DESIGN OF PILES IN SAND
IN THE UK SECTOR OF
THE NORTH SEA

Prepared by the Geotechnical Consulting Group for the Health and Safety Executive
DESIGN OF PILES IN SAND IN THE UK SECTOR OF THE NORTH SEA

Authors

A J Bond, D W Hight

Geotechnical Consulting Group Ltd
1a Queensberry Place
London SW7 2DL

R J Jardine

Imperial College of Science, Technology, and Medicine
Department of Civil Engineering
Imperial College Road
London SW7 2BU

HSE BOOKS

Health and Safety Executive - Offshore Technology Report
This report is published by the Health and Safety Executive as part of a series of reports of work which has been supported by funds provided by the Executive. Neither the Executive, or the contractors concerned assume any liability for the report nor do they necessarily reflect the views or policy of the Executive.

Results, including detailed evaluation and, where relevant, recommendations stemming from their research projects are published in the OTH series of reports.

Background information and data arising from these research projects are published in the OTI series of reports.
SUMMARY

This Report critically reviews current practice in the design of piles in sand in the UK sector of the North Sea. The Report concentrates on the static axial load capacity of single piles and is biased towards the design of piles for steel jackets, these being the prevalent type of offshore structure in the North Sea.

Sands are present in almost all parts of the North Sea, either as a thin surface layer, interbedded with clays, or as the predominant soil type. In the northern North Sea, stiff to very stiff overconsolidated clays are in abundance, although in many areas they are interbedded with dense fine sand. In the central North Sea, interbedded clays and sands predominate, whereas in the south there is a large tract of mainly fine to coarse sand.

Recent research into the behaviour of displacement piles in clays and sand has led to an improved understanding of the mechanisms that control the axial capacity of piles, and in particular the mobilization of shaft resistance.

The design of piles in the North Sea is, with minor variations, currently based on the 15th Edition of the American Petroleum Institute's Recommended practice for planning, designing and constructing fixed offshore platforms (API RP2A).

Design parameters for offshore sands are usually determined from a combination of in situ tests and laboratory tests on reconstituted material.

New design methods, which have arisen from recent research, provide improved correlations with the results of existing pile tests in sand. Although the new methods of designing offshore piles are based on fundamental principles, careful checks are needed to ensure they are fully applicable to offshore design. The new Imperial College method currently offers the most promising improvements in predictive capability.
ACKNOWLEDGEMENTS

The Authors would like to thank the following individuals and organizations for providing valuable information or assistance during the preparation of this Report:

Ms Helen Edmonds of GCG
Mr Chris Menkiti of GCG
Mr Rob Nyren of GCG
Dr Robert Overy of Shell UK Exploration and Production
Mr Mike Sweeney of BP International Ltd
Mr Martin Thompson of the Health and Safety Executive
# CONTENTS

## SUMMARY

## ACKNOWLEDGEMENTS

### 1. INTRODUCTION
- 1.1 OUTLINE OF THE REPORT 1
- 1.2 AREA OF INTEREST 2
- 1.3 FOUNDATION TYPES 4
- 1.4 SOIL TYPES 5
- 1.5 SITE INVESTIGATION 6
- 1.6 METHODS OF DESIGNING PILES IN SAND 10
- 1.7 FUTURE TRENDS 11
- 1.8 REFERENCES 12

### 2. GEOLOGY AND GEOGRAPHY OF NORTH SEA SANDS
- 2.1 INTRODUCTION 15
- 2.2 GEOLOGY OF NORTH SEA SANDS 15
- 2.3 DISTRIBUTION OF SANDS IN UK SECTOR OF NORTH SEA 22
- 2.4 OCCURRENCE 27
- 2.5 REFERENCES 28

### 3. FUNDAMENTAL BEHAVIOUR OF DRIVEN PILES IN SAND
- 3.1 THE NEED TO UNDERSTAND FUNDAMENTAL BEHAVIOUR 35
- 3.2 INSTALLATION 37
- 3.3 EQUALIZATION 47
- 3.4 LOADING 49
- 3.5 LOAD-DISPLACEMENT BEHAVIOUR 54
- 3.6 REFERENCES 56

### 4. EXISTING DESIGN PROCEDURES
- 4.1 INTRODUCTION 61
- 4.2 SHAFT CAPACITY OF SINGLE PILES 62
- 4.3 END BEARING CAPACITY OF SINGLE PILES 70
- 4.4 LOAD AND RESISTANCE FACTOR DESIGN 78
- 4.5 REFERENCES 81

### 5. RECENT DEVELOPMENTS IN DESIGN METHODS FOR DRIVEN PILES IN SAND
- 5.1 INTRODUCTION 87
- 5.2 NEW METHODS FOR CALCULATING
<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>END-BEARING RESISTANCE</td>
<td>88</td>
</tr>
<tr>
<td>5.3 NEW METHODS FOR CALCULATING SHAFT RESISTANCE</td>
<td>91</td>
</tr>
<tr>
<td>5.4 FUTURE NEEDS</td>
<td>101</td>
</tr>
<tr>
<td>5.5 REFERENCES</td>
<td>101</td>
</tr>
<tr>
<td>6. DESIGN PARAMETERS FOR NORTH SEA SANDS</td>
<td></td>
</tr>
<tr>
<td>6.1 PARAMETERS RELEVANT TO THE DESIGN OF PILES IN SAND</td>
<td>105</td>
</tr>
<tr>
<td>6.2 STRATIGRAPHY</td>
<td>107</td>
</tr>
<tr>
<td>6.3 GRADING, GRAIN SHAPE, AND CRUSHABILITY</td>
<td>107</td>
</tr>
<tr>
<td>6.4 IN SITU DENSITY, STATE, AND STRESS HISTORY</td>
<td>111</td>
</tr>
<tr>
<td>6.5 DEFORMATION PARAMETERS</td>
<td>117</td>
</tr>
<tr>
<td>6.6 FRICTIONAL RESISTANCE</td>
<td>122</td>
</tr>
<tr>
<td>6.7 PERMEABILTY</td>
<td>132</td>
</tr>
<tr>
<td>6.8 REFERENCES</td>
<td>132</td>
</tr>
<tr>
<td>7. USING THE METHODS IN PRACTICE</td>
<td></td>
</tr>
<tr>
<td>7.1 INTRODUCTION</td>
<td>137</td>
</tr>
<tr>
<td>7.2 INTERPRETATION OF PARAMETERS</td>
<td>142</td>
</tr>
<tr>
<td>7.3 API RP2A</td>
<td>144</td>
</tr>
<tr>
<td>7.4 TOOLAN, LINGS, AND MIRZA (1990)</td>
<td>146</td>
</tr>
<tr>
<td>7.5 IMPERIAL-COLLEGE/GCG APPROACH</td>
<td>148</td>
</tr>
<tr>
<td>7.6 RANDOLPH, DOLWIN, AND BECK (1994)</td>
<td>150</td>
</tr>
<tr>
<td>7.7 COMPARISON OF RESULTS</td>
<td>151</td>
</tr>
<tr>
<td>7.8 CONCLUSIONS</td>
<td>153</td>
</tr>
<tr>
<td>7.9 REFERENCES</td>
<td>154</td>
</tr>
<tr>
<td>8. CONCLUSIONS</td>
<td></td>
</tr>
<tr>
<td>8.1 INTRODUCTION</td>
<td>157</td>
</tr>
<tr>
<td>8.2 GEOLOGY AND GEOGRAPHY OF NORTH SEA SANDS</td>
<td>157</td>
</tr>
<tr>
<td>8.3 FUNDAMENTAL BEHAVIOUR OF DRIVEN PILES IN SAND</td>
<td>157</td>
</tr>
<tr>
<td>8.4 EXISTING DESIGN PROCEDURES</td>
<td>158</td>
</tr>
<tr>
<td>8.5 RECENT DEVELOPMENTS IN DESIGN METHODS FOR DRIVEN PILES IN SAND</td>
<td>159</td>
</tr>
<tr>
<td>8.6 DESIGN PARAMETERS FOR NORTH SEA SANDS</td>
<td>159</td>
</tr>
<tr>
<td>8.7 USING THE METHODS IN PRACTICE</td>
<td>160</td>
</tr>
<tr>
<td>8.8 REFERENCES</td>
<td>160</td>
</tr>
<tr>
<td>A. PILE TESTS IN SAND – EXISTING DATABASES</td>
<td></td>
</tr>
<tr>
<td>A.1 INTRODUCTION</td>
<td>163</td>
</tr>
</tbody>
</table>
A.2 DENNIS AND OLSEN'S DATABASE 163
A.3 BRIAUD AND TUCKER'S DATABASE 165
A.4 LINGS' DATABASE 165
A.5 OTHER DATABASES 167
A.6 REFERENCES 169

B. METHODS USED TO OBTAIN THE PROPERTIES OF NORTH SEA SANDS
B.1 IN SITU TESTS 173
B.2 LABORATORY TESTS 176
B.3 REFERENCES 179
1. INTRODUCTION

1.1 OUTLINE OF THE REPORT

This Report critically reviews current practice in the design of piles in sand in the UK sector of the North Sea. Recent research into pile behaviour in sand has led to a number of proposals for reform of existing design methods, as codified, for example, in the current API (American Petroleum Institute) recommendations. A thorough review of existing methods and the assumptions on which they are based is therefore both timely and potentially of great value to the offshore industry.

The Report concentrates on the static axial load capacity of single piles. Other aspects of offshore pile design — such as estimating cyclic load capacity, lateral load capacity, and driveability — are discussed only briefly. The reason for this concentration on static axial load capacity is that existing design methods are largely based on its estimation and it is in this area that the greatest advances have been made through research over the past decade or so.

The Report is also biased towards the design of piles for steel jackets, these being the prevalent type of offshore structure in the North Sea. No consideration is given to the design of bucket or silo foundations, as used in the Snorre field (Christophersen et al., 1992) and the Europipe Riser Platform (Tjelta and Haaland, 1992).

A key requirement of this review is that it should be relevant to the conditions likely to be encountered in the UK Sector of the North Sea. Chapter 2 therefore concentrates on establishing the nature and extent (i.e. the geology and geography) of the North Sea sands, in order to place the remainder of the Report in context.

Chapter 3 examines the fundamental behaviour of driven piles in sand, in terms of effective stresses. The importance of having an understanding of fundamental behaviour cannot be overstated: without it, the wrong conclusions can be drawn from existing engineering knowledge and design methods can evolve which are fundamentally unsound if not unsafe. For this reason, the Report devotes a whole chapter to explaining how piles in sand work, as revealed by the latest state-of-the-art research into the subject.

Existing procedures for designing piles in sand are described and discussed in Chapter 4. The evolution of the API recommendations is recorded and the controversial issues that have helped shape its contents are described. The impact of the move towards load and resistance factor design is discussed.
Chapter 5 picks up where Chapter 4 leaves off, by describing new methods of designing piles in sand. These methods are the by-products of a series of major advances in the understanding of the behaviour of displacement piles, not only in sands, but also in clays, that have taken place over the last 10 years. Based on the latest field and laboratory research by a number of leading research institutions and industry-led research projects, these methods offer the first true advance in offshore pile design in nearly 25 years.

Chapter 6 discusses the design parameters that are obtained from field and laboratory tests on North Sea sands, and indicates the latest trends in interpreting this data. As testing techniques improve, there is the danger that existing design methods will not be updated to take that improvement into account.

Chapter 7 discusses the potential impact of adopting the new design methods by performing a number of illustrative calculations of pile capacity at two "typical" North Sea sites: one in the central North Sea and one in the south.

Chapter 8 concludes the Report by drawing together the main findings of the previous seven chapters.

Appendix A describes existing databases of pile tests in sand and Appendix B concentrates on summarizing field and laboratory tests for determining the properties of sand relevant to pile design. As design methods become more sophisticated, there is often a need to obtain more information about a sand than has hitherto been the case. Greater precision in the determination of these parameters is also desirable as design methods become less empirical, more fundamental in nature.

1.2 AREA OF INTEREST

The UK sector of the North Sea covers an area approximately 1000km long by 500km wide, from latitude 51°EN to 62°EN (see Figure 1). In August 1994, there were approximately 100 oil and gas fields in production in the UK sector of the North Sea, with a further 32 fields under development (Offshore Engineer, 1994). Of those in production, 60% were producing oil and 50% gas. The value of oil production alone is estimated to be $16bn at 1994 prices.
Figure 1
The UK sector of the North Sea with a selection of oil and gas fields shown (Thomas, 1989)
1.3 FOUNDATION TYPES
Since the discovery of commercial gas reserves at West Sole in 1966 and commercial oil reserves at Ekofisk in 1968 (Thomas, 1989), there has been a

![Diagram showing the number of installations from 1966 to 1993 for Steel jackets and Gravity-based structures.

Figure 2
Number of offshore installations in the UK sector of the North Sea since 1966 (Fugro-McClelland, 1993)

steady rise in the number of offshore structures founded in the UK sector of the North Sea (see Figure 2). Of these 200 or so structures, over 90 per cent are pile-supported steel jackets.

The smaller jacket structures are sometimes founded on one pile per leg, whereas large jackets are supported by pile groups with typically 4-8 piles in each group. The piles themselves are usually open-ended steel pipes up to 2m in diameter and 80m or more in length. Their size is such that they dwarf their counterparts onshore (see Figure 3).*

*The offshore piles included on this diagram are located in the following North Sea oil fields: A, Auk; B, Brent; C, Claymore; F, Forties; H, North-west Hutton; m, Murchison; M, Magnus; N, Ninian; P, Piper; S, South Brae; and T, Thistle
Modern practice is to drive the piles into the seabed through vertical sleeves using underwater hammers. The piles are then connected to the super-structure by welding above water or grouting or swaging below water (Blair-Fish et al., 1994).

1.4 Soil types

The stratigraphy of the North Sea is complicated, owing to its depositional history (as discussed in Chapter 2). Sands are present in almost all parts of the North Sea, as a thin surface layer, interbedded with clays, or as the predominant soil type.

Thomas (1989) has divided the North Sea into four main provinces, based upon the following generalized soil profiles:

- Stiff to very stiff overconsolidated clays and silty clays
- Very soft to soft normally consolidated clays and silty clays overlying stiff to very stiff overconsolidated silty clays
- Stiff to very stiff overconsolidated silty clays and clays interbedded with dense fine sand
- Fine to coarse sand with scattered seams and beds of soft to stiff clays and silty clays

The first three profiles typically possess a thin surface unit of fine sand.
Figure 4 illustrates the approximate boundaries between the various soil provinces and Figure 5 shows three generalized soil profiles based on the cross-sections marked on Figure 4.

As Figure 4 shows, soil conditions in the UK sector of the northern North Sea differ greatly from those that pertain in the south. In the north, stiff to very stiff overconsolidated clays are in abundance, although in many areas they are interbedded with dense fine sand. In the central North Sea, interbedded clays and sands predominate, whereas in the south there is a large tract of mainly fine to coarse sand. The geological origin of these soils is discussed and explained in Chapter 2.

