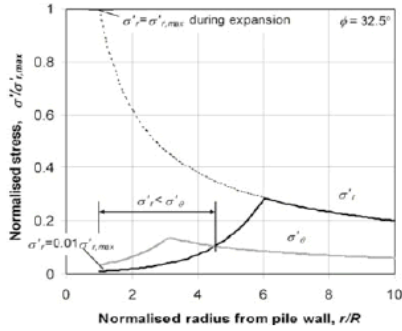


Figure 17: Simple analysis of radial variation in stress near pile after unloading behind tip; from White et al.⁶⁴

process that reduced the radial stresses to just 1% of their maximum installation values, and this is reproduced in Figure 17, giving an impression of the circumferential arching postulated by Chow¹¹ that could shield a pile shaft from higher radial stresses acting at points further from the shaft. As noted earlier, such a stress regime might be important in explaining the strong effects of ageing on shaft capacity.

While the unloading process was not explored fully, it was assumed that the strain fields imposed by steady penetration would be fixed within a short distance of the pile tip passing, suggesting steady stresses when $b > D$. The reductions in σ'_{rc} that are observed to take place as b increases above this value were considered to be results of friction fatigue⁶ and the $(b/D)^c$ term in equation 16 was included to model this process. Although White and Bolton⁶¹ and Randolph²⁶ had suggested that particle crushing and fines migrating out radially from the pile shaft would be important, White⁶³ considered local densification due to cyclic loading (Chow's factor d in Figure 9) to be the main cause, while the purely geometrical effects were expected to be confined to the immediate vicinity of the pile tip. As shown later, the extreme two-way load cycling associated with pile driving is likely to degrade the stresses acting on the pile shaft very significantly.

Although the number of cycles is not included explicitly in the UWA-05 expression, White⁶² suggested that the exponent c in Equation 15 would be related to the number of blows or jack strokes and indicated that values between -0.25 and -0.5 might be applicable to cases involving 10^2 to 10^4 cycles. A c exponent of -0.5 was adopted by Lehane et al.¹⁷ for the UWA approach to give equation 16, and the minimum b/D set at 2.

$$\sigma'_{rc} = f_i/f_c \cdot 0.03 \cdot q_c \cdot A_{n,eff}^{0.5} \cdot (b/D)^{0.5} \quad (16)$$

Local shaft capacity is found in UWA-05 by applying equation 3 with layer specific δ values similar to those specified in the ICP-05. While an alternative expression is used to relate G and q_c , the loading factor, f_i/f_c , drops from 1 in compression to 0.75 in tension, giving a similar reduction to ICP-05 and NGI-05. $A_{n,eff}$ is calculated from equation 14, taking an empirical relationship for IFR in design where

$$IFR = \min[1, (D_t(m)/1.5)^{0.2}] \quad (17)$$

IFR and final filling ratio (FFR) measurements made during installation can be used for hindcasting exercises, and it is possible that the statistical parameters associated with such back-analyses can be different to those applying to design when only equation 17 will be available.

Before moving onto consider the UWA base capacity approach, it is informative to consider Gavin and O'Kelly's⁶⁵ field experiments with closed-ended instrumented piles in over-consolidated very dense fine sand at Blessington, Ireland. They investigated the effects on the σ'_{rc} values developed of the number of cycles, N , imposed during installation of their 73mm-diameter piles. Installation involved either jacking at a steady CPT rate of 20mm/s for a stroke of up to 2.1m, or stop-go cycles applied at either 20 to 40 cycles per metre of penetration, giving a maximum of 150 cycles. In these tests, even steady monotonic installation led to a clear reduction of σ'_{rc} with b/D that led to a 60% or greater reduction in σ'_{rc} being recorded as the pile tip advanced from $b/D = 1.5$ to $b/D = 10$ (the lowest and highest instrument levels). Cyclic installation added to the reduction of σ'_{rc} with b/D , leading to a further reduction of one-third, or more, in the maximum monotonic σ'_{rc} values.

It appears that another monotonic process contributes to the b/D trend at points well above the pile tip. Significant migration of fines is unlikely in the dense Blessington fine sand, and it is suggested here that in addition to the general stabilisation of the pattern of soil flow around the tip, time-dependent straining may be an important, but hitherto neglected, factor. During monotonic penetration, points near the shaft and well above the pile tip experience principally kinematically controlled boundary conditions. The free-field vertical and K_0 lateral stresses are relatively small compared to the extremes experienced as the pile tip passed through the layer in question. The soil positioned near the pile shaft is likely to undergo relatively rapid stress relaxation and slight radial compression, as the sand re-adjusts to its recent massive stress and strain perturbations. Small additional radial movements developed near the interface will have less effect on the higher stresses locked in at greater radii, as sketched in Figure 17, although circumferential creeping further out in the sand mass would tend to redress this imbalance in time, leading to the longer term ageing characteristics outlined in Figures 12 and 13.

Equation 15* is likely to lead to different shaft resistance results to the ICP: it contains no term for σ'_{r0} , which increases resistance with pile length (L), and it deals with pile slenderness (b/D or L/D) and wall thickness ratio (D/t or R/R^*) in different ways. These differences suggest ways in which the field testing databases can be interrogated. Figures 18 to 20 present the answers available from the ICP sand database: no clear or significant bias or skewing is evident for Q_d/Q_m predictions made with the ICP-05 method in relation to these three pile geometry parameters.

Schneider and Lehane⁶⁶ presented summary plots from the restricted database of field tests, for which load transfer

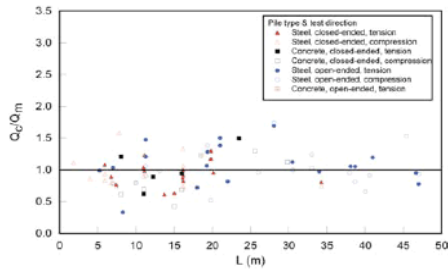


Figure 18: Distribution of Q_c/Q_m with respect to pile length, L ; ICP-05 shaft capacity procedure for sands

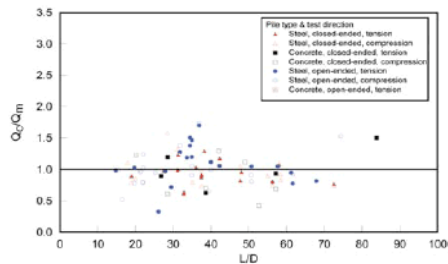


Figure 19: Distribution of Q_c/Q_m with respect to pile slenderness ratio, L/D ; ICP-05 shaft capacity procedure for sands

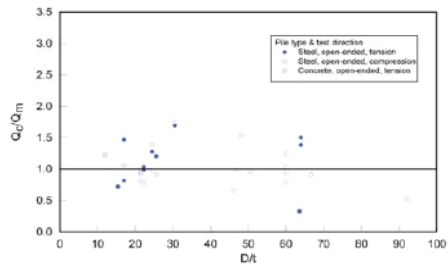


Figure 20: Distribution of Q_c/Q_m with respect to pile wall thickness ratio, D/t ; ICP-05 shaft capacity procedure for sands

curves exist that can be interpreted to show distributions of shaft resistance with depth. Their scatter diagrams are presented in Figure 21, along with general trends drawn for ICP-05 and Fugro-04 τ_{rz}/q_c predictions. It is not clear how unique predictive trends can be drawn on these plots, given the variations with pile lengths, σ'_{v0} levels and contributions of interface dilation between the cases considered, but the distributions seem to imply that the ICP may over-predict at high b/D ratios. However, it should be borne in mind that the plotted data include multiple measurements made in micaceous sand at the Jamuna Bridge site. Schneider *et al.*⁶² argue that the rate of 'friction fatigue' was stronger in these tests than is typical in silica sand, and

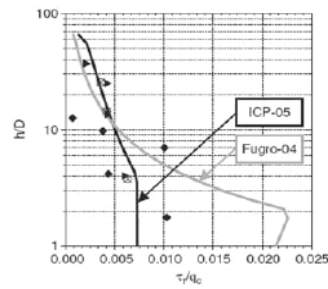


Figure 21a: Comparison between design methods and field data⁶⁶; closed-ended piles in compression*

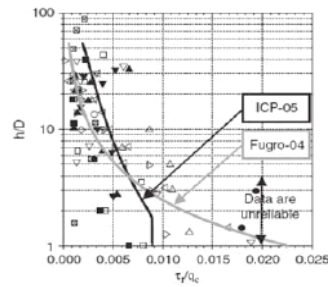


Figure 21b: Open-ended piles in compression and field data⁶⁶*

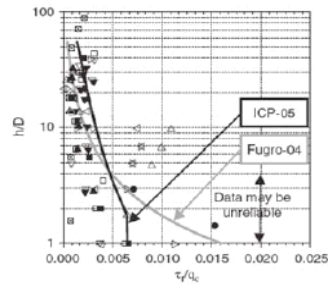


Figure 21c: Closed-ended piles in tension and field data⁶⁶*

*Note data in Figures 21a, b and c include micaceous sand cases and Fugro-04 and ICP-05 trend lines may not be unique

unfortunately including these measurements in Figure 21 introduced some bias.

Given the Blessington data showed reductions in σ'_{rc} falling with b/D in the absence of any load cycling, the relatively close similarities between jacked and driven pile databases found when pile end details are accounted for, and the fact that capacity can recover with time after driving, the recent trend to term all of the factors that lead to stress reductions linked to b , b/D , b/R^* or N as 'friction fatigue' may well prove misleading.

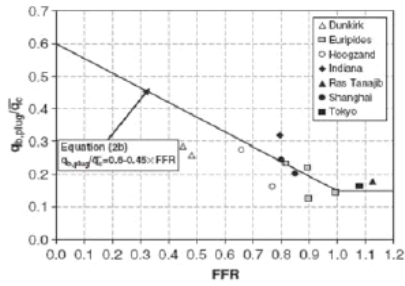


Figure 22: UWA data relating $q_{b,plug}/q_c$ to FFR; after Xu et al.⁶⁹

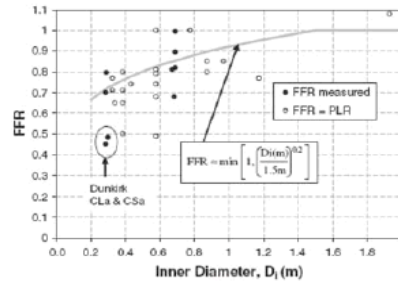


Figure 23: UWA data relating FFR to inner diameter; after Xu et al.⁶⁹

5.2 UWA-05 base capacity methods

End bearing is linked directly to appropriate local average CPT q_c values in the UWA approach. Xu and Lehane⁴⁷ placed particular emphasis on the q_c averaging technique. Unlike NGI-05, Fugro-04 or ICP-05, their interpretation of the available closed-ended pile test database led to a recommendation that mobilisable $q_b = 0.5q_c$ independent of all other factors (D , D_r or σ'_v), provided that the 'Dutch' method proposed by Van Mierlo and Koppejan⁶⁷ was followed to establish average q_c values in any variable CPT profiles. They also considered jacked pile tests misleading, because they involve a higher degree of locked-in base load. If the simpler Bustamante and Gianesselli²⁹ q_c averaging was applied and the jacked piles treated equally with the driven, then they recovered the strong dependence on diameter implicit in ICP-05. As discussed earlier, the question of whether diameter may affect closed-end bearing characteristics remains controversial. Xu⁶⁸ reported on additional research into how q_b varies in layered soil profiles, including interesting data from centrifuge studies on closed-ended piles.