The variability of soil conditions in the northern and central parts of the North Sea is illustrated by cross-sections B-B' and C-C' (see Figure 5). As section C-C' shows in particular, the soil profile can vary greatly from one location to the next even within the same field.

1.5 SITE INVESTIGATION

Site investigations are usually carried out in stages, starting with a reconnaissance or regional appraisal of shallow geological and geotechnical conditions covering all potential platform locations. A more detailed final investigation is carried out when the location of the fixed structure has been determined (Thomas, 1989).

Boreholes are generally sunk using open-hole rotary drilling techniques. Sampling and in situ testing tools are deployed through the drill pipe to the drill bit using wireline technology. Soil samples are obtained using push and (sometimes) piston samplers. It is particularly difficult to obtain undisturbed samples of sand.

In situ soil testing supplements soil sampling and can be performed either directly from the seafloor or in boreholes. In situ tests give valuable information about variations in soil conditions. The correlation of cone penetration resistance and sleeve friction with the strength and deformation properties of clays and sands is used in the evaluation of design parameters (see Chapter 6).
Figure 4
Generalized soil provinces in the North Sea (Thomas, 1989)
Figure 5
Generalized soil profiles in various parts of the North Sea (Thomas, 1989)
The number of borings that are drilled at the site of a jacket structure depends on the lateral extent of the jacket and the intended batter (if any) of its piles (Hobbs, 1992). Typical site investigations for different foundation types are indicated in Table 1.

<table>
<thead>
<tr>
<th>Proposed foundation</th>
<th>Typical scope of geotechnical surveys (with typical duration, including weather down-time)</th>
<th>Northern North Sea</th>
<th>Southern North Sea</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jack-up spud cans</td>
<td>Single boring or 3 deep piezocone tests if soil conditions permit (1 or 2 days)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Piled foundations for jacket</td>
<td>4 borings plus 5 surface cone penetration tests or additional shallow borings (2 weeks)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Piled foundations for a template</td>
<td>1 or 2 borings depending on template size; for small/lightly loaded units, surface piezocone tests may suffice (½ week)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gravity based foundations</td>
<td>5 borings plus 10-20 surface piezocone tests and possibly seabed in situ vane tests (3 weeks)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

In general, boreholes include a mixture of push samples and in situ piezocone penetrometer tests. In sands, additional disturbed samples are taken within the (typically 3m) cone stroke where there are indications of inclusions or variations (Hobbs, 1992). For small structures in predominantly sand sites which are expected to be uniform, an alternative is to have one fully-sampled boring and one piezocone boring. In the case of granular deposits (such as sands and gravels), greater reliance is placed on the results of in situ tests than on laboratory strength tests. Laboratory tests on sand concentrate on determining the sand’s particle size distribution and composition (Fugro-McClelland, 1993).

Boreholes typically extend to at least five diameters or 10m (whichever is the greater) beyond the calculated required pile penetration or likely depth of influence of pile groups (Hobbs, 1992). This calculation is based on a conservative assessment of axial pile capacity and soil conditions.

A full discussion of site investigation techniques relevant to sands is given in
Appendix B.

1.6 METHODS OF DESIGNING PILES IN SAND

Focht and O'Neill (1985) conducted a survey of worldwide practice on various aspects of design and construction of axially loaded piles, covering both onshore and offshore piles. The results of that survey, as far as they concern the axial compressive capacity of driven piles in cohesionless soils, are summarized in 6.

Worldwide, the most popular method of assessing pile capacity in cohesionless soils involves a simple friction analysis based on the soil's angle of internal friction.* In European practice, however, correlations between axial capacity and the results of in situ tests are nearly twice as popular as any other method (Focht and O'Neill, 1985).

Methods currently used to design offshore piles are discussed at length in Chapter 4. In the North Sea, offshore pile design is based largely on the recommendations given in the American Petroleum Institute's Recommended practice for planning, designing and constructing fixed offshore platforms (API RP2A). The current perception of over 30 international experts on pile capacity is that the design method given in API RP2A is conservative in medium dense to very dense sands (Lacasse and Goulois, 1989).

Predictions of foundation behaviour and soil-structure interaction cannot be made with certainty (Lacasse and Nadim, 1994), owing to:

- Spatial variation in soil properties
- Limited site exploration
- Limited calculation models
- Uncertainties in soil parameters
- Uncertainties in loads

The uncertainty that can arise from the choice of calculation model based on model tests is illustrated in Figure 7. Current axial pile capacity calculation methods have been derived predominantly from onshore load tests on small piles. Existing databases of pile tests in sand are reviewed in Appendix A. Penetration depth, pile length, pile diameter, and ultimate load for the largest piles in the reference database are much smaller for the test piles than for those currently used in the North Sea (Lacasse and Nadim, 1994) — see Figures 3 and 7. However, there have been no reports of a jacket failure on the UK continental shelf owing to the inadequacy of its piled foundations (Fugro-McClelland, 1993).

---

*The popularity of static load tests in Figure 6 reflects onshore rather than offshore practice, where it usually impractical to conduct such tests
Figure 6
Popularity of methods of assessing axial compressive capacity of driven piles in cohesionless soils (Focht & O'Neill, 1985)

1.7 FUTURE TRENDS

According to Toolan (1992), by far the greatest influence on geotechnical design and engineering in the 1990s will come from the introduction of new codes and regulations. The greatest impact will arise from the Health and Safety Executive's requirement for the submission of a safety case for offshore installations. The probable introduction of an offshore Eurocode, based on the American Petroleum Institute's Load and resistance factor design (LRFD) version of its recommended practice RP2A, will have a significant effect on design philosophy, procedures, and foundation cost (particularly for piled structures). These topics are discussed further in Chapter 4.

It is widely accepted in the offshore industry that a better understanding of fundamental soil/structure interaction mechanisms is needed in order to reduce unnecessary conservatism in foundation design (Silbert, 1994). Chapter 5 summarizes the most promising recent developments in the design of piles in sands, which are based on the modern understanding of pile behaviour presented in Chapter 2.
Attempts to extend the meagre pile load test database to larger piles and a wider range of soils — including for the first time sands — are described in Chapter 5.

1.8 REFERENCES

Recommended practice for planning, designing and constructing fixed offshore platforms — working stress design.

Offshore piling practice in the North Sea and Arabian Sea.

Behaviour of displacement piles in overconsolidated clays.
PhD thesis, University of London (Imperial College), 725pp.

Innovative foundation systems selected for the Snorre Field development.

Piles and other deep foundations.

UK offshore site investigation and foundation practices.
Report no 92/2549-1 for Health & Safety Executive.

A review of the design and certification of offshore piles, with reference to recent axial pile load tests.

Uncertainty in API parameters for predictions of axial capacity of driven piles in sand.

Reliability issues and future challenges in geotechnical engineering for offshore structures.
Proc. 7th Int. Conf. on Behaviour of Offshore Structures (BOSS '94), Cambridge, Mass., Invited Papers, 9-38.

OFFSHORE ENGINEER (1994).
Offshore oil and gas activity ("activity tables").

An initiative to facilitate implementation of new technology.
Proc. 7th Int. Conf. on Behaviour of Offshore Structures (BOSS '94), Cambridge, Mass., Invited Papers, 1-8.

Geotechnical investigation of UK test sites for the foundations of offshore structures.

Novel foundation concept for a jacket finding its place.

2. GEOLOGY AND GEOGRAPHY OF NORTH SEA SANDS

2.1 INTRODUCTION

Chapter 2 looks at the geology and geography of North Sea sands. Section 2.2 explains the origins of the sands that underlie much of the North Sea, particularly the southern waters; and Section 2.3 analyses the distribution of sands in the UK sector of the North Sea.

2.2 GEOLOGY OF NORTH SEA SANDS

2.2.1 Depositional environments

The sands of the North Sea were laid down under a number of depositional environments, including marine, lacustrine, periglacial, proglacial, and subglacial. Figure 8 summarizes the depositional environments that applied in the North Sea from Pliocene times (approximately 2.5 million years B.P.*) through to Holocene times (approximately 10 000 years B.P.). The following sections discuss each of the geological ages in turn.

2.2.2 Pleistocene age

During the early Pleistocene, the North Sea basin was gently subsiding towards its centre, drawing sediment from the surrounding land masses. In middle to late Pleistocene times, the area was affected by three glaciations in which glaciers advanced across the North Sea continental-shelf and eroded the underlying sediments.

The deposits seen from the early Pleistocene reflect the advancement of a large delta complex towards the north and west across the Southern North Sea. Fed by the northern German and eastern British river systems, the delta expanded across the shelf, depositing up to 550m of deltaic sediments over the underlying marine clays (Cameron et al., 1987).

Figure 9 shows a seismic reflection profile (and its interpretation) across the UK sector of the North Sea. It is part of a series of profiles providing a continuous section through the delta complex. The profile shown is produced by combining eight adjacent seismic traces.

*Before present
The deltaic sediments coarsen upwards from prodelta silty clays through delta-front deposits into sandy delta-top deposits, seen as intertidal and shallow marine sands and clays. Because the delta migrated across the North Sea basin during the Pleistocene, all of these facies (prodelta, delta-front, and delta-top) are diachronous,* becoming progressively younger in the direction of advance of the delta.

*A term applied to lithological units (e.g. a bed of sandstone) which appear to be continuous beds but which in fact represent the development of the same facies at different places at different times

---

**Figure 8**

Depositional environment and inferred chronostratigraphy of the North Sea (Cameron et al., 1992)
Figure 9
Interpretation of seismic reflection profile across the UK sector of the North Sea (Cameron et al., 1993)
Figure 10 shows the principal direction of delta growth as it varied with time and the location of the seismic profile shown in Figure 9 (section A-A'). The delta-front clinoforms* dip in the direction of advance of the delta. The delta expanded towards the northwest during the Pleistocene, so the deltaic deposits are thickest at the eastern end of the profile, where the delta first entered the sector. The continued subsidence of the basin also acted to produce a greater sediment thickness at the eastern end of the profile, nearest the centre of the basin.

*The inclined sediments that build outwards on the seaward face of a delta
As the delta matured, the deltaic sands became interbedded with, and eventually passed upwards into, alluvial fine sands with occasional thin clay horizons. These clay layers can be continuous over many tens of kilometres. Intermittent beds of marine shelly sand mark large scale marine incursions, probably due to storms.

At the same time in the Central North Sea, marine clays with lenses of sand, which are indicative of a temperate shallow marine-shelf environment, were being deposited. The distal influence of the advancing delta front produced more abundant sand lenses to the south.

The equivalent sediments in the Northern North Sea were laid down in a warm-temperate, open marine-shelf environment and consist of well-sorted, fine shelly sands, grading upwards into slightly sandy clay. The clay contains beds of sand which become thicker and more abundant towards the north-east.

2.2.3 Elsterian glaciation

The base of the first (Elsterian) glaciation is marked by an unconformity which dips gently towards the centre of the basin, eroded as the sea retreated, due to falling sea levels, and the British and Scandinavian ice sheets advanced across the North Sea. The ice sheets cut tunnel valleys hundreds of metres deep into the deltaic sediments (Fannin, 1992). These valleys can be clearly seen on Figure 9. Towards the end of the glacial stage, glacial melt waters cut more valleys up to 80m deep in parts into the underlying early Pleistocene sediments (Holmes, 1977). These valleys were partly filled by glaciolacustrine clays and glaciomarine clays, silts, and tills.

The infilling continued into the succeeding Holsteinian interglacial with the deposition of marine sands and clays across the North Sea, as sea levels rebounded as the ice melted.

2.2.4 Saalian glaciation

The glaciers did not extend as far during the Saalian glaciation as during the previous Elsterian period. Because of this, much of the southern North Sea — although exposed by falling sea levels — was not covered by ice but was subject to periglacial processes, producing proglacial clays, wind-blown sands, and fluvioglacial sands (Cameron et al., 1987). Across the rest of the basin, the Saalian event is marked by glaciomarine clay with drop-stones interbedded with silt and shelly sand. Erosion during the retreat of the glaciers etched valleys into these sediments. The valleys in the northern North Sea, which became filled in part with glaciomarine, slightly gravelly, sandy clays, also contain sediments from the next interglacial (Eemian) stage. Passing from west to east in the southern North Sea, these consist of beach, intertidal, and shallow marine sands and clays and, in the remainder of the basin, consist of interbedded marine muds and shelly sands.
2.2.5 Weichselian glaciation

The most recent glaciation (Weichselian) saw environmental conditions similar to those of the Saalian glaciation, with periglacial conditions in the south and glaciomarine sands and tills deposited in the north (see Figure 11). Near the end of this stage, melting ice deposited till and fluvioglacial sands and gravels in the south, while glaciomarine conditions persisted in the north. As with the preceding glaciations, valleys were eroded in the seafloor deposits. In the northern North Sea (close to the glacier in the Norwegian Trench), these valleys have become filled with glaciomarine sands overlain by sandy shelly clays and morainic deposits.

In the southern North Sea the valleys were filled with fluvioglacial sands and glaciolacustrine clays. The present interglacial is represented in this area by brackish marine sands and clays passing upwards into quartzose sands as the sea levels rose to present levels. To the north, silty glauconitic shallow marine sands pass northwards into soft silty marine clay with partings of sand (Pantin, 1991). These recent deposits are relatively thin and the redistribution of sediments continues as the sediment and tidal patterns are still reacting to the end of the last Ice Age.
Figure 11
North Sea during the maximum offshore extension of the late Weichselian ice sheets (Cameron et al., 1987)
2.3 DISTRIBUTION OF SANDS IN UK SECTOR OF NORTH SEA

2.3.1 Borehole records

The distribution of sands in the UK sector of the North Sea has been established by examining borehole records held by the British Geological Survey, BP International, and Shell UK.

A total of 212 records were examined in order to establish:

- The proportion of sand in the top 10m below seabed
- The proportion of sand in the top 60m below seabed
- The maximum thickness of sand in the top 100m below seabed

In compiling these figures, the decision as to which layers should be regarded as sand was based on the soil description given on the borehole log. Silty and clayey sands were both included in the statistics; gravels and silts (even sandy silts) were not.

In some cases, boreholes did not extend to 60m below seabed. In these instances (and provided the borehole was longer than 30m), the proportion of sand in the total length of the borehole was substituted for the proportion of sand in the top 60m.

Similarly, if a borehole did not extend beyond 100m, the maximum thickness of sand anywhere in the borehole was substituted for the maximum thickness in the top 100m (provided the borehole was longer than 50m).

The results of this survey are presented in Figures 12 to 14 and discussed in the sections below.

2.3.2 Superficial sands

Figure 12 shows the proportion of sand in the top 10m below seabed in the UK sector of the North Sea. In roughly 30% of the blocks for which information is available, there is less than 10% (i.e. 1m) of sand in the top 10m; and in a further 25% there is between 10 and 35% sand.

These trends are confirmed by Figure 15, which shows the thickness of superficial sands at existing offshore platforms. Superficial sands greater than 10m in thickness occur at just over 10% of sites.
Figure 12
Proportion of sand in the top 10m below seabed
Figure 13
Proportion of sand in the top 60m below seabed
Figure 14
Maximum thickness of sand in the top 100m below seabed
2.3.3 Deeper sands

Figure 13 shows the proportion of sand in the top 60m below seabed in the UK sector of the North Sea. As expected from the evidence presented in Figures 4 and 5, in the northern and central parts of the North Sea, sand generally makes up less than 35% of the soil strata in the blocks for which information is available. The proportion of sand in the top 60m is generally greater than 35% in the southern North Sea. This only affects a small number of blocks in the UK sector (e.g. 48, 49, and 53).

The abundance of sand in the UK sector of the North Sea is quantified in Figure 16. The proportion of platforms that are founded on the different soil profiles are as follows:

- 33% on interbedded sands and clays
- 21% on all sand
- 15% on clays overlying sand
- 31% on clays with little or no superficial sand
These figures imply that between 54% and 69% of all piled foundations rely on sand providing some part (if not the majority) of their capacity.

2.3.4 Thickness of sands
In the interbedded strata in particular, the thickness of sand layers is important in determining both the shaft and end bearing capacities of piles. Figure 14 shows the maximum thickness of any sand layer in the top 100m below seabed in the UK sector of the North Sea. In the southern North Sea, the sand layers are invariably greater than 20m thick; in the central North Sea they are typically between 5 and 20m thick; and in the northern North Sea they vary greatly in thickness between less than 5m to more than 20m.

2.4 OCCURRENCE
The sands occur as:

- A thin cover (less than 3m) to clays which may be soft to hard (as at Statford, Dunlin A, Gullfaks C).
- A relatively thick cover to stiff clays (up to 10m thick, as at Beryl A; Frigg CDP-1, TP-1, and TCP-2; and Ravenspurn North).
- Inter-beds between stiff silty clays (as at Brent B, C, and D; Cormorant A; Magnus; Ninian Central; Dunlin A; and Hutton TLP). The inter-beds may vary in thickness by several metres over relatively short horizontal distances.
- An extensive deposit, greater than 10m thick (as at Ekofisk — 25 m thick, the Frigg manifold; and Gullfaks B). Clay layers still occur within the sands.
An upper layer of Holocene sand, 5 to 30m thick, which overlies a Pleistocene clay band (typically 5m thick), under which are found deep layers of shelly marine sand over deltaic sand formations. These conditions are typical of some southern North Sea locations.

Typical borehole logs, showing four different distributions of sand, are given at the end of this chapter.

2.5 REFERENCES


The seabed sediments around the United Kingdom: their bathymetric and physical environment, grain size, mineral composition and associated bedforms.

British Geol. Soc. Research Report, SB/90/1, 47pp + maps.
Figure 17
Typical borehole log for Nelson field (courtesy Shell UK)
Figure 18
Typical borehole log for Hutton TLP (after Jardine, 1995)
Dense fine SAND

Firm CLAY, becoming sandy at base

Dense fine SAND

with gravels and shells at +10.0m
thin clay seam at 14.2m
silty to very silty from 14.2m to 24m
shells and thin layers of cemented sand from 27.5m to 30.3m
thin silt seam at 31.7m
silty from 33.7m to 38m

41.8m: end of boring

Figure 19
Typical borehole log for southern North Sea site
Figure 20
Typical borehole log for central North Sea site
3. FUNDAMENTAL BEHAVIOUR OF DRIVEN PILES IN SAND

3.1 THE NEED TO UNDERSTAND FUNDAMENTAL BEHAVIOUR

The common methods of calculating the vertical load capacity of piles in sand are entirely empirical. They rely on field measured capacities being related to parameters that represent the initial state of the sand, for example: Standard Penetration Test blow count; cone tip and sleeve friction resistances; or derived parameters such as relative density. These methods can be used when the database from which the method was developed is relevant to the site and pile in question, in terms of sand type, pile geometry, method of installation, etc. Difficulties arise, however, because:

- Only measured total capacities are usually available, so that separation into shaft and tip components requires assumptions to be made. (Tension tests are easier to interpret than compression tests, because the base resistance tends to zero at failure.)
- Shaft resistance is calculated by integrating the local shear stresses \( \tau_{rz} \) acting along the pile shaft at failure, but distributions of local shear stress are not known. The usual assumption that \( \tau_{rz} \) increases linearly with depth (because of increasing vertical effective stress) to some limiting value is not supported by recent measurements (see Section 3.2). Extrapolation to piles of different lengths to those in the database is therefore uncertain.
- Where piles have been instrumented to allow separation of shaft and tip capacities, the data may be affected by shifts in strain-gauge zeros as a result of pile driving. Re-zeroing of strain-gauges after driving ignores the residual forces acting at the end of installation.
- Interpreted pile capacities depend on the definitions of failure and the way in which the tests are conducted.

The reliability of current empirical design methods for assessing axial capacity of displacement piles in sand is relatively poor (Focht and O'Neill, 1985; Briaud and Tucker, 1988).

*Using an equation of the form:

\[ Q_s = \int_0^L \tau_{rz} \pi D \, dz \]

where \( Q_s \) is the shaft resistance; \( \tau_{rz} \) is the local shear stress; \( L \) is the length of the pile; and \( D \) is its diameter.
<table>
<thead>
<tr>
<th>Reference</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Field research</strong></td>
<td><strong>Test in dense Dunkirk sand with 324mm diameter strain-gauged piles, driven using different pile shoes</strong></td>
</tr>
<tr>
<td>Le Tirant <em>et al.</em> (1991)</td>
<td><strong>Tests performed at Labenne, south west France, with the Imperial College instrumented pile, a 102mm diameter closed-ended steel pipe installed by fast jacking in medium dense to loose dune sand</strong></td>
</tr>
<tr>
<td>Lehane <em>et al.</em> (1993)</td>
<td><strong>Tests at Dunkirk, north west France, with the Imperial College Instrumented Pile, in dense marine sand. Also tests to examine group and aging effects, the latter involving experiments described by Le Tirant <em>et al.</em> (1991)</strong></td>
</tr>
<tr>
<td><strong>Laboratory research</strong></td>
<td><strong>1.85m long, 114mm diameter steel pile jacked into dry Leighton Buzzard sand. Observations of the equalized and failure radial effective stresses and the measured angles of interface friction were consistent with the Labenne findings</strong></td>
</tr>
<tr>
<td>Wersching (1987)</td>
<td><strong>55-70mm diameter model piles jacked or driven into Hostun sand in a stress-controlled calibration chamber. Vertical and radial effective stresses depended on pile tip location. Particle crushing beneath the pile tip depended on the original vertical effective stress</strong></td>
</tr>
<tr>
<td>Foray <em>et al.</em> (1993)</td>
<td><strong>Parametric laboratory studies into sand-steel interface shear resistance</strong></td>
</tr>
<tr>
<td>Jardine <em>et al.</em> (1992)</td>
<td><strong>Model studies of pile plug resistance in sands</strong></td>
</tr>
<tr>
<td>Chow (1996)</td>
<td><strong>Strain path method developed for analysis of undrained pile penetration, closed- and open-ended. The method assumes continuous (rather than discontinuous) penetration. Currently being developed to deal with drained and partially drained penetration</strong></td>
</tr>
<tr>
<td>Hight <em>et al.</em> (1996)</td>
<td><strong>Elasto-plastic solutions for expansion of spherical and cylindrical cavities in idealized sands, considering the effects of dilation, friction angle, initial stress regime, and shear stiffness; applied to closed-ended pile base capacity</strong></td>
</tr>
<tr>
<td>Baligh (1985)</td>
<td><strong>Finite element and database studies of plug resistance and end-bearing in sands</strong></td>
</tr>
<tr>
<td>Whittle (1992)</td>
<td></td>
</tr>
</tbody>
</table>

For offshore piles, the existing databases do not cover the pile diameters and lengths being used and may not include the relevant sand types (see Appendix A). To extrapolate from the databases, therefore, it is necessary to have a
design method that embraces the essential features of the installation process and pile loading mechanism, and can deal with the effects of changes in scale.

The mechanisms by which shaft and tip resistance are mobilized around piles in sand have been clarified by the results of recent field, laboratory, and theoretical research, examples of which are given in Table 2.

To understand the stresses acting around a pile at failure, and to determine the soil strength parameters that control those stresses, it is necessary to consider:

- The process of installation
- The equalization period (i.e. pile "set up", which includes the effect of dissipation of excess pore pressures and ageing)
- The loading mechanism

3.2 INSTALLATION

3.2.1 Mode of penetration

Piles driven into sand may be closed- or open-ended. In open-ended piles (pipe piles), sand may flow unimpeded into the pile during driving or its flow may be partially or fully impeded (see Figure 21). When sand entry into the pile is prevented, the pile is said to be plugged. In this case it behaves during driving as if it is closed-ended.

There are important differences between the installation effects of fully plugged, partially plugged, and unplugged driven pipe piles, in terms of the extent of the zone affected and the magnitude of stress and strain changes in this zone. The radial extent of stress changes and their magnitude will be less for unplugged than for plugged penetration. The sand that enters the pile to form the sand column changes density. Loose sands compact so that measurements of "plug depression" (see Figure 21c) are insufficient to determine whether or not full or partial plugging has occurred: observations of the incremental filling ratio (increase in soil column length, Δh, to increase in pile penetration, Δd) are required, together with an estimate of density changes in the column. Observations in small model piles (Paikowsky, 1990) suggest that the formation and collapse of passive arches cause alternate layers of loose and dense sand to develop inside the sand column, with a dense zone existing at its base.

Whether or not a plug forms during driving appears to depend on:

- Pile diameter
- Internal pile geometry (e.g. reductions in internal bore)
- Sand column length
- Sand density
- Impact characteristics (e.g. energy input)
Figure 21
Modes of penetration for open-ended piles (after Paikowsky et al., 1989)

In model tests involving small diameter pipe piles driven into loose sand, Kishida (1967) found that plugging occurs with pile diameters less than 150mm and that a minimum sand column length is required for a plug to form (see Figure 22).

Experience offshore indicates that full plugging does not occur with large diameter piles driven into dense sand. The failure of a pile to plug during driving does not necessarily mean that it will not plug during static loading, since inertia effects which are present during driving are absent during static loading. The effect of pile diameter and pile type on plug length is illustrated in Figure 23.
Figure 22
Sand plug depression in model open-ended piles (after Kishida, 1967)

Figure 23
Relationship between normalized plug height and pile diameter (after Niyama et al., 1994)

During installation, a pile can alternate between being unplugged, partially plugged, or fully plugged, depending on the bearing capacity of the underlying
material. The mode of penetration affects pile driveability. Hight et al. (1996) describe analytical and physical model studies that provide new insights into the process of pile plugging and the development of base resistance.

Whether pile installation in sand is fully drained, partially drained, or undrained depends on the rate of penetration, the permeability of the sand, and whether or not the pile plugs (undrained penetration is unlikely in sands). The initial state of the sand is modified; large shear strains are imposed; volume strains and changes in density occur (unless penetration is undrained); and high tip stresses are applied transiently. Sand fabric is changed and sand grains may crush.

The extent of the disturbance depends on:
- Pile diameter
- Drainage conditions
- Whether or not there is grain crushing
- Whether the pile is closed- or open-ended
- (For open-ended piles) Whether or not a plug forms

The radial and shear stresses acting on a pile immediately after its installation depend on all these factors, as well as on pile length.

The following sections illustrate some of the effects of installation of closed-ended piles, by summarizing data from the field measurements at Labenne (see Table 2). The trends identified have subsequently been confirmed by further studies at Dunkirk.

3.2.2 Variation of end-bearing during installation

Figure 24 shows the variation with depth of the end-bearing resistance ($q_b$) developed by the pile during jacking at Labenne. The $q_b$ profile for the closed-ended pile was similar to the profile of standard cone tip resistance ($q_c$), but stratum boundaries were less well-defined because of the pile's larger diameter.

Pore water pressure measurements on the shaft indicated that installation was essentially drained during jacking; pore pressures varied by less than ±2kPa from the hydrostatic profile.
3.2.3 Variation in shaft resistance during installation

Figure 25 shows the variation of the local and average shear stresses during pile installation. The local shear stress ($\tau_{rz}$) was measured with surface stress transducers located at approximately 1 m intervals along the pile shaft, and the average shear stress ($f_s$) was calculated from the change in axial load between these positions.

The following points are worth noting from Figure 25:

- At a given depth, $\tau_{rz}$ and $f_s$ reduce as the height above the pile tip increases (this has been referred to as friction fatigue by Hereema, 1980).
- $\tau_{rz}$ and $f_s$ depend strongly on relative density, as expressed by end-bearing resistance $q_b$.

Nauroy et al. (1988) have shown that, in carbonate sands, $\tau_{rz}$ is considerably higher near the pile tip than elsewhere.
Figure 25
Local and average shaft resistances measured at Labenne

Figure 26 shows the average shear stress measured over the full shaft length ($\tau_{av}$) as a function of pile penetration. $\tau_{av}$ increased linearly to a depth of 40 pile radii ($R$) and then remained essentially constant between 20 and 28 kPa. These measurements are consistent with the observations of Vesic (1967) that $\tau_{av}$ tends to an asymptote below a "critical depth" when piles are driven in sand. Rather than showing that the local shaft resistance had stopped increasing with depth, as had been widely assumed, this trend is a direct consequence of the continual reduction in local shear stress ($\tau_{rz}$) at any depth as pile penetration continues beyond that depth.

The value of interface friction angle ($\delta = \tan^{-1} \tau_{rz}/\sigma'_r$) during pile penetration was typically 28° and showed no systematic variation with pile depth, height above the pile tip, or soil density.

3.2.4 Variation in radial effective stresses during installation
The variations in local shear stress reflect the variations in radial effective stress on the pile shaft ($\sigma'_r$). Values of $\sigma'_r$ during each stroke of the jack exceeded those prior to each stroke (when the pile was stationary) by approximately 5 kPa in loose sand and up to 25 kPa in medium dense sand at shallow depths. This is the result of dilation at the shaft interface.
Figure 26
Variations in average shaft resistance with pile penetration at Labenne

Radial effective stresses stabilized rapidly between jacking stages and showed little variation over a period of days. Figure 27 shows profiles of radial effective stress when the pile was stationary ($\sigma'_{rs}$), as recorded at three instrument positions during installation.

The following aspects of this diagram are worth noting:

- $\sigma'_{rs}$ profiles are similar to the profiles of end-bearing and cone resistance shown in Figure 24; it follows that $\sigma'_{rs}$ depends strongly on local relative density.