Xu et al.⁶⁹ set out the corresponding open-ended UWA-05 approach. It is assumed that the plug cannot fail during static loading. Overall base capacity includes a contribution for the pile annulus, over which mobilisable $q_{b,ann}$ is taken as $0.6q_c$, as is a plug contribution $q_{b,plug}$ that applies over the inner area and depends on the FFR and q_c as indicated in equations 18 to 20.

$$q_{b,plug} = q_c (0.6 - 0.45 FFR) \tag{18}$$

$$A_r^* = 1 - FFR (D_i/D)^2 \tag{19}$$

$$FFR = \min[1, (D_i(m)/1.5)^{0.2}] \tag{20}$$

As with the shaft expression, FFR measurements can be fed into equation 19 in hindcast analyses of cases where plug measurements had been made during driving. Equations 18 and 20 are empirical and the datasets from which they were interpreted are shown in Figures 22 and 23. A strong diameter dependence is predicted, as shown in Figure 24, where the method is compared with the broadly similar trend of ICP-05 under plugged conditions. The latter is more conservative over the midrange of diameters. Both methods tend to have the same minimum $q_b/q_c = 0.15$ for large diameter piles with low D/t ratios, which falls below the minimum of 0.20

estimated previously by Lehane and Randolph⁷⁰. Even so, Xu et al.⁶⁹ reported that their approach was marginally non-conservative (giving a mean Q_s/Q_m of 1.08 and CoV of 0.15) when tested against their limited database of 13 tests at seven sites, taking account of site measurements of FFR where possible. The corresponding statistics from other methods were 1.00 ± 0.81 for NGI-05, 1.29 ± 0.55 for Fugro-04 and 0.76 ± 0.29 for the ICP, which included allowance for potential coring failure under the specified conditions.

Pile designers need to be careful when applying CPT based methods to assess base capacity. The profiles for q_c are often variable spatially, and it is hard to know whether the tips of particular piles, driven to pre-defined depths, will rest in high or low q_c layers. While such local variations have less effect on shaft capacity, they can affect q_b considerably and a cautious interpretation is advisable for design.

5.3 UWA-05 offshore approach

The UWA team also proposed a simplified version of its method for 'offshore use' in which

- The dilatant $\Delta\sigma'_v$ term is dropped from equation 3
- The area ratio fed into equation 15 is taken as $A_r = 1 - (D_i/D)^2$
- Base capacity is calculated as $q_b = q_c (0.15 + 0.45 A_r)$.

While these conservative changes to UWA-05 may not affect very large piles greatly, they impact significantly at the lower end of the offshore range. Considering the longest

Figure 24: Overall UWA-05 and ICP-05 relationships between q_b/q_c and diameter, D, taking $D/t = 30$

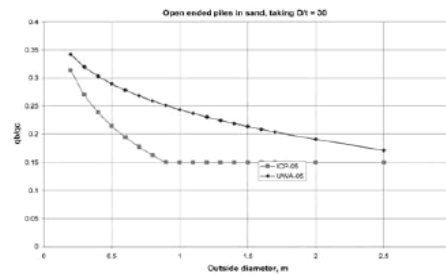


Table 2: Summary of statistics from UWA database assessment; after Lebane et al.¹⁷

Q_c/Q_m values for following:	API-93 μ and CoV	NGI-05 μ and CoV	Fugro-04 μ and CoV	ICP-05 μ and CoV	UWA-05 μ and CoV
All driven, open and closed	0.81±0.67	1.11±0.37	1.11±0.38	0.95±0.30	0.97±0.27
Driven, open compression	0.75±0.68	1.01±0.25	1.14±0.30	0.89±0.28	0.98±0.19
Driven, open tension	0.72±0.76	1.01±0.35	0.90±0.32	0.90±0.27	0.91±0.23

EURIPIDES piles (760mm in diameter), which are similar to some driven at recent offshore installations for Shell⁶, the capacity down-gradings of -15% and -10% for shaft and base, respectively, can be assessed in comparison to the ‘full’ UWA-05 employed in their database study.

5.4 UWA database assessment of API-93, NGI-05, Fugro-04, ICP-05 and UWA-05

Lebane et al.¹⁷ presented their database assessment in a comprehensive report issued shortly before the API/ISO Panel met in September 2005. Table 2 provides a simplified summary of their main results, considering only the arithmetic mean, μ , and \pm CoV values for Q_c/Q_m . Key points from the UWA study include:

- Compatibility with others’ independent assessments, giving confidence in the results presented. Checks of UWA-05 by NGI, and of the ICP-05 and API-93 by the authors, match the global results given in Table 2 to within -0.05 for both μ and CoV. Greater variations might be expected when considering smaller subsets; for example the authors’ mean ICP-05 results for open driven piles fall closer to unity than those given in Table 2
- Clear confirmation of the poor performance of API-93; all four CPT methods perform considerably better.

A slight trend for Fugro-04 to be non-conservative on average for open-ended piles in compression, which is usually the key design case, while NGI-05 was non-conservative for closed-ended piles. The CoVs associated with these two methods generally fall between 0.25 and 0.40.

- UWA-05 and ICP-05 appear marginally conservative in all of the categories (including closed-ended piles) and

show generally lower CoVs (0.19 to 0.32).

- The ICP is marginally more conservative than UWA-05, while the latter has marginally lower CoV values when applied in the hindcasting database assessment. This ranking might change when considering application to design, where IFR and FFR measurements are not available.

Lebane et al.¹⁷ went on to consider the implications of the above statistics regarding overall reliability. An ideal method would be free of both bias and scatter, giving μ close to unity and a low CoV. While the original aim of API-93 was to have mean $Q_c/Q_m = 1$, it appears from Tables 1 and 2 to be conservative compared to the new methods, and it might be argued that this bias offsets its high CoV. Lebane et al.¹⁷ computed Nominal Reliability Index values, β , for the five methods, considering different pile loading cases and factors of safety. The results are presented in Figures 25 and 26. Focusing on open-ended offshore piles, it appears that only the UWA-05 and ICP-05 offer consistently improved reliability at a WSD FoS of 1.5, while all four new approaches do better than API-93 at FoS = 2.

Jardine et al.¹² discussed the same issues and suggested approaches to ensure suitable target reliabilities for foundations by adjusting the LRFD or WSD factors to match different loading environments and ground conditions. Pile design reliability varies with soil type, and even the best of the new methods for sands may not always lead to satisfactory reliability indices in combination with the current industry standard factors. However, the database studies lead to a paradox: if the API-93 method appears so unreliable in database studies, why are in-service foundation problems reported so rarely? Potential explanations include:

Figure 25: Reliability Index based on UWA database study for WSD FoS = 2.0; note OEC is open-ended compression; OET is open-ended tension; CEC is closed-ended compression; CET is closed-ended tension and API-00 is the same as API-93; after Lebane et al.¹⁷

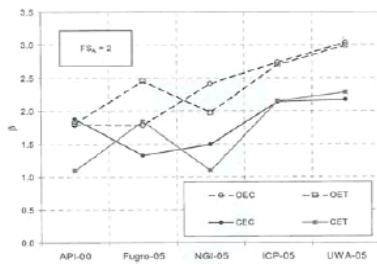
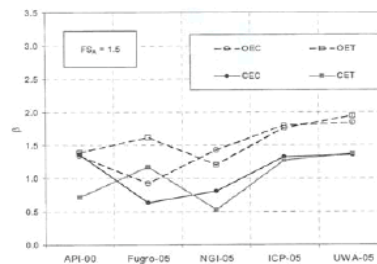


Figure 26: Reliability Index based on UWA database study for WSD FoS = 1.5; note OEC is open-ended compression; OET is open-ended tension; CEC is closed-ended compression; CET is closed-ended tension and API-00 is the same as API-93; after Lebane et al.¹⁷



- The short term capacity calculations do not account for the very large potential gains in shaft capacity with age, as illustrated in Figures 12 and 13.
- Almost no platforms are instrumented to detect the levels of permanent movement incurred, involving perhaps a few centimetres of settlement, when individual piles have been overloaded during storms.
- Many offshore foundation systems have degrees of redundancy.
- The real environmental loads may be less than expected in design, or the extreme wave loads have not yet been encountered.
- A tendency for conditions encountered in some highly developed provinces (Gulf of Mexico, North Sea, etc.) to involve dense sands more often than loose sands; the API-03 tends to be conservative for dense and non-conservative for loose sands.

Simple statistical treatments can give misleading results if any of the methods is significantly skewed with regard to one or more particular variable (as is API-93, see Figures 1 to 6) rather than just producing a distributed scatter, as that method's reliability will change from setting to setting. For example, API-93 is clearly less safe when applied to tension loading for slender piles in loose sand, than with compression loading in dense sand on piles with low L/D (or D). Lehane *et al.*¹⁷ produced a wide range of scatter plots to investigate such features, confirming the above statements regarding API-93. Systematic skewing was harder to detect among the new CPT based approaches, although Fugro-04 tended to be non-conservative with open piles in compression at low L/D (and conservative at high L/D) while ICP appeared over-conservative in a small number of loose or very loose sand cases.

Lehane *et al.*¹⁷ considered a pair of site profiles and pile sizes (1.2 to 2.44m diameter) that were considered typical for platforms in the Gulf of Mexico. Both had variable CPT profiles with most q_c values falling between 30 and 60MPa. Considering penetration down to L/D of 75 at one location and 25 at another, they found that the API-93 generally gave around the lowest capacity predictions at both sites. Given the EURIPIDES results and the database studies, these results are not surprising. The opposite conclusions would be drawn from examples involving conditions such as those at Hound Point, Sungai Perak or Jamuna Bridge, where API-93 gave unsafe design estimates³.

5.5 The 2007 edition of API-RP2A

While the 2007 edition of the API-RP2A seemed set to adopt a CPT based approach for sand, last minute concern that NGI-05, Fugro-04, ICP-05 and UWA-05 predictions did not converge closely and would lead to shorter piles in Gulf of Mexico settings led to an intermediate solution being adopted. The 2007 Main Text aims to avoid potential unconservatism and states that its recommendations are not applicable for gravels, silts or loose to very loose sands. To avoid confusion with the CPT based methods, local shaft capacity is calculated in terms of coefficients $\beta = \tau_{r,z}/\sigma'_{vo} = K \tan \delta$, along with the usual upper limits.