- At any given depth, $\sigma'_{rs}$ reduces as the height above the pile tip increases, i.e. as pile penetration continues below that depth.

- $\sigma'_{rs}$ varies from 0.5 to 1.1 times the original vertical effective stress ($\sigma'_{vo}$), which compares to an initial range of between 0.5 and 0.75 times $\sigma'_{vo}$.
3.2.5 The h/R effect

Figure 28 shows the envelopes established from approximately 30 individual $\sigma'_{fs}$ measurements made in two pile installations. Plotting the ratio $\sigma'_{fs}/q_{p}$ against the h/R values at which the radial effective stresses were measured brings the data into a relatively narrow band and demonstrates the strong dependence of $\sigma'_{fs}$ on the relative position of the pile tip (h/R). As mentioned above, the normalized stresses are seen to reduce by approximately 50% as the pile tip advances from a level 8 radii below a given soil horizon to a depth 50 radii below it.

Tests with the Imperial College pile in dense sand at Dunkirk confirmed that $\sigma'_{fs}/q_{vo}$ rises to still higher ratios as relative density increases and that this ratio reduces systematically with h/R. Data from the larger driven piles tested at Dunkirk showed that $\sigma'_{fs}/q_{c}$ decays more steeply with h/R when the piles are installed open ended.
3.2.6 Explaining the critical depth effect

The importance of the dependence of radial effective stress and hence shear stress on the normalized height above the pile tip (h/R) is illustrated on Figure 29.

Figure 29a shows the general form of the relationship between $K (= \alpha'_{ts}/\alpha'_{vo})$ and h/R, based on the results of the Labenne experiments. Note how the variation in K differs from the normal assumption of constant K. The precise form of this relationship depends on details of the installation, including the number of pulses of loading and drainage.

Figure 29b shows the local shear stress ($\tau_{rr}$) that would be mobilized adjacent to two piles (A and B) that are installed to different depths in a uniform sand layer. Note that:

- The maximum value of normalized shear stress occurs at the pile toe and increases linearly from the ground surface with the depth of toe.
- The normalized shear stress reduces rapidly with distance above the pile tip, owing in part to the corresponding reduction in vertical effective stress, but also due to the h/R effect described above.
- The normalized shear stress at any one depth is smaller next to the longer pile (B) than next to the shorter pile (A).

Also shown on Figure 29b is the normalized shear stress that would be obtained if K was constant and no cut-off value was applied to the local shear stress ($\tau_{rr}$).
Figure 29
Variation in (a) K, (b) local, and (c) average shear stress with depth
Figure 29c shows the distribution of average shear stress ($\tau_{av}$) for cases in which (i) $K$ is constant; and (ii) $K$ reduces with increasing $h/R$ (as defined by the $\psi$-curve in Figure 29a). The following points are worth noting from this figure:

- The line for $K = \text{constant}$ suggests that the average normalized shear stress mobilized along the pile shaft increases indefinitely with increasing pile length ($L/R$), which is contrary to current expectations (for this reason, it is common to apply a cut-off value to the local $\tau_{rb}$).
- The line derived from the $\psi$-curve implies that the average normalized shear stress tends towards a limiting value with increasing $L/R$.
- The observation that the average shear stress reaches a limiting value at some depth has led Vesic (1967) and others to suggest that there is a "critical depth", below which the local shear stress no longer increases with depth. Figure 29 explains this phenomenon.
- The line derived from the $\psi$-curve is similar in form to the empirical correlations for shaft capacity obtained by Coyle and Castello (1981), which are discussed in Chapter 4.

In conclusion, it appears that the $\psi$-curve shown on Figure 29, which has been established from instrumented pile tests in sands (at Labenne) and in clays (see Bond et al., 1992), reproduces the observed form of the relationship between average shear stress and $L/R$ without the need to apply a cut-off value to the local shear stress.

### 3.3 EQUALIZATION

The change in stresses between pile installation and loading depend largely on the drainage conditions during installation, i.e. on whether excess pore pressures remain after driving. Two extremes can be considered: undrained and drained installation. Stress changes after undrained installation may show some similarities to those observed in clays, with piles in dense sand being similar to piles in high OCR clays and piles in loose sand being similar to piles in low OCR clays.*

#### 3.3.1 Equalization after drained installation

As an example of equalization after drained installation, Figure 30 shows the variations of radial total stress ($\sigma_r$), pore pressure ($u$), and local shear stress ($\tau_{rz}$) with time following the installation of a typical pile at Labenne. Both $\sigma_r$ and $\tau_{rz}$ reduce immediately the pile comes to rest and equilibrium conditions (constant $\sigma_r$, $u$, and $\tau_{rz}$) are achieved within one minute.

The equilibrium radial effective stress ($\sigma'_{re}$) is controlled by the combination

---

*See Bond and Jardine (1991) and Lehane and Jardine (1992) for data relevant to high- and low-OCR clays.*
of the initial in situ conditions and the process of installation, and is a function of initial relative density, the original vertical effective stress, and height above the pile tip.

3.3.2 Residual stresses

After installation and equalization, significant forces act in the pile shaft. Rebound after jacking or driving is resisted by downward shear stresses over the shaft length, which lock-in part of the base capacity. During compression pile loading, the shaft shear stresses are reversed; this, and pre-stressing of the toe, influence the pile’s load-displacement behaviour. Dennis and Olsen (1983)

![Graphs showing variation of stresses with time at Labenne](image)

**Figure 30**

Variation of stresses with time at Labenne

48
inferred that the residual base loads developed in driven piles amounted to approximately 33% of their tensile shaft capacities. At LaBenne, downward residual shear stresses were uniform over the pile shaft (see Figure 30) and amounted to 25% of the tensile shaft capacity. These stresses balanced an upward locked-in load of 14kN measured at the pile toe.

3.3.3 Time effects and ageing

One key component of the recent Imperial College field research at Dunkirk was the re-testing of open-ended, steel piles driven into dense marine sand. The measured shaft capacity was a remarkable 85% higher in tests performed five years after installation compared with tests performed after six months.

Chow (1996) undertook a major review of published and archived pile test data which revealed similar increases in shaft capacity with time for steel, concrete, and timber piles in saturated, unsaturated, and carbonate-free sands. There was no comparable increase in base resistance. Possible reasons for this phenomenon include:

- Chemical processes (particularly corrosion)
- Changes in sand properties due to ageing
- Long-term increases in \( \sigma'_{rc} \)

The third hypothesis is currently the most plausible explanation for the observed set-up: creep may reduce arching effects around the pile shaft, so increasing the radial effective stresses, \( \sigma'_{rc} \). Some supporting evidence is provided by a limited set of long term \( \sigma'_{rc} \) field measurements (Chow et al. 1996). However, ageing may also result in stronger dilation during shearing and larger increases in \( \Delta\sigma'_r \) during pile loading.

3.4 Loading

3.4.1 Changes in radial effective stress

The LaBenne tests showed that the radial effective stresses on the pile shaft change during pile loading. When the peak shear resistance of the shaft (\( \tau_{rac} \)) was mobilized, the radial effective stress at failure (\( \sigma'_{rf} \)) was approximately 1.4 times the radial effective stress at the start of loading (\( \sigma'_{rc} \)) in the compression tests and approximately 1.2 times \( \sigma'_{rc} \) in the tension tests. Local shaft failure occurred when \( \sigma'_{rf} \) stopped increasing. The changes in shear stress (\( \tau_{ra} \)) and radial effective stress (\( \sigma'_{rf} \)) during compression and tension loading at LaBenne are shown in Figure 31. The smaller load capacities that were measured in tension tests were the result of the lower \( \sigma'_{rf} \) that developed.

During the early stages of loading, the values of \( \sigma'_{rf} \) reduce, probably as a result of anisotropy in the sand. The subsequent — and more substantial — increases in \( \sigma'_{rf} \) arise because the radial displacements that develop adjacent to the pile shaft (\( d_r \)) as a result of dilation are resisted by the shear stiffness of the
surrounding ground. The displacement \( d \), is likely to depend on the value of density and \( \sigma' \); the roughness of the interface; and the size and shape of the sand grains. It is, however, independent of the radius of the pile (\( R \)).

![Graph showing effective stress paths](image)

**Figure 31**

**Effective stress paths followed in first-time compression and tension tests at Labenne**

A simple elastic cylindrical cavity-expansion model predicts that the resulting radial effective stress change (\( \Delta \sigma'_r \)) at the pile shaft can be calculated as:

\[
\Delta \sigma'_r = \frac{2Gd_r}{R}
\]

where \( G \) is the elastic shear stiffness; \( d_r \) is the movement of the sand at the pile/soil interface; and \( r \) is the radius of the pile. Seed and Idriss (1970), amongst others, have shown that (at small strains) \( G \) is proportional to the square root of the mean effective stress and increases with relative density.

It can be seen from the equation above that \( \Delta \sigma'_r \) is inversely proportional to the pile radius. Currently available information indicates that dilation is unlikely to contribute more than 10% of the shaft capacity of large offshore piles, even in dense sands. It is clearly essential, however, to account for the effect of differences in pile radius when projecting field behaviour from the results of experiments on small model piles, for which dilation may contribute the major part of the shaft capacity.

Dilation at the interface as the grains closest to the pile begin to disengage from the micro-depressions of the pile surface: the radial effective stress stops increasing when dilation ceases. In circumstances where contraction (or
crushing) occurs at the interface, reductions in \( \sigma' \) can presumably occur.

### 3.4.2 Operational friction angle

The operational friction angle (\( \delta_f \)) in the Labenne tests was shown to be 28.5° ±1.5° and to compare closely with the constant volume interface friction angles (\( \delta_{CV} \)) measured in direct shear interface tests and discussed in Chapter 6. \( \delta_f \) was independent of initial relative density (\( D_s \)). Comparable values of \( \delta_f \) were seen in pile tension and compression tests.

### 3.4.3 Dependence of shaft capacity on pile tip detail and sand compressibility

The radial — and hence shear — stress that develops on the side of a pile depends on, amongst other things, whether the pile is open- or closed-ended (or plugged) and on the compressibility of the sand.

Figure 32 illustrates that:

- Shaft friction is higher in closed- than in open-ended piles: Beringen et al., 1979, report open-ended piles having 20% lower shaft capacity than their open-ended equivalents; as noted earlier, the recent Dunkirk experiments show that this is related to a steeper decay of \( \sigma'_{rc}/q_c \) with \( h/R \).
- Shaft friction reduces as the compressibility of the sand increases.
- In very compressible carbonate sands, the average skin friction may be less than 5kPa.

The limiting compressibility index used in Figure 32 is the slope of the void ratio vs log\( \varepsilon \) curve in an isotropic triaxial compression test.

### 3.4.4 End-bearing capacity

The static end-bearing capacity of a closed-ended pile is determined by a contained failure mechanism which develops in the surrounding sand. Analogies with cavity expansion theory show that the limiting pressures are controlled by the sand's friction angle, dilation characteristics, and stiffness behaviour. Bearing capacity reduces as sand compressibility increases.

---

\*\( \delta_f \) was calculated from the formula:

\[
\delta_f = \tan^{-1} \left( \frac{\tau_{rf}}{\sigma'_{rf}} \right)
\]

where \( \tau_{rf} \) is the local shear stress at failure and \( \sigma'_{rf} \) is the local radial effective stress at failure.
The static end bearing capacity of an open-ended pile will be reached when there is either:
- Failure or large displacement in the sand plug (Figure 33a).
- Bearing failure of the sand beneath and around the pile tip (Figure 33b).

The capacity and compressibility of a sand plug loaded under drained conditions depends on:
- Pile diameter
- Sand plug length
- Sand plug density and crushability
This dependence of capacity on pile diameter (D) and plug length (H) is illustrated in Figure 34. In medium dense to dense sands, there appears to be an exponential reduction in capacity with increase in pile diameter. Capacities are low for diameters greater than 800mm.

Analysis of load test data indicates that relatively small diameter (D < 600mm) open-ended piles in medium dense to dense sands tend to plug and behave as closed-end piles. Larger diameter piles (D > 700mm) tend to fail as a result of excessive displacement in the plug; their end bearing capacity* (q_{lb}) is less than that for closed-ended piles. This change in mechanism contributes to a strong trend for q_{lb} to reduce with increasing diameter of open-ended piles (see Figure 35).

*Defined at a settlement of 10% of pile diameter
Figure 34
Effect of sand plug length and pile diameter on plug capacity (Kishida & Isemoto, 1977)

In open-ended piles which are non-plugging the annular end bearing of the pile wall contributes to the tip capacity. Scale effects in bearing capacity need to be taken into account when assessing this contribution.

The end-bearing capacity also depends on drainage conditions in the plug, i.e. on the rate of loading and permeability of the sand. If loading is undrained, the shear stress that develops on the inside wall depends only on the initial stresses; if loading is drained, then the inner shear stress varies with the applied tip load (see Randolph et al., 1991).

3.5 LOAD-DISPLACEMENT BEHAVIOUR

The load-displacement behaviour of piles in sand under axial loading depends on:

- The profiles with depth of limiting shaft resistance (τ₀)
- The variations of the sand's shear stiffness (G) with shear strain (γ) and mean effective stress (p')
- The effective axial stiffness of the pile
- The conditions at the pile base (plugged, unplugged or closed)
- The sense of the loading (tension or compression)

The response to lateral loading is affected more by the properties of the near surface soils than is the axial response. Additional factors which influence lateral behaviour include:

- The limiting lateral pressures that can be sustained at given depths
- The flexural stiffness of the piles

![Graph showing ultimate end-bearing capacity versus open-ended pile diameter](image)

**Figure 35**

Ultimate end-bearing capacity versus open-ended pile diameter

The lateral response of an offshore pile group is also likely to be affected more by group action effects than is the axial behaviour (Ganendra, 1994).

### 3.5.1 Shaft response to axial load

The full shaft resistance of a pile in sand can usually be mobilised after relatively small pile head displacements (2 to 20mm) which depend on the pile length and radius, and on the soil conditions.

Numerical models of offshore piles performed using realistic soil models show that at any given depth, developing the full shaft resistance usually generates only relatively small shear strains in the soil close to the pile wall. Furthermore, the vertical shear stresses and shear strains diminish rapidly with radial distance from the pile. Once local shaft failure occurs, a thin shear band forms close to the pile which allows the shaft to move without any further straining of the surrounding soil. Shaft failure propagates progressively from the pile head downwards at a rate that depends on the pile's axial stiffness (see Jardine...
3.5.2 Pile base loading

Greater axial movements are required to develop the full end bearing capacity of a closed-ended or fully plugged pile. Here a relatively large volume of soil has to undergo a general shear (or bearing capacity) failure. Pile tip movements exceeding 10% of the diameter may be required in dense sands and the full shaft resistance is likely to have been mobilised long before the pile tip develops its full capacity.

Large diameter open-ended piles may show a still softer response because the soil column within the pile may experience compression movements (relative to the shaft) before reaching a limiting bearing pressure. Numerical simulations of this problem are strongly influenced by the assumptions made concerning: the sand's shear stiffness, its friction and dilation characteristics and behaviour at the interface between the pile and the soil plug.

3.5.3 Lateral loading

A long offshore pile subjected to large lateral loads is likely to develop a contained zone of laterally failing soil near to the mudline. The failure mechanism involves large strains developing within a zone extending around two diameters from the pile axis. The strain levels diminish rapidly with depth, at a rate which is controlled by the flexural stiffness of the pile and the stiffness variations within the soil. As with end bearing capacity, the limiting pressures available at depth are governed by a contained failure mechanism which depends on:

- The sand's shear stiffness, friction and dilation characteristics
- The relative roughness of the pile-soil interface

The overall lateral load-displacement behaviour is particularly sensitive to the soil's shear stiffness characteristics. Non-linear finite element approaches have been found to be far more appropriate than elastic methods or conventional P-Y analyses (Ganendra, 1994).

3.6 REFERENCES


*Effects of installing displacement piles in a high OCR clay.*
Geotechnique, 41(3), 341-363.

*Factors affecting the shaft capacity of displacement piles in clays.*

*Measured and predicted axial response of 98 piles.*

*Cavity expansion in cohesive frictional soils.*
Geotechnique, 36(3), 349-353.

*Field measurements of stress interactions between displacement piles in sand.*
Ground Engineering, 28(6), 36-40.

*Investigations into displacement pile behaviour for offshore foundations.*

*Research into the behaviour of displacement piles for offshore foundations.*

*The effects of time on the capacity of pipe piles in dense marine sand.*

*New design correlations for piles in sand.*

*Axial capacity of steel pipe piles in clay.*
Proc. Conf. on Geotech. Practice on Offshore Engng, Austin, Texas, 370-388.

*Piles and other deep foundations.*
Bearing capacity of driven model piles in dense sands from calibration chamber tests.

Finite element analysis of laterally loaded piles.

Predicting pile driveability: Heather as an illustration of the friction fatigue theory.
Ground Engng, 13, Apr., 15-37.

Evidence for scale effects in the end bearing capacity of open-ended piles in sand.


Research into the behaviour of offshore piles: field experiments in sand and clay.
Health & Safety Executive, OTH Report 93 401, HMSO, London.

Friction coefficients for piles in sands and silts.

Hutton Tension Leg Platform Foundations: an approach to the prediction of pile behaviour.
Géotechnique, 38(2), 231-252.

Magnus foundations: Soil properties and predictions of field behaviour.

KISHIDA H. (1967).
The ultimate bearing capacity of pipe piles in sand.
*Behavior of sand plugs in open-end steel pipe piles.*  

*Dimensionnement des pieux de fondation offshore dans les sables dense.*  

*The behaviour of a displacement pile in Bothkennar clay.*  

*Mechanisms of shaft friction in sand from instrumented pile tests.*  

*Skin friction of piles in calcareous sands.*  
Proc. Conf. on Engineering for Calcareous Sediments, 239-244, Balkema, Rotterdam.

*Toe resistance evaluation in open toe piles.*  

*The mechanism of pile plugging in sand.*  

*The effects of plugging on pile performance and design.*  

*A new look at the phenomenon of offshore pile plugging.*  
Marine Geotechnology, 8, 213-230.

*One-dimensional analysis of soil plugs in pipe piles.*  
Geotechnique, 41(4), 587-598.

*Soil moduli and damping factors from dynamic response analyses.*
EERC Report No 70-10, Berkeley, California, USA.

Deformation characteristics of soils and rocks from field and laboratory tests.  

VESIC A.S. (1967).  
A study of bearing capacity of deep foundations.  
Final Report, Project B-189, Georgia Inst. of Tech., Atlanta.

The development of shaft friction and end bearing for piles in homogeneous and layered soils.  
PhD thesis, Polytechnic of South Wales.

Installation stress conditions: piles penetrometers in sands.  
Informal contribution to Amoco/Fugro-McClelland sponsored seminar on piles in sand.

Finite cavity expansion in dilatant soils: loading analysis.  
Geotechnique, 41(2), 173-183.
4. EXISTING DESIGN PROCEDURES

4.1 INTRODUCTION

4.1.1 Axial capacity of single piles
The ultimate axial capacity of a single pile ($Q$) is commonly expressed as the sum of its ultimate end-bearing resistance ($Q_b$) and its ultimate shaft resistance ($Q_s$):

$$Q = Q_b + Q_s$$

The allowable load for a single pile is calculated either by dividing the ultimate axial capacity by a single, "lumped", factor-of-safety ($F$):

$$Q_{allowable} = \frac{Q}{F} = \frac{Q_b}{F} + \frac{Q_s}{F}$$

or by dividing the individual components by partial factors ($F_b$ and $F_s$):

$$Q_{allowable} = \frac{Q_b}{F_b} + \frac{Q_s}{F_s}$$

Sections 4.2 and 4.3 discuss ways of determining the values of $Q_b$ and $Q_s$ to use in these equations.

4.1.2 Axial capacity of pile groups
The axial capacity of closely-spaced piles is usually determined as the lesser of:

- The sum of the capacities of the individual piles
- The capacity of the piles acting as a group

For piles in sand, the group capacity may be higher than the sum of the capacities of the individual piles (API, 1993b).

4.1.3 Vertical load-displacement behaviour
The load-column method of Coyle and Reese (1966) is widely used for predicting the axial load-displacement curves of single offshore piles. The pile is treated as an elastic column which is discretised into elements. Non-linear springs act on its boundaries and the shaft shear forces ($t$) applied to each pile element are assumed to be related uniquely to the absolute vertical movements ($z$) of that element. A similar assumption is made for the base-load. The $t-z$ and base-load curves vary with soil type and incorporate the standard design assumptions concerning peak local skin friction and tip resistance. In falling branch methods a post-peak local softening characteristic may be included.
Group interaction effects are usually assessed through elastic analyses of the piles embedded in a linear soil continuum. Finite element, boundary element, and approximate closed-form solutions are available (Poulos, 1989). However, field measurements show that the standard approaches can seriously over-predict group settlements under working load (see Jardine and Potts, 1988 and 1992). To make accurate predictions it is essential to account for soil non-linearity and high stiffness at small strains.

4.1.4 Lateral capacity of single piles
Classical methods of calculating the ultimate lateral capacity of single piles assume simplified failure mechanisms and pile/soil stress distributions (e.g. Reese et al., 1970). Large movements are required to develop full lateral capacity. The uppermost soil layers play the greatest role in controlling pile head movements (Jardine and Christoulas, 1991). Formulae for determining the lateral bearing capacity of piles in sand are given in the American Petroleum Institute's *Recommended practice for planning, designing, and constructing fixed offshore platforms* (API RP 2A).

4.1.5 Lateral capacity of pile groups
A group of piles with the same pile head fixity conditions would normally experience greater lateral deflection than that of a single pile under the average load of the group. The major factors influencing the deflection of the group and the distribution of load between its piles are (API, 1993b):

- Pile spacing
- Ratio of pile penetration to diameter
- Pile flexibility relative to that of the soil
- Dimensions of the pile group
- Variations in the soil's stiffness with depth

Further discussion of the lateral capacity of pile groups is outside the scope of this report.

4.1.6 Lateral load-displacement behaviour
The load/deflection behaviour of piles in sand is usually calculated using subgrade reaction analyses, in which the net horizontal pressure (p) acting on the pile at any given depth is uniquely related to the pile movement at that depth (y). Computer codes have been written that allow non-linear load-deflection (p-y) curves and variations in soil and pile properties with depth to be specified. API RP 2A recommends a non-linear p-y curve for sand based on back-analyses of horizontal pile load tests.

4.2 SHAFT CAPACITY OF SINGLE PILES
The shaft capacity of a single pile (Q_s) is equal to the product of the average shaft friction acting on its surface (τ_s) and the total surface area of its shaft (A_n):

62
\[ Q_s = \int_0^L \tau_s \pi D \, dz = \bar{\tau}_s \pi D L = \bar{\tau}_s A_s \]

where \( D \) is the pile's diameter and \( L \) its embedded length.

### 4.2.1 API recommendations

The American Petroleum Institute's *Recommended practice for planning, designing, and constructing fixed offshore platforms* (API RP2A) gives the following formula for determining the ultimate shaft resistance of a single pile in sand:

\[ \tau_s = K \sigma'_v \tan \delta \times \tau_{\text{max}} \]

where \( K \) is a dimensionless earth pressure coefficient; \( \sigma'_v \) is the original vertical effective stress in the ground; \( \delta \) is the angle of interface friction between the sand and the pile wall; and \( \tau_{\text{max}} \) is the limiting value of shaft friction.

Since it first appeared in 1969, API RP2A has recommended a number of different values for \( K \), \( \delta \), and \( \tau_{\text{max}} \), as summarized in Table 3.

According to Toolan and Horsnell (1992), the changes made in the 3rd Edition of RP2A may have been triggered by the results of pile tests in silica sand presented by McClelland (1974), although their background is not entirely clear.

The changes to the earth pressure coefficients (\( K \)) that were made in the 15th Edition of API RP2A have not been universally adopted, for fear that they are unconservative for piles installed in North Sea sands (Toolan and Ims, 1988). The 19th Edition included a warning that the existing recommendations may not be reliable.

Current UK practice is to use the recommendations given in the 15th Edition of API RP2A for \( \delta \) and \( \tau_{\text{max}} \) but to use the following values for \( K \):

- 0.7 in compression
- 0.5 in tension

Typical North Sea practice is to limit the value of \( \tau_s \) to 120kN/m² (de Ruiter and Beringen, 1979).
Table 3
Chronology of changes to API RP2A recommendations for shaft resistance of piles in sand (after Toolan and Horsnell, 1992)

<table>
<thead>
<tr>
<th>API RP2A edition of publication</th>
<th>Year</th>
<th>Soil type (medium-dense to dense, except where stated)</th>
<th>( \delta ) (deg)</th>
<th>( \tau_{\text{max}} ) (kPa)</th>
<th>Earth pressure coefficient, K</th>
</tr>
</thead>
<tbody>
<tr>
<td>1st</td>
<td>1969</td>
<td>Clean sand</td>
<td>30</td>
<td>96</td>
<td>0.7</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Silty sand</td>
<td>25</td>
<td>82</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Sandy silt</td>
<td>20</td>
<td>67</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Silt</td>
<td>15</td>
<td>48</td>
<td></td>
</tr>
<tr>
<td>3rd</td>
<td>1972</td>
<td>u/c</td>
<td>u/c</td>
<td>w/d*</td>
<td>0.7</td>
</tr>
<tr>
<td>15th</td>
<td>1984</td>
<td>Dense gravel</td>
<td>35</td>
<td>115</td>
<td>0.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Very dense sand</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Dense sand</td>
<td>30</td>
<td>96</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Very dense sand-silt</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Medium dense sand</td>
<td>25</td>
<td>81</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Dense sand-silt</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Loose sand</td>
<td>20</td>
<td>67</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>M. dense sand-silt</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Dense silt</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Very loose sand</td>
<td>15</td>
<td>48</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Loose sand-silt</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Medium dense silt</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>20th</td>
<td>1993</td>
<td>LRFD version of API-RP2A issued (see section 4.4)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

\( u/c = \text{unchanged; } w/d = \text{withdrawn} \)

*Includes a comment that \( \tau_{\text{max}} \) should be determined for local conditions

The variation in shaft resistance with depth below mudline according to UK practice is illustrated on Figure 36. The limiting values of shaft resistance come into play at about 25m in compression and about 35m in tension.

According to expert opinion, the conservatism in the current API recommendations for \( \tau_{\text{max}} \) may be as much as 10-30%, whereas there is little (< 10%) conservatism in the recommendations for \( \delta \) and K (Lacasse and
Goulois, 1989).

**Figure 36**

Variation in shaft resistance with depth below mudline according to UK practice (after Toolan and Horsnell, 1992)

4.2.2 Methods based on standard penetration tests

According to Meyerhof (1976), the *average* ultimate shaft resistance of a driven pile in sand (\( \bar{\tau}_s \), in kN/m²) is given roughly by:

\[
\bar{\tau}_s = 2\bar{N}
\]

where \( \bar{N} \) is the average standard penetration resistance* (in blows per 0.3m penetration) over the embedded length of the pile.

Coyle and Castello (1981) used the results of full-scale load tests on 16 piles driven into sand and silt to develop correlations between average shaft resistance (\( \bar{\tau}_s \)) and:

- Average relative depth (L/2D) of the piles
- Angle of friction of the soil

---

*The standard penetration test is not carried out routinely in offshore investigations and there are, in any event, uncertainties over the energy corrections that would need to be applied*
Figure 37 shows the relationships established between \( \bar{\tau}_x \) and relative depth, for friction angles of 30, 33, and 37°. Soil parameters were based on the results of standard penetration tests, in conjunction with soil descriptions. Shaft resistances were determined in tension load tests.

Briaud and Tucker (1984) described a method for calculating the shaft resistance of piles in sand, taking account of residual driving stresses. Hyperbolic transfer curves were derived from the results of 33 instrumented pile tests at ten sites. Unfortunately, the correlation coefficients obtained for the parameters that define the transfer curves are far too low to make the method reliable as a predictive tool (the standard deviation for the ratio of predicted to measured shaft capacity is as high as 0.38).

4.2.3 Methods based on cone penetration tests

Cone penetration tests provide measurements of cone resistance and local side friction that can be used to estimate the ultimate shaft resistance of driven piles in sand.

Normal practice is to determine the average shaft friction acting on the pile (\( \tau_s \)) by multiplying the cone resistance (\( q_c \)) by a factor (\( \beta_c \)):

\[
\tau_s = \beta_c q_c
\]

where \( \beta_c \) is typically between 1/100 and 1/300 (Meigh, 1987). Alternatively, \( \tau_s \) can be calculated by multiplying the local side friction (\( f_s \)) by a factor (\( \beta_s \)):
\[ \tau_s = \beta_s f_s \]

where $\beta_s$ is typically between 0.7 and 1.0.

Since measurements of cone resistance are often more accurate and more easily interpreted than those of local side friction, the ultimate shaft resistance of piles is frequently based on $q_c$ rather than $f_s$ (Meigh, 1987).

Kraft (1990) developed a relationship between the earth pressure coefficient ($K$) and the relative density of sand ($D_r$), as illustrated on Figure 38, based on the following assumptions:

- In a very loose sand ($D_r = 0\%$), the lateral stresses acting on a low displacement pile after installation are equal to the initial at-rest stresses in the ground (i.e. about $0.5\sigma_0'$), which, during loading, reduce by approximately 20% owing to the sand's contractive nature. Hence, the value of $K$ at failure is approximately 0.4.
- In a medium dense sand ($D_r = 50\%$), the lateral stresses at failure equal the at-rest stresses (i.e. $K = 0.5$).
- In a very dense sand ($D_r = 100\%$), the lateral stresses at failure equal the overburden pressure (i.e. $K = 1$).