Users are invited to consider the simplified versions of the NGI-05, Fugro-04, ICP-05 and UWA-05 methods that are given in the Commentary and considered to be 'preferable to the Main Text' approach, provided they are used by appropriately qualified engineers and that site investigations are adequate. Readers are given advice on how to distinguish free draining sands from other soils from piezocone tests and are warned not to apply the Commentary CPT based methods to calcareous or micaceous sands.

While the NGI-05 and Fugro-04 methods have not been adjusted from those discussed previously and tested in the UWA database, the conservative 'Offshore' UWA variant is specified along with a similarly simplified ICP-05 variant that drops the dilatant shear component and rounds all of the exponents conservatively. Equation 1 is now effectively combined with equation 3 and recast as:

$$\sigma'_{rc} = 0.023 q_c (\sigma'_{vo}/P_a)^{0.1} A_r^{0.2} (b/R)^{0.4} \tan \delta \quad (21)$$

With $A_r = 1 - (D/D)^2$, the end-bearing expressions are left unchanged. No analysis or justification is offered for the effects of the adjustments made to the UWA and ICP, or their impact on the database statistics. These statistical data are vital input parameters when adopting a reliability based design approach; adopting methods with unknown conservative biases and CoVs hampers their application.

The new Commentary also allows the full ICP-05 to be applied, particularly to piles with $D < 0.76m$, 'provided that larger factors of safety be considered in the WSD design. Reference should be made to Jardine *et al.*¹² for a discussion on reliability based design'. It recommends that similarly raised FoS should be applied to the Fugro-04 and NGI-05 approaches. Given the better reliability indicated for the original ICP-05 by the database studies and its successful use in practice since the publication of MTD-96, it is suggested that the full method, including the reliability based rationale for LRFD or WSD factors, can be used more widely and safely than suggested by the Commentary. A further factor in favour of using ICP-05 is that a compatible method exists to cover the prediction of shaft and base capacity in clays.

6. ICP Approach for Clays

The ICP-05 formulation for piles driven in clays is identical to that in MTD-96, although the commentary offers more guidance on parameter selection, considers a broader range of material types, adds new case-history material and offers a larger database study to demonstrate the method's practical application. The approach was developed from intensive field work with highly instrumented closed-ended ICPs at four clay sites covering Eocene stiff marine clay (Canons Park), stiff Glacial till (Cowden), low OCR Holocene shallow marine (Bothkennar) and low OCR Glacio-lacustrine clay-silts (Pentre). The field tests carefully followed the changes in the effective stresses developed on the shafts during installation, pore pressure equalisation and load testing, including some cyclic loading experiments.

The effective stress Coulomb law was found to control shaft failure at all sites. Rather than rise during loading to failure (as in sands, equation 3), the radial effective stresses generally



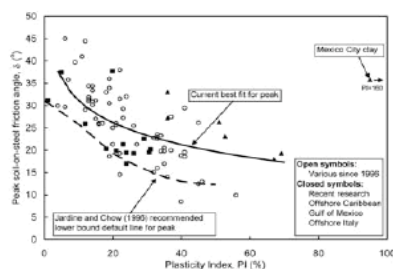
Figure 27: Thin-section through interface shear zone sampled from clay 'cake' adhering to ICP tested at Pentre, showing system of multiple oriented 'residual' shear surfaces¹¹

fell by around 20%, leading to equation 22 being recommended for design by Lehane²⁰ and Lehane *et al.*²⁴. Unlike sands, the clays did not show any systematic differences in the σ'_r responses developed on the shaft in tension and compression tests, probably because the response of the surrounding clay mass is primarily undrained.

$$\tau_f = 0.8 \sigma'_{rc} \tan \delta \quad (22)$$

The interface-shear parameter δ was found to be determined by the large shear displacement residual strength behaviour of the clay in question. Bond and Jardine^{71,72} showed that fast-jacked and driven piles generate imperfectly developed sets of polished slickensided shear surfaces in high OCR plastic clays. The thin-section given in Figure 27, which was prepared from the interface shear zone of one of Chow's pile tests at Pentre, shows a similar shear fabric in a low OCR, low plasticity clay-silt. Equivalent results were found in interface shear zone samples taken in highly plastic under-consolidated clays from piles extracted at the WD58A platform site in the Gulf of Mexico⁷³. This partially developed residual fabric leads to local peak δ values that usually

Figure 28: Database of peak δ values for clays as assessed from ring shear interface shear tests following ICP procedures¹²



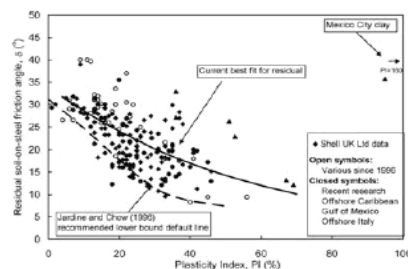
fall well below critical state ϕ' , but above the ultimate angles developed in slow drained interface shear. Laboratory tests show that only a few millimetres of post-peak slip are needed to fall from δ_{peak} to δ_{ult} , leading to a potentially brittle local shear response during slow load testing and the potential for progressive failure down the pile shaft.

The possible range of peak and ultimate δ values for clays is large; data falling from below 10° to over 40° are shown in Figures 28 and 29. Fortunately, ring-shear interface laboratory tests (as specified in ICP-05) were able to match the characteristics seen in the field at all of the ICP sites and led to successful predictions for capacities at many others.

As in sands, the profiles of σ'_r developed after full pore pressure equalisation depend critically on both the initial state of the ground prior to installation and on the relative pile tip position, b/R . Reviews of instrumented pile tests by others, cavity expansion approaches and strain path analysis conducted in conjunction with Professor Whittle at the Massachusetts Institute of Technology (MIT) led Lehane *et al.*²⁴ to propose a simplified design approach, in which $\sigma'_r = K_c \sigma'_{v0}$ and K_c was a function of the clay's YSR (or apparent OCR), its sensitivity, S_p , and the pile tip position, b/R . Fitting a power law, Lehane *et al.* found that an exponent of around -0.2 worked best, implying a gentler variation than that seen in sands, for which they found an exponent of around -0.4 (see equation 1).

Chow¹¹ gave careful consideration to the possible impact on K_c of the installation factors outlined in Figures 9 and 11. Her field tests at Pentre on low plasticity, low OCR clay-silt explored the effects of the number of cycles imposed during installation, using a range of jack strokes between 25 and 800mm to install her ICPs to penetrations of up to 8.5m. She found for this relatively permeable, often laminated soil that jack stroke pauses extending beyond about 2min allowed partial excess pore pressure dissipation that could be detected at the pile shaft. Chow experimented with both short (less than 10sec) and extended pauses (40min). The tests with the shortest jack strokes (PT7 and PT6) involved over 300 practically undrained cycles, N_{und} , and relatively few 'dissipation' cycles, N_{diss} , while tests with long stroke jacks involved a far smaller number undrained cycles. Her 'standard' tests involved intermediate ranges for N_{und} and N_{diss} . The results,

Figure 29: Database of ultimate δ values for clays as assessed from ring shear interface shear tests following ICP procedures¹²



which are reproduced in Figure 30, showed steep reductions in K_c with h/R , even in tests where N_{diss} was kept small. However, ‘dissipation’ cycles clearly impacted negatively on K_c and both types of cycle contributed, along with other possible factors, in the reduction of K_c with h/R .

Also shown on Figure 30 are ranges of predictions for K_c based on Lehane’s expression. While the detailed effects of N_{und} and N_{diss} could not be captured, the predictions fell in the correct range even for this partially draining clay-silt. Encouraged by these findings, Chow tested Lehane’s expression further against a database of high quality field tests, making the same R^* substitution as in sands (equations 1 and 2) to allow for open-ended pile conditions and adopting same minimum of 8 for h/R^* . Although generally good results were found, Chow considered that equation 23 would be marginally preferable for practical use. As shown later, the results provided a good fit to her substantial database of high quality field tests in clays.

$$K_c = [2.2 + 0.016 YSR - 0.87 \log S_u] YSR^{0.42} (h/R^*)^{-0.20} \quad (23)$$

Both Lehane’s expression and equation 23 predict that YSR is the most influential factor, followed by h/R^* and finally sensitivity, S_r . Following the arguments previously set out for sand, the h/R^* dependence can be seen as developing due to several processes: geometrical changes in the strain paths experienced as soil flows round the pile tip; the effects on this regime of the pile tip geometry (or area ratio); undrained and partially drained load cycling (or friction fatigue); and short-term stress relaxation (time-dependency of clay behaviour).

The MTD-96 method for clays adopted equations 22 and 23 for shaft resistance and added a simple CPT based approach for end bearing. Chow found that the traditional $q_b = 9S_u$ led to wide scatter. With plugged piles, the mobilisable q_b could be better taken as $0.4q_c$ for undrained conditions, while it was safe to take unfactored q_c over the annular area if the piles cored. Values 60% higher could be taken if the loads were applied sufficiently slowly to allow drainage to take place. These changes do not have a major impact on total capacity though, as end bearing provides a relatively minor component for most piles driven in clays.

Applying the new methods to an extended database of 68 high quality tests that ranged from Gulf of Mexico and Mexico City clays to Japanese diatomaceous mudstone, including the LDPT tests performed in the UK, they confirmed Chow’s earlier assessment that the MTD/ICP-05 approach eliminated API-93’s biases with respect to YSR and L/D and led to markedly better CoVs (see Table 3 and Jardine *et al.*¹²)

Table 3: Summary of statistical analysis of ICP-05 and API-93 pile axial capacity for clays¹²

	Mean Q_c/Q_m	CoV in Q_c/Q_m
Shaft Resistance: ICP-05	1.03	0.20
Base Resistance: ICP-05	1.01	0.19
Shaft Resistance: API-93	0.99	0.33
Base Resistance: API-93	0.83	0.73

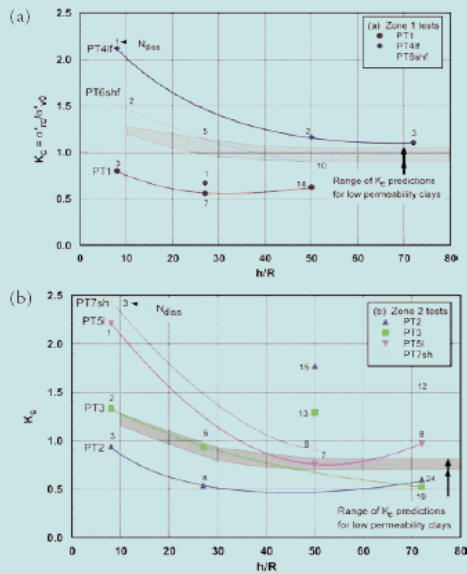


Figure 30: Trends for K_c to fall with h/R in ICP tests at Pentre depending on number of undrained and partially drained ‘dissipation’ cycles (N_{und} and N_{dis}) imposed during installation¹¹

When applying the ICP method, it is essential to design the site investigation to obtain the key input parameters, including

- Site specific interface ring shear tests
- A good assessment of sensitivity, ideally involving both remoulded and peak S_u measurements, as well as intact and reconstituted oedometer tests
- Reliable estimates of YSR. The latter is best obtained through a holistic interpretation of geology, oedometer yield points, high quality triaxial S_u/σ'_{v0} data and *in situ* test profiles.