![Figure 38](image_url)

**Figure 38**

Relationship between the earth pressure coefficient ($K$) and relative density ($D_r$) assumed by Kraft (1990)

Kraft recommended that the angle of interface friction ($\delta$) be taken as 0.7 times the soil's angle of internal friction ($\phi$) and that shaft friction ($\tau_s$) should not, in principle, be limited (although, in practice, $\tau_s$ should be limited to 190 kPa when there is no cone or laboratory shear data to support shearing resistances in excess of this value).
The variation in shaft resistance with depth below mudline according to Kraft's method is compared with current UK practice on Figure 39. Kraft comments that the lower values of $\tau$, in the top 20-40m (for compression) are compatible with experience and fundamentals, and that there is no data to support the continued use of limiting values (except as noted above). In Kraft's opinion, it is more likely that the shaft resistance in dense to very dense sands will be underestimated by his criterion than overestimated.

4.2.4 Methods based on pressuremeter tests

The shaft capacity of piles in sand can be estimated from the limit pressure ($p_{lm}$) measured in Menard pressuremeter tests, using the curves shown on Figure 40. These curves are based on the results of nearly 240 pressuremeter tests at 63 different sites, on 115 piles (of which 89% were instrumented all along the shaft, mainly with removable extensometers).

The pressuremeter design curves are used extensively in France, where they are incorporated in the Laboratoire des Ponts et Chaussées/SETRA (1985) recommendations for foundation design.

![Figure 39](image)

Variation in shaft resistance with depth below mudline according to Kraft (1990)
4.2.5 Limiting values of shaft resistances

Arguments for and against the application of limiting values of shaft resistance in the design of piles in sand have been presented by a number of authors.

The concept of a critical depth — below which the shaft and end-bearing resistance of a pile remains essentially constant — is commonly attributed to Vesic (1967). In fact, it was Kerisel (1961) who originally put forward the hypothesis. Vesic used the critical depth idea to explain the results of laboratory tests and (in a paper published in 1970) field tests.

The explanation usually given for limiting values of shaft resistance is a reduction in horizontal stress with increasing depth owing to arching.

Kulhawy (1984) argued that the shaft resistance of a pile appears to reach a limit at a critical depth because of the counteracting effect of the product of the earth pressure coefficient (K) and the angle of interface friction (δ) decreasing with depth at the same rate as the vertical effective stress (σ′ v) increases with depth. The net effect is for

\[ \tau_v = (K \tan \delta) \sigma'_v \]

to remain essentially constant (or to increase very slowly) with depth.
Kraft (1991) gives a comprehensive critique of limiting values as applied to axially loaded piles in sand and concludes that limiting values do not, in general, exist. Kraft explains the downward curvature of the shaft-resistance versus depth curve as being a consequence of:

- Soil-to-soil (\( \phi \)) and pile-to-soil (\( \delta \)) friction angles decreasing with depth as a result of the increase in effective overburden pressure (\( \sigma' \))
- Sands exhibiting less dilative behaviour with depth as a result of the increase in \( \sigma' \)
- Horizontal stresses increasing from at-rest values owing to displacements caused by pile installation, but then decreasing once the pile toe passes owing to vibrations induced by pile driving

Shaft resistances that exceed the API limiting values have been measured in cone penetration tests to 100m depth (McNeilan and Bugno, 1984) and Kraft argues that cone penetrometers are more likely to underestimate than overestimate the shaft resistance of full-displacement piles.

### 4.3 END BEARING CAPACITY OF SINGLE PILES

#### 4.3.1 API recommendations

The American Petroleum Institute's *Recommended practice for planning, designing, and constructing fixed offshore platforms* (API RP2A) gives the following formula for determining the end-bearing resistance (\( Q_b \)) of a single pile in sand:

\[
Q_b = q_b A_b = \sigma_{vb} N_q A_b + q_{max} A_b
\]

where \( A_b \) is the area of the base of the pile; \( \sigma_{vb} \) is the original vertical effective stress in the ground at the pile toe; \( N_q \) is a dimensionless bearing capacity factor; and \( q_{max} \) is the limiting value of end-bearing.

Since it first appeared in 1969, API RP2A has recommended a number of different values for \( N_q \) and \( q_{max} \), as summarized in Table 4.

Current UK practice is to use the recommendations given in the 15th Edition of API RP2A. The variation in end-bearing resistance with depth below mudline with these parameters is illustrated on Figure 41. The limiting values of end-bearing resistance come into play at around 24m depth.
<table>
<thead>
<tr>
<th>API RP2A edition</th>
<th>Year of publication</th>
<th>Soil type (medium-dense to dense, except where stated)</th>
<th>( N_q )</th>
<th>( q_{\text{max}} ) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1st</td>
<td>1969</td>
<td>Clean sand</td>
<td>40</td>
<td>9.6</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Silty sand</td>
<td>20</td>
<td>4.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Sandy silt</td>
<td>12</td>
<td>2.9</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Silt</td>
<td>8</td>
<td>1.9</td>
</tr>
<tr>
<td>3rd</td>
<td>1972</td>
<td>u/c</td>
<td>u/c</td>
<td>w/d*</td>
</tr>
<tr>
<td>9th</td>
<td>1977</td>
<td>u/c</td>
<td>u/c†</td>
<td>u/c</td>
</tr>
<tr>
<td>15th</td>
<td>1984</td>
<td>Dense gravel</td>
<td>50</td>
<td>12.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Very dense sand</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Dense sand</td>
<td>40</td>
<td>9.6</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Very dense sand-silt</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Medium dense sand</td>
<td>20</td>
<td>4.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Dense sand-silt</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Loose sand</td>
<td>12</td>
<td>2.9</td>
</tr>
<tr>
<td></td>
<td></td>
<td>M. dense sand-silt</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Dense silt</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Very loose sand</td>
<td>8</td>
<td>1.9</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Loose sand-silt</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Medium dense silt</td>
<td></td>
<td></td>
</tr>
<tr>
<td>20th</td>
<td>1993</td>
<td>LRFD version of API-RP2A issued (see section 4.4)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

\( u/c = \text{unchanged}; \) \( w/d = \text{withdrawn} \)

*Includes comment that \( q_{\text{max}} \) should be determined for local conditions

†Includes statement that adequate penetration into sand layer had to be obtained to guarantee \( N_q \)
According to expert opinion, the conservatism in the current API recommendations may be as much as 25-40% for $q_{max}$ and as much as 35-50% for $N_q$ (Lacasse and Goulois, 1989).

### 4.3.2 Punching failure

Bearing capacity theories have been presented by, amongst others, Terzaghi (1943), Meyerhof (1952), Caquot and Kerisel (1956), Berezantsev et al. (1961), Brinch-Hansen (1961), and Vesci (1972). Values of the bearing capacity factor ($N_d$) from these various authors are compared with the recommended values from API RP2A (15th edition) in Figure 42. As can be seen, the API recommendations are considerably lower than the values given by the other theories.
Figure 42
Bearing capacity factor $N_q$ according to various authors compared with recommendations of API RP2A

4.3.3 Cavity expansion
Movement of the pile tip through the ground has been likened to the expansion of a spherical cavity. Solutions for the limit pressure required to cause infinite expansion of a spherical cavity in frictional, dilatant, soil have been presented by Carter et al. (1986), Yu and Houtsby (1991), and Collins et al. (1992).
4.3.4 Methods based on standard penetration tests

According to Meyerhof (1976), the ultimate end-bearing resistance of a driven pile in sand \((q_b\), in kN/m²) is given approximately by:

\[
q_b = \frac{40 \ N \ L}{D} \leq 400 \ N
\]

where \(N\) is the average standard penetration resistance (in blows per 0.3m penetration) near the base of the pile; \(L\) is the embedded length of the pile; and \(D\) is the pile's diameter.

Coyle and Castello (1981) established a relationship between end-bearing resistance and:

- Relative depth (L/D) of the piles
- Angle of friction of the soil

as shown in Figure 43.

![Figure 43](image)

**Figure 43**

Relationship between end-bearing resistance and friction angle (Coyle and Castello, 1981)

Briaud and Tucker (1984) described a method for calculating the end-bearing resistance of piles in sand, taking account of residual driving stresses (see page 66). The correlation coefficients obtained for the equations defining the end-bearing are too low to make the method reliable as a predictive tool (the standard deviation for the ratio of predicted to measured end-bearing capacity is as high as 0.55).
4.3.5 Methods based on cone penetration tests

Kraft's (1990) approach to estimating end-bearing in sands is to rely on direct measurements of end-bearing resistance from cone penetration tests together with appropriate adjustments to account for scale effects and soil variability. The ultimate end-bearing resistance of a pile is given by:

\[ q_b = V_k q_c \]

where \( V_k \) is a correction factor for variability in an otherwise homogeneous stratum and \( q_c \) is the cone resistance. Recommended values of \( V_k \) are:
- 0.5 for very dense and dense sands
- 0.55 for medium-dense sands
- 0.6 for loose and very loose sands

The variation in end-bearing resistance with depth below mudline according to Kraft's method is compared with the recommendations of API RP2A on Figure 44.

![Graph showing variation in end-bearing resistance with depth below mudline](image)

**Figure 44**
Variation in end-bearing resistance with depth below mudline according to Kraft (1990)

4.3.6 Methods based on pressuremeter tests

The end-bearing resistance of piles in sand can be estimated from the limit pressure (\( p_{lim} \)) measured in Menard pressuremeter tests, using the formula:
\[ q_h = k (p_{lim} - \sigma_v) + \sigma_v \]

where \( k \) is an empirical bearing capacity factor; \( \sigma_h \) is the original horizontal total stress; and \( \sigma_v \) the original vertical total stress at the pile tip.

For full-displacement piles in sand or gravel, the empirical factor \( k \) is typically 3.2 in dense sand and 4.2 in loose (Baguelin et al., 1986). Values of \( k \) for intermediate densities should be interpolated from these values.

The original horizontal total stress in the ground is obtained from the pressuremeter test and the vertical total stress is estimated on the basis of soil density and depth.

4.3.7 Limiting values of end bearing

The most conclusive evidence of limiting end-bearing resistance is provided by tests on model piles in dry sand. Kraft (1991) argues that the results of such tests are influenced by the boundaries of the test chambers and are not representative of the behaviour of piles in the field.

According to Kraft (1991), Vesic does not necessarily apply limiting values of end-bearing resistance if a reduction in friction angle with increasing overburden pressure and soil compressibility is accounted for in the bearing capacity equations: the only exception would be in dense sands where arching might occur to reduce the stresses and result in a limiting end-bearing resistance.

Kulhawy (1984) argued that the downward curvature of the end-bearing resistance versus depth curve is caused by the sand's friction angle \( (\phi) \) and rigidity index \( (I_r) \) decreasing with depth.

Cone resistances that exceed the API limiting values by a factor of four have been measured in cone penetration tests to 60m depth in dense sand (McNeilan and Bugno, 1984).

4.3.8 Build-up/drop-off in bearing capacity in layered soils

The end-bearing resistance of piles in layered soils depends to a large extent on the relative strength of the layers and the position of the pile toe relative to the boundary between them. The resistance drops-off over a distance \( y_{sw} \) (see Figure 45) as the pile toe approaches a weak layer underlying a strong layer; conversely, the resistance builds-up over a distance \( y_{ss} \) as the toe enters into a strong layer underlying a weak layer.
Meyerhof (1976) suggests that the build-up of end-resistance as a pile enters a strong layer underlying a weak layer takes place over a distance approximately ten times the diameter of the pile, i.e.:

\[ y_{ws} \approx 10 \ D \]

According to Kraft (1990), data from full-scale pile load tests suggests that the transition zone varies between five and fifteen diameters in length. The value of \( y_{ws} \) should decrease as the:
- Difference in strength between the layers decreases
- Overburden stress increases
- Density of the sand increases

Kraft recommended using \( y_{ws} = 5D \) when the weak and strong layers are sands of different density.

Semple (1980) argued that the dilatancy of dense sands beneath a pile tip is suppressed by the high overburden pressures that apply at the depths associated with offshore piling, and hence \( y_{ws} \) may be closer to three times the pile's diameter than ten times.

Hanna (1981) has studied the effects of a weak layer of sand underlying a strong layer and has found that the end-bearing resistance of circular foundations drops-off over a distance \( y_{wr} \) equal to 2-3 times the foundation's diameter:

\[ y_{wr} = 2-3 \ D \]

Hobbs (1992) recommends that piles are founded at least five diameters above significantly weaker strata to guard against punch-through failure.
Hanna and Meyerhof (1980) have developed design charts for calculating the bearing capacity of foundations on sand overlying soft clay.

4.4 LOAD AND RESISTANCE FACTOR DESIGN

In 1989, the American Petroleum Institute issued for industry review a Draft recommended practice for planning, designing, and constructing fixed offshore platforms — load and resistance factor design (API RP2A-LRFD). In contrast to the working strength design approach of the existing API RP2A (hereafter denoted API RP2A-WSD), the approach of RP2A-LRFD is reliability-based: uncertainties that naturally occur in the determination of loads and member strengths are explicitly accounted for in the load and resistance factor method.

In 1993, the American Petroleum Institute adopted API RP2A-LRFD as an alternative to RP2A-WSD, which was re-issued at the same time in its 20th edition. The LRFD version is a self-contained document and it is not necessary to refer to RP2A-WSD to use it. The International Standards Organization is planning to issue an international code based in part on the contents of API RP2A-LRFD.

4.4.1 Philosophy of the WSD approach

Under the working strength design (WSD) approach, expected working loads are applied to a platform and safety is achieved by providing adequate margins between calculated member stresses and member limiting stresses at failure. These margins do not account for the different uncertainties associated with the various load sources. Depending on the ratio of gravity to environmental loading, different levels of reliability are achieved following WSD practice.

4.4.2 Philosophy of the LRFD approach

The load and resistance factor design (LRFD) approach explicitly accounts for load and resistance uncertainties and thereby achieves more uniform reliability. Loads are modified by factors chosen on the basis of the load uncertainties. Similarly, calculated resistances are reduced by a factor that accounts for the uncertainties associated with the predictability of the failure mechanism.

4.4.3 Calibration of the LRFD approach

Specific probabilities were not assumed in the development of API RP2A-LRFD: the selection of load and resistance factors was based on the same average reliability as provided in RP2A-WSD. Part of the reason for this was that adopting a common probability of failure would have led to significant changes in component sizes. These changes would have conflicted with the premise of relying heavily on prior experience and judgement.