These data are not always available, even for well established test sites, making database assessments of the ICP-05 clay approach harder to achieve than is the case for the sand method. A feature of note is the difficulty in assessing YSR from undrained triaxial tests on over-consolidated samples that bifurcate during shear. The latter behaviour, which is typical of medium to high plasticity clays, leads to a disconnect between S_u/σ'_{v0} and apparent OCR (or YSR) as illustrated in Figure 31. S_u measurements are highly sensitive to the method of strength determination applied. Sampling disturbance; re-consolidation procedure (isotropic, anisotropic or none); testing style (triaxial compression or extension, simple shear or other); and strain rate can all have important influences, particularly with sensitive anisotropic soils. A similar set of considerations applies to *in situ* testing⁷⁴. Jardine *et al.*¹² recommend that consolidated, anisotropically sheared undrained (CAU) triaxial compression procedures should be followed if possible when applying

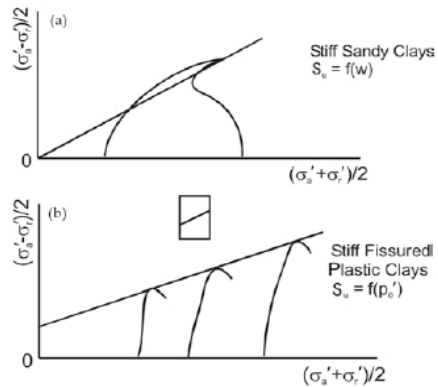


Figure 31: Undrained effective stress paths of (a) stiff clays that tend to critical state and (b) those that bifurcate and progress towards residual shear strength

the ICP for design, along with high quality push sampling with thin-walled, sharp-edged stainless-steel sample tubes.

7. Alternative Approaches for Clays and Critiques of the ICP Methodology

In most database studies, the API-93 clay approach leads to CoVs around half those found for piles driven in sand (see Table 1), and the practical need to update the clay method appears to be less urgent. Nevertheless the field work of Bond¹⁹, Lehane²⁰ and Chow¹¹ challenges the fundamental basis of the current total stress approach and points to areas in which the current approach may be systematically biased. The authors are aware of cases in practice where features such as low δ angles or high L/D ratios have contributed to concern over pile capacities, especially in low OCR deposits. One well known example is the large diameter pile test (LDPT) conducted at Pentre in the late 1980s, where API-93 over-predicted the shaft capacity by around 50% at peak². However, the ICP gives a marginally conservative peak capacity¹¹.

Account was taken of the reductions in σ'_{vc} and hence $\tau_{z\theta}$ with h/R seen in the ICP experiments by Kolk and van de Velde⁷⁵, who proposed a variation of the API-93 method to account for pile length effects, where the average coefficient $\alpha = \tau_{z\theta}/S_u$ depended on the pile length:

$$\alpha = \tau_{z\theta}/S_u = 0.55 (\sigma'_{v0}/S_w)^{0.3} (L/40D)^{-0.2} \quad (24)$$

The above coefficients came from a statistical analysis of 25 driven pile tests and Kolk and van de Velde⁷⁵ found that including L/D in their regression approximately halved the CoV. Naturally, the results show more divergence when adding other data points that are not included in the original regression. The method can be recast in an effective stress format to take account of variations in δ between sites, but it is not clear how the approach should be applied in layered deposits.

Clausen and Aas⁷⁶ performed a capacity assessment for the

API-93 and ICP-05 clay methods. Considering 43 pile load tests at 15 sites and noting significant long term capacity changes with time, they projected ultimate loads to a reference age of 100 days. Their results gave Q_u/Q_m means close to unity for both methods, but with much higher CoVs (0.54 and 0.71 respectively) than other authors. Clausen and Aas attributed the variability to a non-conservative bias for piles driven in low OCR clays of low plasticity, notably at Onsey, Lierstranda and Pentre. When the 11 pile tests performed at these three sites were removed from the database, the standard deviation reduced sharply. They also noted that API predicted pile capacities were between 10 and 30% higher than those calculated using the ICP and NGI procedures.

Ridgway and Jardine⁷⁷ discussed the same issue, noting that part of the scatter found when using the ICP was a lack of the key site investigation data. They referred to Chow's¹¹ analysis to show the ICP does in fact give good predictions for Pentre. However, they also agreed that a class of low OCR sensitive clays does exist in which driven piles mobilise substantially lower medium term capacities than might be expected. While the cases involving plasticity indices, I_p , less than around 17% showed the greatest shortfalls, they noted that the 'problem' clays were not exclusively of low plasticity, and that shortfalls in pile capacity should not necessarily be expected in less sensitive low OCR, low plasticity deposits. Ridgway and Jardine suggested that continuous piezocone test (CPTU) tests may provide the most useful way of identifying potentially difficult soils, showing how the sites with low capacities plot in classification diagrams, as illustrated in Figure 32. It is also interesting that the sensitive 'problem' clays may show capacity growth with time, as suggested in Figure 33^{78,79}, in a similar way to piles driven in sands. Other plots given by Flaate⁷⁸ suggested that capacity growth may continue beyond the three month period shown in Figure 33. It seems likely that the rates of decay of K_c with h/R are far steeper than usual in these 'problem' soils, which may be showing features that are intermediate between those expected from the ICP methods for clays and sands.

Karlsrud *et al.*¹⁴ proposed an NGI-05 clay method that sought to match the trends seen in the Clausen and Aas⁷⁶ database, including the 'problem' cases discussed above. Treating plasticity index, I_p , and S_u/σ'_{v0} as the controlling variables, they developed the curves shown in Figure 34 for the total stress α parameter that have a dramatic impact on low OCR, low I_p clays. NGI noted a generally improved fit to its pile load test database, finding CoV values just below 0.30; Figure 35 shows its improved predictions for the distribution of shaft resistance for the Pentre LDPT test, which is poorly predicted by API-93. ICP-05 gives similarly good predictions for the measured profile, except over the final few metres where the data are uncertain due to field strain gauge failures. In common with API-93, the NGI-05 approach includes no implicit allowance for any effect of relative pile tip depth (b , b/R or h/R^*) that could lead to reductions in σ'_{vc} (and hence $\tau_{z\theta}$) at points above the pile tip.

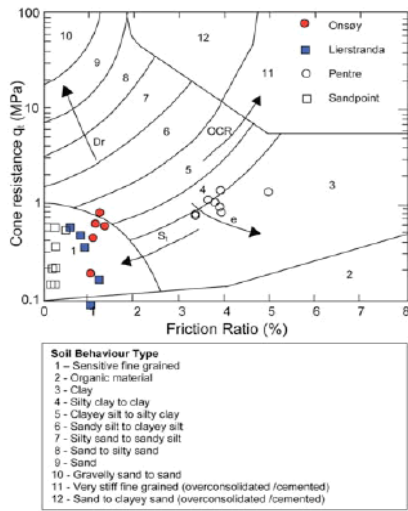


Figure 32: Classification charts showing data points for sensitive low OCR clays from Lierstranda, Onsøy and Sandpoint, where pile capacities are lower than expected. Note that Pentre capacities are well predicted by ICP-05, but not API-93; after Ridgway and Jardine⁷⁷

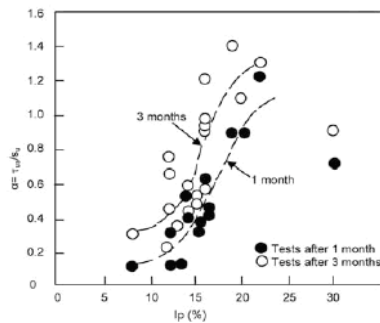


Figure 33: Relationship between α -values and plasticity index from load tests on timber piles in Norway, based on data from Flaate⁷⁸ quoted by Karlsrud et al.⁷⁹ and Ridgway and Jardine⁷⁷. Note that capacities may continue to grow after 100 days

Randolph²⁶ and Randolph *et al.*⁸ speculated that the stress conditions applied to the shafts of offshore piles driven in clay might not be subject to any significant dependence on relative pile tip position. These works considered that partial drainage developing during pile installation could be an important factor that might at least partially explain the h/R trends seen in the ICP experiments, and possibly explain the trends noted by NGI in the low plasticity soils. If any h/R trends could be proven to apply, they needed to

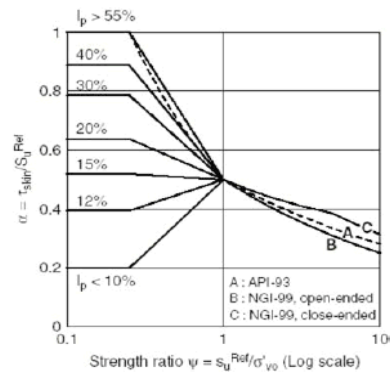


Figure 34: Comparison between API-93 and NGI-99 recommendations for α ; after Karlsrud et al.¹⁴

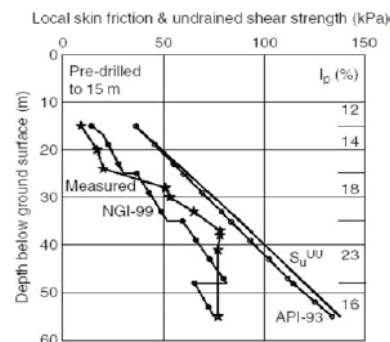


Figure 35: Comparisons between shaft friction values from Pentre LDPT test and predictions by API-93 and NGI-99; after Karlsrud et al.¹⁴. Note ICP-05 also provides good fit to measurements¹¹

be explained and addressed through a framework similar to that proposed by White⁶³ for sands.

It is useful to recall here that the 'friction fatigue' concept cited by White and Lehane²⁷ and others was first coined by Heerema³² to explain the dynamic measurements made during driving offshore piles in clay. While the dependence on h , h/R or h/R^* may be more acute in sands, it is also very clear during driving in clays, which are also known to be affected by the extreme two-way cyclic loading associated with pile driving. Figures 36 and 37 display the interpretations offered by Randolph⁸⁰ of stress wave measurements made on the 762mm diameter LDPT piles at the low YSR clay-silt Pentre and high YSR (glacial till over plastic clay) Tilbrook Grange sites. It is important to recall that Pentre has relatively high permeability, and dissipation is likely to have had a cumulative impact on the upper layers during the 2hr it had taken to drive to 55m penetration, leading to a very strong dependence on h/R (see for example the two-

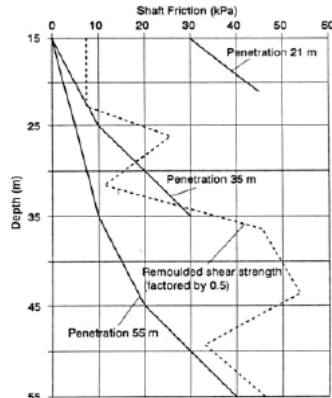


Figure 36: Shaft resistance profiles developed on Pentre LDPT piles during driving interpreted from stress wave measurements by Randolph⁸⁰

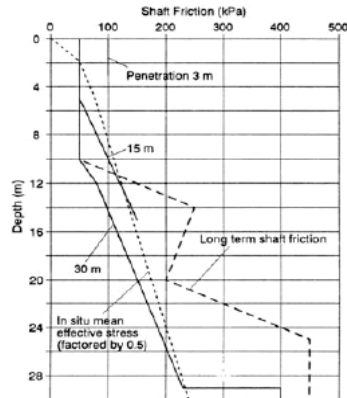


Figure 37: Shaft resistance profiles developed on Tilbrook LDPT piles during driving interpreted from stress wave measurements by Randolph⁸⁰

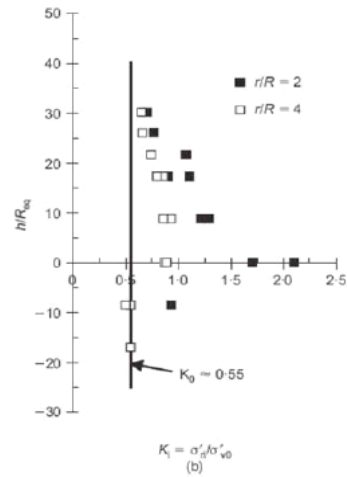
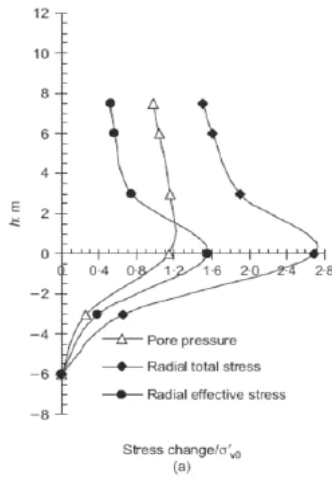


Figure 38: (a) Variation of stress change with h at $r/R = 2$ and $z = 6m$ seen in tests with PCC pile in soft silty Shanghai clay; and (b) variation of effective stress ratio, K_i , equal with h/R^* (note $R^* = R_{eq}$); after Xu et al.⁸²

third reduction in local resistance seen at pile mid-depth). A weaker, but still highly significant, dependence on h/R is apparent for Tilbrook Grange (amounting to around one-third at pile mid-depth), where pore pressure dissipation was very slow and failed to reach completion in the 130 days between driving and testing⁸¹. Naturally, the progressive development of residual fabric in the interface shear zone could also have contributed to the progressive loss in τ_{rz} with h , but the ICP tests showed that such fabric changes would have been completed within the first 0.4m of the pile tip's arrival at the level in question. The continuing changes in τ_{rz} were related to further reductions in σ_r .

It is also interesting to note that at both sites the shaft frictions developed in static tests (performed after periods of pore pressure equalisation) are substantially greater than the 'end of driving' resistances. Correlating the data shown in Figures 36 and 37, the set-up factors can be seen to reduce with depth and rise with h/R at Pentre. The factor of around 5 seen at the mid-depth reduces the effect on final static capacity of h/R . A set-up factor of around 2 applies more evenly to the lower two-thirds of the Tilbrook pile^c.

Xu et al.⁸² reported new field experiments in soft clay (capped by fill) at a Shanghai site with two 1.02m diameter PCC piles, a new type of driven cast-*in situ* foundation, that were installed to depths of 12 and 13.5m. The PCC installation involved vibro-driving a steel hollow annular shell, which was closed at its base during installation and behaved as a thick-walled steel pile, having a D/t ratio of around 7. The piles cored under practically undrained conditions and gave plugs that extended slightly above ground level. Measurements were made of displacements, pore water pressures, u , and radial total stress increments, $\Delta\sigma_r$, in the ground around the pile as it penetrated under vibration without developing the discrete load cycles associated with conventional driving or short-stroke jacking. Assessing values of the undisturbed K_0 radial effective stresses and pore pressures allowed the variations in σ_{ri} and σ_{ri}' with steadily continuing penetration to be assessed as functions of both h and h/R^* .

Figure 38 reproduces the measurements made by instruments located at radial distances, r , of 2 and 4 pile radii, R , at one fixed depth (6m), giving similar data to Chow's experimental results tests in dense sand (see Figure 10). Plotting the results also as a variation of $K_i = \sigma_{ri}'/\sigma_{v0}'$, Xu et al.⁸² noted a variation with h that continued until the pile reached its final depth (with $h = 7.5m$) and was es-

^c The static field measurements made at shallower levels appear anomalous and do not correspond well with a second tension test made at the same site⁸¹

pecially steep at the $r/R = 2$ location. Xu *et al.*⁸² reported that K_c followed a similar form to that anticipated from the ICP K_c expression (equation 23), reducing approximately in proportion to $(b/R^*)^{-0.340.1}$. They also noted good agreement between their field measurements of displacements and porewater pressures and shallow strain path method analyses, which treated the PCC as an equivalent solid pile with a radius of R^* (equation 2).

8. Overall Summary of Effects of Relative Pile Tip Position on Stress Regime Developed around Driven Piles

Considering all of the evidence reviewed up to this point, it is concluded

- The stress conditions developed on the shafts of piles during driving vary with b in both sands and clays.
- The first part of the reduction takes place over the distance required for the strain regime induced by continuous steady penetration to stabilise. Constant volume strain path analyses of 'simple closed piles' suggest that this could be complete within 5 radii of the pile tip, although different distances may be required with sands and more realistic closed- or open-ended pile geometries. Considering the drained response of piles in sand, ICP field measurements show that this part of the process involves around a hundred-fold reduction between the maximum radial effective stress developed near the pile tip and that acting on the shaft at $b/R = 8$. The scope for effective stress reduction is far less in undrained clays and is limited by clay's sensitivity, S_r .
- Further reductions take place, even if the pile is advanced monotonically, as in Gavin and O'Kelly's⁶⁹ tests in dense sand at Blessington, or Xu *et al.*'s⁸² tests in Shanghai soft clay. These features may well be associated with drained or undrained stress relaxation in clays or sands that develop under the kinematically controlled conditions applying near the shaft. It is now known that time dependency is of major importance in sands, and that angular particles are particularly susceptible⁸³. Particle crushing and differential grain sorting could also affect coarse grained soils, as suggested by White and Bolton⁶¹.
- If the pile is driven conventionally, or subjected to short-stroke jacking, the pile shaft experiences extreme load cycling. Further reductions in local radial effective stresses take place under drained or undrained conditions that increase systematically with N_{und} and N_{dis} . It appears that drained, or partly drained, cycling has a more powerful influence because it leads to radial unloading as the surrounding soil mass strains to accommodate the compressive volume strains induced by cycling. This leads to sands experiencing sharper reductions in radial effective stresses with continuing driving cycles than most clays.
- Further changes develop in the stress regime after installation as the result of both pore pressure equalisation and effective stress independent creep phenomena. These lead to positive setup in almost all cases, although short-term minima in axial capacities may be developed in some cases, such as the Cowden ICP tests described

by Lehane and Jardine⁸⁴.

- The medium term profiles of σ'_{rc} developed on the shafts of piles driven in sands and clays are predicted reasonably well by the simple ICP expressions, even though they do not account explicitly for all of the phenomena listed previously.
- When combined with the Coulomb effective stress shaft failure law, allowance for stress changes experienced as the pile is loaded to failure and the appropriate site-specific δ values, good predictions can be made for a wide range of sand and clay conditions.
- Uncertainty remains over intermediate soils, such as clays that can drain partially during driving, or silty sands that cannot dissipate pore pressures fully during installation. Highly sensitive lean or silty clays may develop far lower capacities than might be expected.

9. Field Evidence from Recent North Sea Pile Driving Monitoring

Overy⁶ presented a very valuable summary of pile driving monitoring from nine North Sea platform sites where MTD-96, ICP-05's precursor, was used as the primary design tool. He presented soil resistance to driving (SRD) data from 12 different pile installations covering a wide range of pile diameters (0.66 to 2.134m), lengths (26m to 87m) and soil profiles (with sand dominating in five cases, clays in three and four having mixed profiles). The profiles of SRD with depth were correlated with static capacity profiles generated by applying both API-93 and MTD-05, based on appropriate site investigations. The ICP-05 axial capacities ranged from under 10MN to around 100MN and showed, in some cases, substantial variations from the API-93 estimates. These variations rose above the latter by a maximum of 52% (Clipper PR, in tension) and fell up to 43% below for the Brigantine piles. On average, the ICP led to higher capacities with the piles considered in this particular spread of ground conditions.

Overy concluded that the 'correlation between ICP capacity predictions and dynamic SRD is good, often showing both a quantitative and qualitative match to the field data regardless of pile soil or soil conditions. The same is not true for the API predictions where the best fit requires an inconsistent selection of possible capacities'. Figure 39 reproduces one example from one of the Galleon PG's 1.372m diameter leg piles that was installed in 1996. The ICP curves provide a similarly better fit than API-93 (clay and sand methods) to the SRD profiles in eight out of 12 cases, with the reverse applying in just three. The SRD profiles fall significantly below the ICP predictions in only two instances. Bearing in mind the very probable development of positive setup in both clays and sands, the good agreement between static capacities and SRDs indicates a highly encouraging trend and gives reason to expect fully satisfactory in-service performance.

Overy also described how adopting the ICP/MTD approach has led to significant practical benefits for Shell UK, allowing substantial cost savings in many cases and improved safety in others. It has also allowed for innovative platform design solutions to be pursued in particular projects, such as the Skiff development.