4.4.4 Load factors

API RP2A-LRFD divides the loads that act on a platform into the following components:
• Dead load 1 (D₁): the self-weight of the structure
• Dead load 2 (D₂): the load imposed on the platform by the weight of equipment and other objects
• Live load 1 (L₁): the weight of consumable supplies and fluids in pipes and tanks
• Live load 2 (L₂): the short-duration force exerted on the structure from operations such as lifting of drill strings
• Extreme wind, wave, and current load (Wₑ): the force applied to the structure due to the combined action of the extreme wave (with typically a 100 year return-period) and associated current and wind
• Operating wind, wave, and current load (Wₒ): the force applied to the structure due to the combined action of the extreme wave during operating conditions
• Extreme inertial load (Dₙ): the nominal inertial load computed by an extreme-wave dynamic analysis

The LRFD practice requires each foundation component to be designed to resist the combined action of these loads (Q), where:

\[ Q = \gamma_{D₁}D₁ + \gamma_{D₂}D₂ + \gamma_{L₁}L₁ + \gamma_{L₂}L₂ + \gamma_{Wₑ}Wₑ + \gamma_{Wₒ}Wₒ + \gamma_{Dₙ}Dₙ \]

and the various load factors (\(\gamma_p\)) are as summarized in Table 5.

**Table 5**

<table>
<thead>
<tr>
<th>Combination</th>
<th>Load factor applied to...</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>D₁</td>
</tr>
<tr>
<td>Gravity loads</td>
<td>1.3</td>
</tr>
<tr>
<td>Wind, wave, &amp; current loads with gravity loads</td>
<td>1.1</td>
</tr>
<tr>
<td>Wind, wave, &amp; current loads with opposing gravity loads</td>
<td>0.9</td>
</tr>
<tr>
<td>Operating wind, wave, &amp; current loads with gravity loads</td>
<td>1.3</td>
</tr>
</tbody>
</table>

This approach differs from the approach taken by API RP2A-WSD, which recommends the application of "lumped" factors-of-safety of 2.0 for operating and 1.5 for extreme-environmental conditions. In terms of the components
Given above, the combined action of these loads ($Q$) is given by the greater of:

$$Q = 2.0 \left[ D_1 + D_2 + L_1 + L_2 \right]$$

and

$$Q = 1.5 \left[ D_1 + D_2 + L_1 + L_2 + W_e + W_o + D_n \right]$$

### 4.4.5 Resistance factors

According to API RP2A-LRFD, the maximum axial compression or tension in a pile, as calculated using factored loads, should be less than or equal to the ultimate axial pile capacity ($Q_u$) multiplied by a suitable resistance factor ($\Phi$):

$$\text{Maximum Axial load} \leq \Phi \cdot Q_u$$

The LRFD practice recommends $\Phi = 0.8$, both for compression and tension piles.

### 4.4.6 Commentary on API RP2A-LRFD

The load and resistance factor (LRFD) approach provides more consistent reliability, across a range of environmental to gravity load ratios, than does the working stress design (WSD) approach (Moses and Larrabee, 1988; Lloyd and Karsan, 1988).

The LRFD format is similar in many respects to the limit state format adopted by the Eurocodes and the ISO drafts. However, it lacks the rigorous definitions of limit states, design situations, load combination rules, and characteristic and design values of parameters (Arup, 1993).

The current draft of Eurocode 7, which is concerned mainly with buildings, proposes a partial factor ($\gamma_e$) equal to 1.0 on the weights of both soil and supported structures. Nevertheless, the load factors of 1.3 and 1.1 for gravity loads under operating and extreme environmental conditions should not cause difficulties in the geotechnical design of offshore structures. The values of the load factors in API RP2A-LRFD are fairly similar to those adopted in other offshore codes (Arup, 1993).

According to Toolan (1992), the load and resistance factor design approach has the advantage of taking risk and reliability into account and in the longer term, with fine tuning, should result in lower costs for a given specified level of safety.

The impact of a switch from the working stress design (WSD) approach of API RP2A-WSD on the design of piles in sand is illustrated on Figure 46. Piles designed according to LRFD practice would typically be up to 25% longer than piles designed using working stresses.
4.5 REFERENCES

Recommended practice for planning, designing and constructing fixed offshore platforms.  

AMERICAN PETROLEUM INSTITUTE (1972).  
Recommended practice for planning, designing and constructing fixed offshore platforms.  

Recommended practice for planning, designing and constructing fixed offshore platforms.  

Recommended practice for planning, designing and constructing fixed offshore platforms.  
*Draft recommended practice for planning, designing and constructing fixed offshore platforms — load and resistance factor design.*

*Recommended practice for planning, designing and constructing fixed offshore platforms.*

AMERICAN PETROLEUM INSTITUTE (1993a).
*Recommended practice for planning, designing and constructing fixed offshore platforms — load and resistance factor design.*

AMERICAN PETROLEUM INSTITUTE (1993b).
*Recommended practice for planning, designing and constructing fixed offshore platforms — working stress design.*

ARUP, OVE, and PARTNERS (1993).
*Geotechnical aspects of API RP2A-LRFD: draft recommended practice for fixed offshore platforms.*

*The pressuremeter for foundations: French experience.*

*Load bearing capacity and deformation of piled foundations.*

*Piles in sand: a method including residual stresses.*

BRINCH-HANSEN (1961).
*A general formula for bearing capacity.*
Ingeniøren (C) 5, No 2, 38-46.
*Prévision de la capacité portante des pieux isolés sous charge verticale.*  
Bull. liaison Labo. P. et Ch., 113, Mai-Juin, ref. 2536.

*Tables for the calculation of passive pressure, active pressure and bearing capacity of foundations.*  

*Cavity expansion in cohesive frictional soils.*  
Géotechnique, 36(3), 349-353.

*Cavity expansion in sands under drained loading conditions.*  
Int. J. for Num. and Analytical Methods in Geomechanics, 16(1), 3-23.

*New design correlations for piles in sand.*  

*Load transfer for axially loaded piles in clay.*  

*Foundations on strong sand overlying weak sand.*  

*Design charts for ultimate bearing capacity of foundations on sand overlying soft clay.*  

*A review of the design and certification of offshore piles, with reference to recent axial pile load tests.*  

*Recent developments in defining and measuring static piling parameters.*  
*Fondations profondes en milieu sableux.*  

*Computing axial pile capacity in sands for offshore conditions.*  
Marine Geotechnology, 9, 61-92.

*Performance of axially loaded pipe piles in sand.*  

*Limiting tip and side resistance: fact or fallacy?*  

LABORATOIRE DES PONTS ET CHAUSSEES/SETRA (1985).  
*Regles de justification des fonduations sur pieux a partir des resultats des essais pressiométriques.*  
Laboratoire Central des Ponts et Chaussees, Service d'Etudes Techniques des Routes et Autoroutes, Paris.

*Uncertainty in API parameters for prediction of axial capacity of driven piles in sand.*  

*Development of a reliability-based alternative to API RP2A.*  

*Design of deep penetration piles for ocean structures.*  

*Cone penetration test results in offshore California silts.*  

*Cone penetration testing: methods and interpretation.*  
*The bearing capacity of foundations under eccentric and inclined loads.*  

*Bearing capacity and settlement of pile foundations.*  

*Calibration of the draft RP2A-LRFD for fixed platforms.*  

*Pile behaviour — theory and application.*  
Rankine Lecture, Géotechnique 34(2), 365-415.

*Rational design concept for breasting dolphins.*  

*Pile foundations for large North Sea structures.*  

*Contribution to discussion of Session 8.*  

TERZAGHI K. (1943).  
*Theoretical soil mechanics.*  

*Geotechnical design and engineering for the 90's.*  
Proc. 6th Int. Conf. on Behaviour of Offshore Structures (BOSS '92), London, Supplement.

*The evolution of offshore pile design codes and future developments.*  

*Impact of recent changes in the API recommended practice for offshore piles in sand and clays.*

85
VESIC A.S. (1967).
A study of bearing capacity of deep foundations: final report.
School of Civ. Engng, Georgia Inst. of Tech., Atlanta.

Tests on uninstrumented piles. Ogeechee River site.

Expansion of cavities in infinite soil mass.

Finite cavity expansion in dilatant soils: loading analysis.
Géotechnique, 41(2), 173-183.
5. RECENT DEVELOPMENTS IN DESIGN METHODS FOR DRIVEN PILES IN SAND

5.1 INTRODUCTION

5.1.1 General
Earlier chapters of this report have set out how recent research has helped identify the basic mechanisms and soil characteristics that govern the behaviour of driven piles in sand. The research provided the background for the critical review of existing design methods that was presented in Chapter 4. The relatively poor reliability of current procedures can now be understood and ways of improving the situation can be identified.

5.1.2 Developments between 1990 and 1994
Between 1990 and 1994, proposals for new axial capacity design recommendations which account for recent research findings were published by four groups. Firstly, Tooan et al. (1990) suggested a series of modifications for shaft capacity calculations which were developed from Lings' (1985) critical review of the API database. Following from this, Yu and Houlsby (1991) considered how cavity expansion solutions could be applied to predict end bearing for fully plugged or closed ended piles. Then Lehane and Jardine (1994) presented new provisional design methods for shaft capacity. Their approach had been developed from an interpretation of the Labenne field research programme and from a critical review of other data. Finally, Randolph et al. (1994) proposed design methods which integrated a cavity expansion approach for end bearing capacity with a revised shaft resistance calculation method. The latter incorporated some of the key findings from work by Vesic (1970) and the Labenne field test programme.

5.1.3 Recent JIP (Joint Industry Project) research
Two field-based research projects have recently taken place which should influence the methods used in practice for assessing the capacity of offshore piles. Both projects involved dense marine sands that are representative of some North Sea sediments.

The first project was carried out by a group from Imperial College, London, with the intention to test hypotheses concerning:

- The effects of relative density and scale
- Pile tip details
- The effective stresses set up by driving and load testing
- Possible tip position (h/R) effects
- Tension versus compression loading
The field work was performed at Dunkirk, northern France, and the first element was a set of four tests on instrumented piles similar to those deployed at Labenne (Lehane et al., 1991). Measurements have been made of the effective stresses developed on the piles' shafts during installation, equalization, and load testing to failure. A study of pile group effects was also undertaken (Chow, 1995).

The second element of fieldwork at Dunkirk involved extending the CLAROM programme of tests performed on 324 mm diameter, open-ended driven pipe-piles (Brucy et al., 1991). Some of the CLAROM piles have been re-tested five years after first driving, showing important effects of pile age; others are being load-tested to failure for the first time (Chow and Jardine, 1996). In addition, a more advanced site characterization study is being performed. Following these field tests, improved predictive methods have been developed for both shaft and end resistance which have been verified against a newly assembled database of 65 reliable pile tests (Jardine and Chow, 1996).

The second relevant programme of research is the EURIPIDES project at Eemshaven, north Holland, where about 24m of looser and more mixed sediments overly deep very dense sands. An important Joint Industry Project (JIP) was organized to test highly instrumented large-scale (762mm diameter), open-ended pipe-piles. In 1995, two piles were driven 30m into a very dense sand and subjected to multiple load tests at various levels. Instruments were provided to study conditions within the soil plug, as well as on the pile shaft (Zuidberg and Vergobbi, 1996). It is possible that further tests will be undertaken to study the effects of time on shaft and base capacity.

5.1.4 Load-displacement behaviour

As mentioned in Chapter 3, considerable improvements have also been made in assessing load-displacement behaviour under working loads. Non-linear numerical methods based on parameters from advanced laboratory soil stiffness measurements now appear to offer far greater accuracy than conventional approaches (the latter usually involve "t-z" and "p-y" routines combined with linear elastic pile group calculations). Such improvements were demonstrated through comparisons with field measurements taken at the Magnus and Hutton TLP sites (see Jardine and Potts, 1988 and 1992, and Ganendra, 1994). However, the effectiveness of the non-linear approach has yet to be assessed at a predominantly sandy site.

5.2 NEW METHODS FOR CALCULATING END-BEARING RESISTANCE

The following paragraphs discuss new methods of calculating the end-bearing resistance of open- and closed-ended piles.
5.2.1 Closed-ended and plugged piles
Randolph and Dolwin (1992) proposed calculating end-bearing capacity on the basis of the limit pressure for spherical cavity expansion. This requires knowledge of the sand's relative density, constant-volume angle of friction, and operational shear modulus and relies on recently published solutions for the expansion of a spherical cavity in frictional-dilatant soils (Carter et al., 1986; Yu and Houlsby, 1991; Collins et al., 1992). Randolph and Dolwin saw this approach as complementary to estimation of end-bearing capacity directly from cone data.

5.2.2 Open-ended piles
Randolph et al. (1991) and others have shown that for drained loading there is an exponential increase in plug capacity with plug length:

$$q_{pd} = (e^\alpha - 1) \left( p + \frac{(\gamma - \gamma_w)h}{\alpha} \right) - (\gamma - \gamma_w)h$$

where $p$ is the effective surcharge at the top of the plug; $\gamma$ is the unit weight of the soil and $\gamma_w$ that of water; $h$ is the plug length; $d$ is the pile diameter; and $\alpha$ is given by:

$$\alpha = \frac{4\beta h}{d} = \frac{4h}{d} \left( \frac{\tau_i}{\sigma'_v} \right)$$

where $\tau_i$ is the average internal skin friction in the pile; $\sigma'_v$ is the vertical effective stress; and $\beta (= \tau_i/\sigma'_v)$ probably lies between 0.15 and 0.25.

Randolph et al. (1991) have extended the analysis to consider the effects of partial drainage and present the design charts shown in Figure 47. Observations support the one dimensional approach in small diameter piles in which arching occurs, even in shallow plugs, and in which the major part of the deformation occurs within two pile diameters from the plug toe. Field evidence presented in Chapter 3 suggests it may not apply in large diameter piles.

Hight et al. (1996) summarized a comprehensive study into the end bearing capacity available to open-ended piles, which involved numerical, model and field research. Their data suggest that there are large effects of scale and that cavity-expansion solutions, the current API recommendations, and the use of unmodified cone tip resistance are all likely to be non-conservative in predicting allowable end-bearing values.
Figure 47
Design charts for determining plug capacity (Randolph et al., 1991)
Jardine and Overy (1996) suggested that the lower limit available to open ended piles when settlements are less than 10% of diameter might be given by:

\[ Q_e = S_e A_b q_e \]

where \( A_b \) is the annular area; \( S_e \) is a shape factor (which may be slightly less than unity); and \( q_e \) is the cone tip resistance. In this equation, all inner plug friction is ignored.

Chow (1996) compiled a new database of high quality measurements of base resistance, from which he proposed the design procedure set out in Table 6. Jardine and Chow (1996) have subsequently verified that the new method is reliable for offshore use.

5.3 NEW METHODS FOR CALCULATING SHAFT RESISTANCE

Three new design methods for calculating the shaft resistance of piles in sand are outlined below.

The first was developed by Toolan et al. (1990) from Lings' (1985) critical review of the American Petroleum Institute's pile test database. Lings' study was prompted by the controversial recommendations made in the 15th edition of the API's *Recommended practice for planning, designing, and constructing offshore platforms* (API RP2A). Lings' treatment of the API database is discussed in Appendix A.