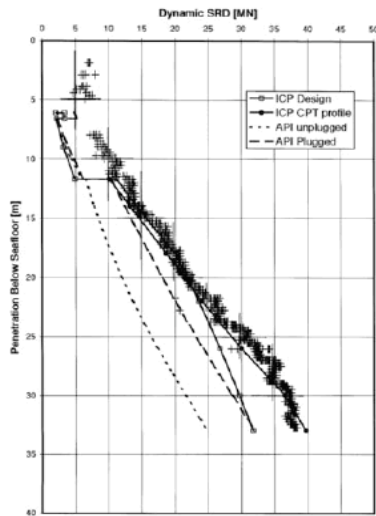
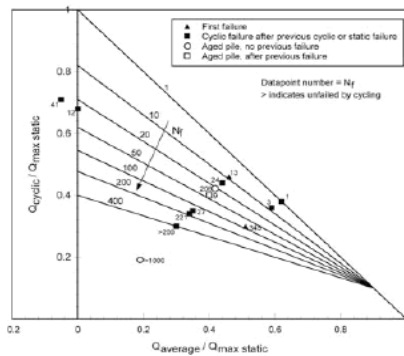


Figure 39: Dynamic soil resistance to driving (SRD) data compared with compression pile capacity estimates by API-93 and MTD-96 approaches. Galleon PG platform leg pile (1.372m diameter) installed in dense sands and stiff clays, North Sea, after Overy⁶

10. Behaviour of Piles in Service

The main part of this paper has focused on improving the ability to predict the outcomes of static pile load tests. While this is vital, given the poor predictive performance of conventional methods, there are many other points of concern in offshore pile piling. No attempt is made here to cover pile driving predictions, it should be noted that Overy⁶ and Overy and Sayer⁸⁵ obtained good matches between field SRDs and MTD-96 predictions, and that the ICP-05

Figure 40: Cyclic interaction diagram from one way tension loading tests on 19m long, 457mm diameter piles, in dense sand at Dunkirk⁴⁴



is often used for driveability predictions in UK practice. Instead, this final section is devoted to a brief discussion of four additional aspects of foundation performance in-service – the effects of cyclic loading; load-displacement behaviour; pile group interaction; and potential effects of disturbance by well drilling – before some final points are made concerning foundation monitoring.

10.1 Response to cyclic loading

The critical design conditions for many offshore foundations involve storm loading. While the design of major gravity-based structures (GBSs) invariably includes demonstrating their ability to stand the cyclic loads imposed by waves, cycling is rarely addressed explicitly in pile design. Historically, engineers have taken comfort in the positive effects of loading rate that apply to clays. Static load tests take between 30 minutes and several days to complete, depending on the constant rate of penetration (CRP) or maintained load (ML) procedures followed – which can have an important effect on the capacities developed^{86, d}. Considering typical wave periods, each storm loading cycle is complete within about 10sec, so the storm loads are applied around 10^2 to 10^4 times more quickly than the static tests and the positive effects are often thought to cancel out the impact of cycling (see for example Bea⁸⁸). Recent work by Saldívar Moguel⁸⁹ has shown that this may indeed occur and lead to a marginally positive capacity bias when even more rapidly applied seismic forces impact on foundations during earthquakes.

Workers in Norway, the UK and elsewhere have demonstrated that piles driven in clay can experience significant capacity losses when subjected to high level cyclic loading^{35, 36, 79, 90, 91}. Practical testing difficulties usually lead to the cycles being applied with periods that are significantly longer than those associated with storms – for example around one cycle per minute in field tests performed by the authors – but the rate effects are likely to be relatively minor.

It is less well appreciated that piles driven in sands can also show large losses in capacity through cyclic loading. Jardine and Standing⁴⁴ reported the only field-scale tests of which the authors are aware. They conducted multiple tests on open-ended, 456mm-diameter, mostly 19m-long pipe piles driven at Dunkirk, France, leading to the cyclic interaction diagram shown in Figure 40. Here, the numbers of cycles required to bring about failure depend on the combination of the normalised cyclic amplitudes, $Q_{cyclic}/Q_{max static}$, and the average (mid-cycle) loading level, $Q_{average}/Q_{max static}$. With both clays and sands, the rates of loss in capacity increase systematically with $Q_{cyclic}/Q_{max static}$ which has a maximum value of around unity (when $Q_{average}/Q_{max static} = 0$) for extreme two-way loading. The latter takes the shaft from compression failure through to tension failure with each cycle and leads to rapid losses of resistance. Under one-way loading (in tension or compression) $Q_{cyclic}/Q_{max static}$ is limited to a maximum of around 0.5, which applies when $Q_{average}/Q_{max static}$ is also around 0.5; degradation rates are sub-

^d Still more significant effects can be seen when the fast loading STATNOMIC system is employed⁸⁷

stantial, but lower than under two-way extreme loading. It is not wise to rely on significant base resistance under cyclic loading in most cases, because the shaft movements required to mobilise the base lead to extreme two-way shaft loading. The same process applies during pile driving and is one of the causes of the 'friction fatigue' contribution to the b/R^* effects discussed earlier.

Severe cyclic loading effects are unlikely with heavy jacket structures if they experience relatively low $Q_{cyclic}/Q_{max static}$ (and moderate $Q_{average}/Q_{max static}$) levels that keep the piles within the relatively stable region of their cyclic interaction diagrams. Jardine and Standing⁴⁴ even found that low level cyclic effects could be beneficial in sands by accelerating the ageing processes that lead to capacity growth with time. However, it is advisable to check the potential impact of cycling at the design stage, particularly with lightweight platform designs that may attract higher normalised cyclic amplitudes. Randolph *et al.*⁸ summarised the RATZ methodology for tackling cyclic loading effects on pile behaviour. Jardine *et al.*¹² offer another simple approach that has been applied by the authors in several UK offshore sector developments.

10.2 Load-displacement behaviour and pile group interactions

The offshore industry's standard practice of applying T-Z and P-Y analyses for single piles and elastic interaction factors for pile groups has changed little in recent years. Field measurements are rarely made of foundation response to static or storm loading, but when they have been made large discrepancies have been found, particularly for pile groups such as those installed in the 1980s at the Magnus and Hutton North Sea sites. Jardine *et al.*³² described the instrument system outlined in Figure 41, which they had designed to resolve pile group vertical movements to within $\pm 0.03\text{mm}$ on the seabed. They also presented the vertical stiffness and moment rotation data recorded at Hutton

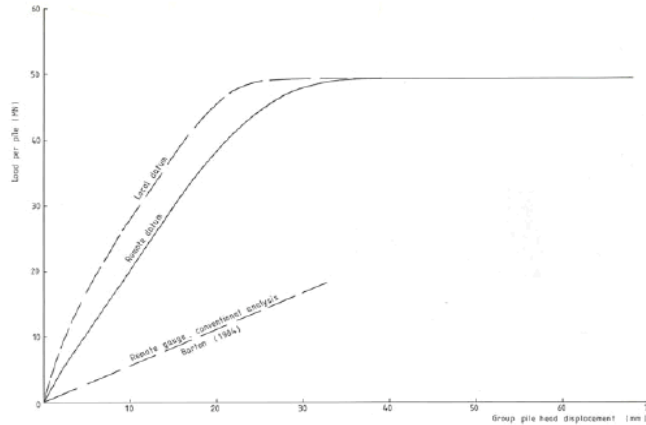
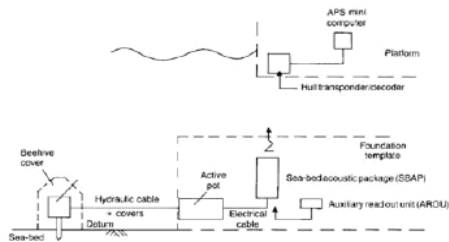


Figure 42: Predictions for axial load-displacement behaviour of Hutton TLP pile groups by (i) conventional design approach and (ii) non-linear FE analysis³⁴. Note that latter predictions refer to both remote and local measuring datums employed for monitoring system illustrated in Figure 41

during the installation of the first TLP. Sharp³⁵ reported dynamic measurements of the axial and lateral foundation stiffnesses developed by the Magnus foundations during storm loading. In both cases the axial stiffnesses were four to five times greater than expected from the conventional design procedures, while the lateral movements for Magnus and the moment-rotation stiffnesses for both Hutton and Magnus were under-estimated by factors of 2 to 3. Jardine and Potts^{34, 35} reported 'Class A' FE analyses of the Hutton and Magnus foundations that predicted the field measurements within around $\pm 25\%$ for both sites. Figure 42 summarises the differences between predictions of the Hutton tension leg platform (TLP) pile groups' axial stiffness response that were made (i) by the designers using conventional T-Z analyses and elastic pile group interaction factors and (ii) the non-linear FE analyses. The key features incorporated into the latter were

- The succession of strata and their detailed variations in mechanical properties
- The effective stress level dependence and extreme stiffness non-linearity shown by the glacial, interglacial and glaciomarine soils from very small strains, when applying models that could capture the trends seen in advanced laboratory stress path testing performed on samples from the site, or in similar strata from the same province
- Potentially brittle and progressive local failure along the pile shaft, which develops from a relatively early stage of loading when considering long offshore piles. This requires an effective stress procedure to predict the local shaft resistance reliably. The developments outlined in the main part of the paper would now allow this part of the analysis to be improved significantly
- Use of a robust and flexible FE code employing the principles outlined by Potts and Zdravkovic³⁶.

Figure 41: Vertical movement monitoring system developed for foundations of Hutton TLP³²



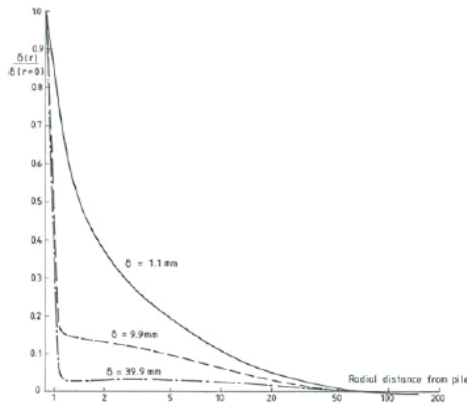


Figure 43: Profiles of vertical movements δ_r , expected at given radial distances, r , from single pile centre normalised by pile head movement $\delta_{r=0}$ for three stages in pile loading to failure, as predicted by fully non-linear FE analysis for Hutton TLP⁹⁴

Ganendra⁹⁷ and Potts and Zdravkovic⁹⁶ showed that the observed Magnus lateral loading response could also be predicted much more accurately through a three-dimensional extension of the same non-linear techniques, allowing for failure in the soil continuum around the piles and leading to better predictions for both displacements and bending moments induced in the piles.

The small-strain analytical techniques applied to the Magnus and Hutton cases, as well as alternative approaches such as those described by Simpson⁹⁸ or Whittle *et al.*⁹⁹, are now applied routinely to model foundations, deep excavations and embankments, and other onshore applications¹⁰⁰. Their use may be warranted in a wider range of offshore piled foundation projects, particularly when considering issues such as the fatigue of steel structures, or the need to set criteria for in-service monitoring of platforms. Such measurements may be desirable when installing novel foundation systems, dealing with unfamiliar soil types, or considering the potential impact of negative phenomena such as well drilling disturbance¹⁰¹.

10.3 Group action effects

The non-linear features emphasised previously are particularly important when considering pile groups. Figure 43^e shows how the ground surface vertical movement profiles expected around a single Hutton pile undergoing tension loading is likely to change in shape as the pile head movement grows and the FoS falls. The non-linear effects occur from even very small movements, leading to far smaller settlement interaction factors that are expected from elastic analyses.

^e Where δ_r is the surface vertical movement at a radial distance, r , from the pile; $\delta_{r=0}$ is the pile head movement; and R is the pile radius

Field tests with groups of five 6m-long, 0.2m-wide, driven concrete piles have identified some other negative aspects of pile group interaction that are rarely considered in offshore design. Lehane *et al.*^{102, 103} reported that the interacting shear stress fields developed around the piles led to

- Group static capacities that were significantly less than the sum of the individual piles and failures that did not involve the equivalent caisson mechanism that is usually considered in offshore analysis
- A greater susceptibility of the pile group to cyclic loading, with more severe cyclic degradation developing under given loading levels than was seen with single piles.

Figure 44 illustrates the latter feature, showing how the single pile and pile group capacities, as assessed in static tests conducted after the end of load cycling, varied according to the maximum level of cyclic loading that had been applied previously. The exacerbating effects of group action are clear and may be important in offshore pile group design, which frequently employs closer group spacing ratios (s/D , where s is the pile to pile spacing) than equivalent onshore pile designs. Offshore groups may therefore be subject to potentially significant negative capacity interactions in clays¹².

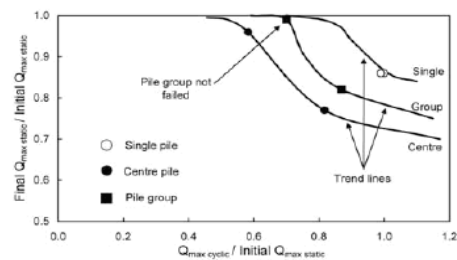
Chow⁹⁴ showed that in sands the interactions can be more positive, leading to overall gains in shaft capacity. However, mobilisable base resistance was reduced very significantly by installing neighbouring piles.

10.4 Potential effects of pile disturbance by well drilling and other activities

The main emphasis of this review has been on improvements in understanding the factors that control the axial capacity of driven piles. Overy and Sayer⁸⁵ discussed how they applied the MTD-96 approach to consider rationally the negative effects of drill-driving on capacity. Pre-drilling in advance of driving led to a carefully designed and deliberate reduction in axial capacity in very dense sands, which enabled a new platform concept to be realised as part of the Skiff development in the UK North Sea.

It is appropriate to consider also how similar operations might affect capacity in unanticipated ways in cases where pre-drilling, jetting or vibration is applied to case conditions

Figure 44: The negative effects of group action on cyclic degradation of axial capacity. Results from field tests on group of five concrete piles (6m long, 0.2m wide) driven in soft clay; after Lehane *et al.*^{102, 103}



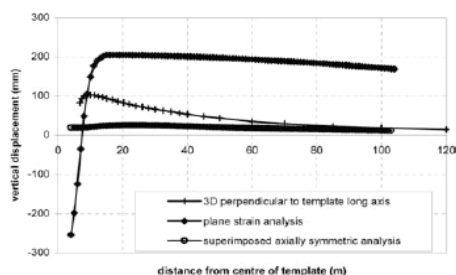


Figure 45: Non-linear FE predictions for vertical ground surface movements caused by drilling disturbance; after Schroeder *et al.*¹⁰¹

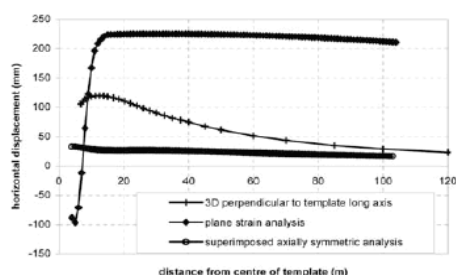


Figure 45: Non-linear FE predictions for horizontal ground surface movements caused by drilling disturbance; after Schroeder *et al.*¹⁰¹

when pile driving proves harder than expected. Although model studies reported by Jaime *et al.*¹⁰⁴ suggest that pre-boring effects can be very significant in clays, the authors are aware of cases where consultants have advised that pre-boring operations might be adopted while stating that they should not affect capacity provided the work is controlled carefully. Measures designed to ease driving were frequently applied until offshore pile driving hammers reached the capacities that are now available. The arguments presented in this paper all point towards such remedial measures downgrading axial capacity. Careful investigations employing advanced numerical methods, centrifuge studies or even better field trials would be needed to be certain of the potential impact of pre-drilling, jetting or vibration on static capacity.

Other forms of disturbance can apply during a structure's service life. Schroeder *et al.*¹⁰¹ discussed how hydraulic fracturing, associated with 'packed-off' conditions in wells drilled in collapsing clay, affected the capacity of nearby conductors at one site and may have posed a risk to nearby foundation piles. Applying a fully non-linear FE analysis, they considered the possible impact on pile capacity and ground movements of the hydraulic fracturing process involved and subsequent drilling water flows into the ground. Ground movements were considered the most serious issue, and their prediction required an advanced coupled analysis using non-linear models, similar to those described previously. Figures 45 and 46 reproduce Schroeder *et al.*'s¹⁰¹ vertical and lateral ground movement predictions, indicating levels that might affect the platform's soil-structure interaction. While the movements proved to be sensitive to the geometrical modelling of this complex problem, it is interesting that two-dimensional plane strain and axisymmetric analyses provided upper and lower bounds to the results found from full three-dimensional analysis.

10.5 Foundation monitoring in service

It is common in onshore geotechnics to monitor the behaviour of expensive prototype structures. Although this practice is rarely followed with piled offshore foundations, it may be appropriate to consider foundation settlement monitoring in-service in cases that might involve: novel types of foundations; 'unusual' soils; or potential difficul-

ties such as greater-than-anticipated loading or the possible impact of some form of disturbance to the foundations. High resolution settlement or uplift monitoring has been technically possible since the 1980s, through systems such as those shown in Figure 41. Lower resolution GPS based systems are now available that could provide useful platform level information in certain cases. Schroeder *et al.*¹⁰¹ described how crucial information was obtained regarding the extent of the drilling disturbance at one location, with site measurements allowing the numerical analyses undertaken to be at least partially calibrated and validated.

In the same way, planned sequences of trial pile driving and re-strike tests may be invaluable when considering potentially 'problematic' soils as a means of managing risk during critical projects. Incorporating pull-tests on redundant piles during platform decommissioning could also provide critical evidence regarding the ageing characteristics of piles in service that could have considerable impact when considering whether existing facilities can be reconfigured safely and economically. In general, significant uncertainties remain regarding the behaviour of offshore piles, and field observations provide crucial 'reality checks' for offshore geotechnical engineers.

11. Summary and Conclusions

This paper has reviewed the research and debates that have led to substantial changes being made to the API-RP2A recommendations for pile axial capacity. The reasons for the conventional Main Text method's large scatter, strong skewing and significant biases have been explored, with particular emphasis on piles driven in sand. Recent alternative frameworks have been reviewed critically, and the incorporation of four new CPT based sand methods into the new Commentary section has been discussed in detail. A case has been made for making changes to the design recommendations for clay, and consideration has been given to a range of other significant issues.

The main conclusions are as follows:

1. The conventional methods of offshore pile design are subject to considerable scatter, strong skewing and significant bias, particularly when considering sands. The methods,

which are not normally tested offshore, often give poor predictions for pile capacities when pile load tests are performed for onshore projects or research studies.

2. The low incidence of reported offshore pile foundation failures may reflect unaccounted for features of behaviour, such as pile capacity growth with time. Other explanations include potentially lower-than-expected service loads, system redundancy or a possibly conservative bias in the conventional API methods towards the soil conditions encountered in certain offshore provinces.
3. A considerable volume of research has been conducted over the last 20 years that has led to a spread of new approaches for pile design. Field tests with instrumented piles have been particularly important in this research.
4. A vigorous debate has ensued between the advocates of the various methods. A database study has been conducted by a team from UWA considering piles in sands that concludes that the UWA-05 and ICP-05 approaches give the best reliability parameters. However, these two methods apply different weightings to factors relating to open-end conditions and 'friction fatigue,' and do not give coincident results when applied to the same soil profiles.
5. The 2007 API-RP2A recommendations have been modified cautiously to reflect the range of opinions expressed in the recent debate. While the new Main Text method for sands is a modified version of the conventional approach, practitioners are encouraged to consider the four CPT based methods that are set out in the Commentary for sand. No change has been made for axial capacities in clay.
6. A simplified version of the ICP procedures developed by the authors and their colleagues is included in the new API-RP2A Commentary for sands.
7. Arguments have been presented in this paper to show that the ICP-05 procedures for clays and sands can be used safely without the recommended simplifications or modifications. A form of the ICP-05 has been in use for over a decade, and analyses of instrumented driving records support the methods' application in practice.
8. All four of the CPT based methods address pile end details and allow for the reduction of stresses with pile tip relative depth, b , in some fashion. This paper argues that cyclic action, or 'friction fatigue', is only one of the factors that contributes to the latter process.
9. It has also been argued that there are shortfalls in conventional practice for predicting load-displacement behaviour, assessing capacity loss under load cycling considering group interaction effects and gauging foundation disturbance due to drilling. All of these factors may significantly impact the design and management of offshore piled foundations.
10. Additional monitoring of field performance in uncertain areas and field measurements of the ageing trends shown by offshore piles would be highly beneficial.

Acknowledgments

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Appendix

Notes from interview with Mr John Price – Independent Consultant, conducted on 27th March 2007 to assess current practice and thinking relating to piled foundation design in UK Sector, North Sea

Mr Price has extensive experience of checking and certifying offshore foundations and the interview concentrated on his experience of current UK North Sea Practice.

Notes were made on a number of areas as given below:

1. Site investigation requirements: boreholes, sampling types, soundings, lab testing etc

- **Boreholes** New build structures: depends on structure scale and soils encountered., but minimum of one borehole per platform – with a mix of sampling and CPT. Good practice to have separate boreholes for each 10m apart.
- Use of geophysics – shallow seismic survey beneficial, but not usually stipulated, although often present for pipe-line or other purposes. Deeper seismic data is available from exploration activities, also bathymetric plus side scan survey for debris etc. SPTs are ‘never’ performed in N Sea.
- For bigger platforms, or complexes, probably would want four to five boreholes.
- Single multi-purpose borehole for well-head platforms. Alternate CPT-sampling coverage.

- **Sampling.** Percussive sampling only used in cases where recovery is not possible with pushed sampling. If an unusual CPT trace is encountered should consider going back and sampling.
- Intelligent real-time guidance during SI helps.
- Rotary sampling can be applied in soft rocks, (76mm or smaller diameters); hard to do with Chalk (prefer CPTs), including mudrocks. Trimming of softened zone not usually done; rotary can be difficult to

get down hole in some cases. Thin walled hydraulic 76mm thin walled tubes, 5 degree cutting edge design not common.

- **Testing.** Clays; UU triaxial; CAU tests usually reserved for gravity based platforms. CIU tests. Su values can vary hugely between test types but not addressed when applying empirical relationships.
- Spreads of index tests and oedometer tests (CRS tests) are usually commissioned.
- Cyclic testing is almost never done.
- With sand layers, triaxial and shear boxes, grading curves are performed. Interface shear tests are performed, but not usually ring shear.
- Unclear over standard practice with interface types and roughnesses.
- **Re-assessment** of existing foundations: depends on scale, soils and age. New boreholes are uncommon for re-assessment, even with old platforms and very old SIs.
- **Re-interpretation** is carried out of 'old' soil data, particularly selected design shear strength profile, submerged unit weight, cone penetration test data and soil/steel interface friction angles and other parameters, depending on the pile design method.

2. Soils that need special SI treatment

- **Laminated clay/sands**
Low Su values may be seen due to disturbance. Ignoring some lab test results may be part of the interpretation, nothing special is done routinely.
- **'Low plasticity clays'**
FMclelland '92 method which is a downrated API main text type approach is sometimes applied to design, based on Pentre LDP experience. Sometimes used for calcareous clays/silts. Not applied widely in N Sea, some concern over PI range 10 to 25%. Mostly, addressed by standard API, or 'N Sea' variant

- **Loose sands**

Not common, nothing special done – usually limited depths

- **Calcareous sands/clays**

Not common, CaCO₃ normally shells or chalk nodules. FMclelland '92 method may be used for clays.

- **Soft rocks, cemented soils**

Morecambe Bay, Mercia Mudstone – UCS tests on rotary cores, CPT if possible. Chalk CPTs if possible (then use CIRIA K tan delta approach, originally SPT linked with limiting tau (100 or 120 kPa) Hammer/push samples if rotary coring is not possible.

- **Fissured plastic clays (with slickensides?)**

Low Su values due to disturbance – interpretation may involve ignoring some lab test results, all part of standard practice, nothing special done routinely.

- **Very dense tills (as in Clair field West of Shetlands)**

Nothing special done routinely. Very high strengths at other locations – eg Beryl platform area. Near surface K tan delta limits sometimes limit. API naturally produces very low near surface alphas. Traditional to take alpha = 0.5, as at Magnus.

3. **Design issues for single piles: Shaft and base resistances for piles in sands**

- **Current standard practice – methods, use, FoSs etc**

Shaft resistance K = 0.5 tension, 0.7 compression, API limits (Fugro UK). Some contractors use standard API, K = 0.8 for both (eg Fugro Holland). Formerly different limits 120 kPa, 15 MPa base, above standard API (115 kPa and 12 MPa base). Less worry on achieving penetration depths now, as hammers are more capable. Less emphasis on limiting depths to absolute minimum. FoS 1.5 for extreme and 2 for operating; or LRFD methodology.

- **New API/ISO variants for sands – comments? Changes to SI? etc**

ICP used systematically and historically only by Shell and NAM (or do they use Fugro Holland?) No use yet of other methods, such as UWA-05, Fugro-05 or NGI-05 for sands. Need to address interface shear

needs for ICP, essential to have near continuous CPT profile. Possible problems with frequent CPT refusal. Use of limiting values, high capacity cones (5cm²), interpretation of very high values. Continuous profiles versus 3m isolated drives. Not used as much because of engineer time involved. Plus the learning curves. FoS values to be different: reliability based approach allowed.

4. **Shaft and base resistances for piles in clays: Currently standard practice – methods, use, FoSs etc**

- Main text API approach is standard, sometimes modified to take $\alpha = 0.5$. Alternative methods: ICP, Kolk and van de Velde (1997); NGI approaches etc. ICP automatically includes length effects. K&V approach unusual in North Sea. John Price has also been using NGI (2005) approach, which takes much lower alphas for low plasticity clays.
- ICP restricted to Shell (Clair for BP) but otherwise unusual. SI implications. Must have interface ring shear tests (client resistance); different results according to calculation route (sensitivity parameters). Can confuse unless carefully done; hopefully clarified in 2005 ICP edition.
- **Special considerations** – cyclic effects; ageing; scale or length effects
Cycling not usually addressed, but has been requested in chalk (see Fugro papers) and in some Shell and BP cases. Ageing not usually addressed. Falling branch T-Z analysis (0.8 factor on residual) tends to cut capacity. Occasionally used, but with some logical inconsistency.
- Standard practice is to work from ‘moderately conservative best-estimate’ interpretation of SI. Other approaches include strong soil – weak soil combinations, different cases considered.
- Scale effects; none applied for diameter. Sometimes a length effect is considered (as per API), but rarely. Rigden type clay model doesn’t come into play with 2m piles until very long (>70m).

5. Is re-assessment any different to new build?

- As noted earlier, SI re-interpretation often made. Other aspects that can be reviewed include the pile driving records in terms of blowcount and hammer efficiency, preferably from pile/hammer instrumentation data, versus depth of penetration. May be possible to determine strata changes due to changes in blowcount and enable some re-interpretation of strata thicknesses and penetration into an end bearing stratum.
- Reviews also often refer to similarities between site soil conditions and soil conditions for specific pile load tests – may be possible to increase pile capacity on a one-off basis based on high quality pile load test data. Consideration may be given to time effects on pile capacity.
- The platform design loads may also be reviewed and may be down graded if appropriate. It is possible the platform has been analysed for omni-directional wave heights rather than directional wave heights, which could reduce pile loads for critical piles. Review of platform weight and assumptions made during design for 'blanket' loading on lay-down and work areas.

6. Can overall foundation capacity ever be improved in critical cases?

- Possible methods of increasing overall platform capacity include utilisation of mudmats (North Rankin), installation of pile plugs to increase end bearing capacity, infilling of scour pits to re-establish overburden pressure/provide scour protection, insert piles, 'piggy-back' piles (Valhall), belled footings at pile tip and ballasting of piles to reduce critical tension loads. Use of mudmats may be a problem due to relative rates of mobilisation of soil resistance between pile and mudmat and creep. Not usually accounted for, problems with scour and certainty of connection to piles. In the Magnus FMP study a fair proportion of the mudmat load reverted back into the pile at depth (RJJ).

- Differences of opinion exist as to the benefit of pile plugs – should they be at the tip or can they be at the top of the soil plug (mobilisation issue). Insert piles and belled footings can really only be installed on existing platforms with skirt piles unless work is done soon after pile installation before topsides are placed.

7. Axial and lateral load-deflection analyses

Piles in sand

- Standard T-Z and P-Y analyses undertaken
- Continuum analysis is unusual, although is applied for suction caissons

Piles in clay

- Broadly as above, P-Y uses Matlock Cyclic curves
- Some Fugro modifications for stiff clays, noting a historical mismatch between predictions and measurements

8. Potential group effects on capacity and load-displacement behaviour

Piles in sand

- Mainly small groups or single piles in modern practice, large (8, 9 pile) clusters unusual
- Equivalent caisson approach remains most common design method, no interaction through soil mass. P and Z modifiers used for load-deflection analysis
- Pile group interaction factors are usually linear elastic. No account of non-linearity effects on movements, excessive displacement interaction predictions or erroneous shaft load distributions

Piles in clay

- Broadly as above, but note that pile group effects on capacity are more significant than in sand.

9. Installation issues: comments on current practice

- North Sea piles up to 102” (2.5m), 54” (1.25m) is most common, some down even to 36”
- Wind turbine foundations larger 3 to 4m

- Wall thicknesses typically $t/D = 27$, giving 40 to 50mm. Stress relieving needed for welds once $WT > 50\text{mm}$
- API WT rules give thinner tubes, considered non-conservative
- Buckling capacity can be degraded by dents sustained during handling, or when rocks/boulders are encountered
- Stepped piles are a problem
- Pile tip shape can also be a problem
- Chamfered piles driven at Triumph, with big refusal problems (also RJJ experience)
- Move in practice towards square ends to avoid buckling
- **Estimating SRDs, sizing hammers etc?**
SRD assessment typically by Alme and Hamre, or Toolan and Fox approaches. Predictions often quite different from field experience
Very low blow counts with modern hammers – hard to tell much.
- **Critical cases: problems of refusal, or buckling, or flutter?**
Driving into rock is main cause of problems
- **Use of pile instrumentation and driving records to check design**
Benefits are that you have checks that pile isn't damaged and the hammer is working well. Doubtful that static capacities can be predicted for tubular piles from pile driving monitoring. One problem is that driving takes place without plugging, while static behaviour is invariably plugged. Not always clear whether refusal is due to ground conditions or due to dynamic behaviour of hammer and/or followers.
- **Set up and ageing; SRDs, re-drives and ultimate capacity?**
Re-drives are used to give qualitative assessment of capacity-time trends. Link to long term capacity is not clear

10. Well-drilling problems – affecting foundation behaviour

- Paper prepared by Hobbs to 1998 SUT Conference.
- Problems in sand involve washout of sand layers, can be due to high (artesian) water pressures in sands in some cases (eg GoM)
- Elevated drilling fluid pressures can also cause hydraulic fracturing problems in clays Tend to control by adoption of suitable conductor to pile spaced – Shell criteria via empirical equation?

- Some critical cases have been investigated, showing evidence of reduced CPT q_c values. Can end up grouting up sand layers, leaving hard horizons
- Main problems emerge from loss of circulation, also drilling ahead of conductors and creating enlarged holes that may then have ground improvement by grouting
- Can do check SI's from platforms through vacant well template slots
- Extreme case was Shell Leman blow out case in 1960s

11. Accidents? Such as interaction with Jack-up spud cans

- Studies prompted by queries to Amoco.
- Rule of thumb guidance is for edge-to-edge space to equal can diameter. Now tending to move closer, if supported by analysis or checks. Typical checks involve PLAXIS runs and T-Z analysis, axial loads can be significant if jack up can is close to piles
- Reverse problem is also significant – problems with extraction especially with soft clays

12. Field monitoring of any critical cases

- Most platforms were monitored for tilt in the “early days”, this being checked every two years.
- Air gap monitoring is also undertaken by seabed pressure measurement. Vital readings at Ekofisk doubted until late on.
- Structural monitoring has been undertaken, measuring natural periods etc or strain gauging members.
- Magnus and Hutton TLP exercises (RJJ) Also NGI monitoring of Beryl structures.
- Problem is looking at data – what to do if criteria are exceeded

Review of technical issues relating to foundations and geotechnics for offshore installations in the UKCS

Foundation design and especially pile design and analysis are currently undergoing an important stage of technical development, with new methodologies and recommendations coming into practice. Detailed guidance on technical issues and best practice recommendations are provided in Parts 1 to 3 of this Review on the critical design issues and topics that need to be addressed in both site investigation and re analysis. Consideration is also given to possible monitoring and strengthening of foundations systems. The Parts also provide lists of relevant publications and useful references to background material and guidance on specific topics.

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