The second was initially derived by researchers at Imperial College from the Labenne field testing programme and subsequently updated after further research (Chow and Jardine, 1996).

The third emerged from a review of recent developments by Randolph and Dolwin (1992), which has been circulated throughout the offshore industry by Amoco Production Company and is now in the public domain (Randolph et al., 1994).

5.3.1 Methods proposed by Toolan, Lings, and Mirza

Toolan, Lings, and Mirza (1990) made a thorough appraisal of the design recommendations given in the 15th edition of API RP2A and identified some important limitations of those recommendations. One feature that is missing from RP2A is an allowance for the average shaft resistance in sands tending to a quasi-constant value once a critical depth of penetration has been exceeded (see Chapters 3 and 6). Toolan et al. proposed two methods of allowing for this feature in design.
### Table 6
New Imperial College procedures for base capacity calculations in sands (after Jardine and Chow, 1996)

<table>
<thead>
<tr>
<th>A1</th>
<th>Closed-Ended Piles</th>
<th>[ Q_b = q_b \pi D^2 / 4 ]</th>
</tr>
</thead>
<tbody>
<tr>
<td>( q_b = q_d [1 - 0.5 \log(D/D_{CPT})] )</td>
<td>Pile base resistance</td>
<td>Related to the CPT end resistance. ( \bar{q}<em>b ) is averaged 1.5 pile diameters above and below the pile toe. Depends on pile diameter. Note ( D</em>{CPT} = 0.036m ) and a lower bound ( q_b = 0.13 \bar{q} ) applies for ( D &gt; 2m ).</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>B1</th>
<th>Base Capacity of Open-Ended Piles</th>
<th>A rigid basal plug can only develop if ( D &lt; 0.02 [D_s, \leq 30] )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( q_b = q_b \pi D^2 / 4 ) ( q_b = q_d [1 - 0.5 \log(D/D_{CPT})] / 2 )</td>
<td>Fully plugged piles</td>
<td></td>
</tr>
<tr>
<td>Develop 50% lower end resistance than comparable closed-ended piles after a pile head displacement of ( D/10 ). B3 provides a lower bound to B2 at large diameters.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>B3</th>
<th>Lower bound for unplugged and large open ended piles</th>
<th>( Q_b = q_{ba} \pi (R_o^2 - R_i^2) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( q_{ba} = q_c )</td>
<td>Allow end bearing on the annular base area of steel-only, resistance is equal to average CPT end resistance at the founding depth. Contributions from internal shear stresses should be ignored.</td>
<td></td>
</tr>
</tbody>
</table>

In Method 1, the chart shown in Figure 48 is used to assess the local shaft resistance at the pile tip, which depends only on the sand's relative density and whether the pile is closed- or open-ended. The shear stress is assumed to decrease linearly between the pile tip and the ground surface and may jump in value at the boundaries between layers of different density (as illustrated in Figure 49). Implicit in the method is the tendency for the local shaft resistance in any one soil horizon to fall linearly as the height of that horizon above the pile tip increases. The method predicts that, for a given uniform sand, the average shaft resistance is constant, independent of pile depth (or vertical effective stress). No account is taken of the sand's coefficient of friction.
Figure 48
Chart for determining skin friction at pile tip from relative density (Toolan et al., 1990)

Figure 49
Typical variation of local skin friction with depth (Toolan et al., 1990)
Toolan et al.'s second method has a more theoretical basis. It was developed in terms of the skin friction coefficient ($\beta$), defined as:

$$\beta = \frac{\tau_f}{\sigma_{vo}'}$$

where $\tau_f$ is the shaft friction acting on the pile shaft at failure and $\sigma_{vo}'$ is the original vertical effective stress in the soil. Again, the pile-soil interface friction angle is not included in the analysis.

Toolan et al. produced a design chart for determining the value of $\beta$ that applied near to the pile tip by combining:

- The relationship between the lateral stress developed on a cone penetrometer sleeve, the maximum dilation angle of the sand ($\psi$), and the original radial effective stress, as given by Hughes and Robertson (1985).
- Equations linking $\psi$ to the sand's angle of friction, mean effective stress, and relative density, as given by Bolton (1986).

Toolan et al. assumed that the pile-tip value of $\beta$ is 20% lower for open- than for closed-ended piles and adopted typical values for a number of other soil parameters.*

Figure 50 shows the resulting design chart for $\beta$ at the pile tip. As the figure

![Figure 50](image)

**Design chart for earth pressure coefficient ($\beta$) (after Toolan et al., 1990)**

*Toolan et al. adopted the following typical parameters for a sand: $K_0 = 0.4$; $\gamma' = 10$ kN/m$^3$; $\phi_{cs} = 34^\circ$; and $\delta = 29^\circ$.*
shows, $\beta$ varies strongly with the sand's relative density and declines with the overall penetration of the pile. The value of $\beta$ from the chart is assumed to apply over the bottom 10m of the pile only; above this level a fixed value of $\beta$ (equal to the lesser of the chart value and 0.24) is used.

The methods due to Toolan et al. incorporate some of the features of pile behaviour in sands that have been discovered in subsequent research. However, they do not give accurate predictions of the field behaviour. For example, the variation of shear stress with depth that was measured at Labenne was not triangular in shape, nor did it follow a sharp step function. Similarly, the methods do not allow for the effects of roughness on the shear behaviour at the pile/soil interface; sand size, stiffness, and grain shape are not considered; and the pile radius and initial stress level are ignored.

Nevertheless, the methods incorporate a vital parameter that was previously overlooked: the fact that local shaft resistance reduces as height above the pile tip increases. Toolan et al. showed that the inclusion of this feature leads to considerable improvements in the reliability of pile capacity predictions. Considering the 25 pile tests listed in their database, they obtained a ratio of calculated to measured shaft capacity ($Q_c/Q_m$) close to 1.0 with a standard deviation as low as 0.22 (using their first method). In contrast, predictions based on the 14th edition of API RP2A gave a mean value for ($Q_c/Q_m$) of 1.12 with a standard deviation of 0.67.

5.3.2 Methods proposed by Lehane and Jardine
The field research at Labenne led researchers at Imperial College, London, to propose two new design approaches, the first of which was based on a reinterpretation of Lings' database of tension pile tests in sand. The second method was derived directly from the results of the Labenne experiments on closed-ended piles (Lehane and Jardine, 1994) and has proved to be the more suitable for further development.

Lehane and Jardine's second method involves calculating the radial effective stress acting along the pile shaft at failure ($\sigma'_r$) and then obtaining the corresponding shear stress ($\tau$) from Coulomb's equation:

$$\tau = \sigma'_r \tan \delta$$

where $\delta$ is the angle of interface friction of the soil.

The radial effective stress acting along the pile shaft at failure is calculated in a series of steps, as follows:

- First, the radial effective stress acting along the pile shaft at the end of equalization ($\sigma'_r$) is determined from the results of cone penetration tests
- Second, the change in radial effective stress owing to dilation during pile loading ($\Delta \sigma'_r$) is determined from a knowledge of the shear stiffness characteristics of the soil
Finally, these two components are combined to give the radial effective stress at failure ($\sigma'_{re}$)

The tests at Labenne showed that the radial effective stress acting in any one soil horizon at the end of equalization ($\sigma'_{ew}$) is a function of the closed-ended end-bearing resistance of that horizon ($q_b$) and the normalized height of that horizon ($h/R$) above the pile tip (where $R$ is the radius of the pile). Lehane and Jardine established the following relationship between these parameters from a statistical analysis of over sixty sets of recordings on the same piles:

$$\sigma'_{re} = 0.024 \cdot q_b \left( \frac{h}{R} \right)^{-0.33}$$

A better fit to the database was obtained when the tendency for $\sigma'_{ew}/q_b$ to increase with depth was taken into account, resulting in the following equation:

$$\sigma'_{re} = 0.0286 \cdot q_b \left( \frac{\sigma'_{ew}}{P_A} \right)^{0.2} \left( \frac{h}{R} \right)^{-0.3}$$

where $\sigma'_{ew}$ is the original vertical effective stress in the soil and $P_A$ is standard atmospheric pressure.

This second equation gives an excellent fit to the Labenne data. It also matches independent field measurements made at Drammen by Gregersen et al. (1983). However, the range of stresses and sand types covered by these two sites is limited, so this equation must be regarded as provisional at present.

The second step in the method is to evaluate the change in radial effective stress ($\Delta \sigma'_r$) as the pile is loaded to failure. Lehane and Jardine assumed that, in compression tests, dilation at the pile-soil interface is the only mechanism that causes a change in radial effective stress and, in cases where the secant shear modulus of the sand ($G$) is known, the following equation can be used to evaluate $\Delta \sigma'_r$:

$$\Delta \sigma'_r = 2G \frac{d_r}{R}$$

where $d_r$ is the radial dilation of the sand at the pile/soil interface (equal to 2-3 times the surface roughness of the pile) and $R$ is the pile's radius. The best means of evaluating $G$ is to perform small-strain loops in full-displacement in situ pressuremeter tests, but a range of alternative techniques exists to determine this parameter. In cases where no suitable measurements of $G$ exist, Lehane and Jardine suggested using the following equation, which was derived from an analysis of six independent laboratory and field studies:

$$\Delta \sigma'_r = 4D_r \cdot P_A \left( \frac{\sigma'_{ew}}{P_A} \right)^{0.5} \frac{R_{CL}}{R}$$
where $D_r$ is the sand's relative density (expressed as a percentage) and $R_{CLA}$ is the centre-line average roughness of the pile surface.

Dilatancy is likely to be relatively unimportant for large offshore piles: for a 2m diameter pile driven into a very dense sand (with $D_r = 100\%$), the equation above suggests that $\Delta \sigma'_r$ would be less than 6kPa at a point 20m below the seabed.

The final step in Lehane and Jardine's method involves combining $\sigma'_w$ with $\Delta \sigma'_r$ to obtain the radial effective stress acting along the pile shaft at failure ($\sigma'_f$), using the equation:

$$\sigma'_f = \sigma'_w + \Delta \sigma'_r$$

for piles loaded in compression.

In tension tests at Labenne, the radial effective stress reduced significantly during the initial stages of pile loading, and hence Lehane and Jardine recommend reducing the contribution of the $\sigma'_w$ term by 20%, to give:

$$\sigma'_f = 0.8 \sigma'_w + \Delta \sigma'_r$$

for piles loaded in tension. They suggested, however, that a smaller reduction in $\sigma'_w$ might be appropriate for dense sands (following Lings, 1985).

Further reductions in radial effective stress could occur during tension loading of relatively extensible piles, owing to radial contractions of the pile shaft. This is apparent from elasto-plastic finite element analyses of the Hutton TLP piles (Jardine and Potts, 1988), which predicted an 8% reduction in radial effective stress in dense sand layers. Local shaft resistance is then calculated from:

$$\tau_f = \sigma'_f \tan \delta_{cr}$$

where $\delta_{cr}$ is the constant volume angle of interface friction measured in site-specific laboratory shear tests.

Lehane and Jardine demonstrated the reliability of their method using a database of more than 20 compression and tension field tests, performed at seven sites, on closed-ended or plugged piles. With two exceptions, all of the predictions fell within about 25% of the measurements. The mean ratio of calculated to measured shaft capacity ($Q_c/Q_m$) was 0.98, with a standard deviation of 0.21.

The Lehane and Jardine approach has also been applied to larger diameter open-ended piles. Comparisons with field tests are presented by Jardine and Overy (1996) and Chow et al. (1996) which indicate that the method can be used safely when three minor changes are made:

- A modified radius $R^*$ is substituted into the equation for calculating $\sigma'_w$. 

97
The open-ended R*-value is the radius of the solid cylinder which has the same area as the annular pile base $A_b$, so $R^* = (R^2_{\text{outer}} - R^2_{\text{inner}})^{0.5}$. The unmodified radius $R$ is used for all other parts of the calculation.

- A lower limit of 8 is specified for $h/R^*$ in the $\sigma'_{re}$ equation.
- A 15% reduction is applied to the equation for $\tau_{nf}$, the local shaft resistance.

The recent Joint Industry Project (JIP) research described by Chow and Jardine (1996) has led to further refinements being made to the recommendations for calculating shaft and end resistance, which are summarized in Table 7. Jardine and Chow show that the latest recommendations provide far more reliable predictions than the other available approaches.

**5.3.3 Method proposed by Randolph, Dolwin, and Beck**

The design approach proposed by Randolph, Dolwin, and Beck (1994) has several features in common with Lehane and Jardine's method. Both start from the assumption that the local radial effective stress acting at any point on the pile shaft is principally a function of:

- The end-bearing resistance of the pile at that point ($q_b$).
- The normalized height of that point above the pile tip ($h/R$).

Following Vesic (1970), Randolph *et al.* assumed that, in a compression test, the radial effective stress developed at failure at the base of the pile ($\sigma'_{rh}$) can be written as an exponential function of $q_b$ and the constant volume angle of friction of the sand ($\phi_{cv}$):

$$\sigma'_{rh} = a \ q_b \ \ e^{-b \ \tan \ \phi_{cv}}$$

where the constants $a = 2$ and $b = 7$ (on the basis of field data). With $\phi_{cv} = 32^\circ$, this equation reduces to $\sigma'_{rh} = 0.025q_b$ which is similar to Lehane and Jardine's equation for $\sigma'_{re}$ on page 97 (with $h/R = 0$).

Randolph *et al.* modelled the variation of $\sigma'_{re}$ with $h/R$ by assuming that the value of $K$ (defined as $\sigma'_{re}/\sigma_{vob}$, where $\sigma_{vob}$ is the original vertical effective stress in the soil) varies exponentially between two limits ($K_{\text{max}}$ and $K_{\text{min}}$) as defined below:

$$K = K_{\text{min}} + (K_{\text{max}} - K_{\text{min}}) \ e^{-\frac{h}{2R}}$$

where $K_{\text{max}} = \sigma'_{rh}/\sigma_{vob}$; $K_{\text{min}}$ may be linked to the active earth pressure coefficient ($K_a$); $h$ is the height above the pile tip; $R$ is the pile's radius; and $\mu$ is a constant between 0.025 and 0.1. Randolph *et al.* suggested a typical value for $\mu$ is 0.05.
Table 7

New Imperial College procedures for shaft capacity calculations in sands (after Jardine and Chow, 1996)

**CLOSED-ENDED PILES**

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Formula</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>$Q_\delta = \pi D \int \tau_f , dz$</td>
<td>Shaft capacity Integral of local shear stresses along the embedded shaft length.</td>
</tr>
<tr>
<td>C2</td>
<td>$\tau_f = \sigma'_{rt} \tan \delta_f$</td>
<td>Local shear stress</td>
</tr>
<tr>
<td>C3</td>
<td>$\sigma'<em>{rc} = \sigma'</em>{rc} + \Delta \sigma'_{rd}$</td>
<td>Coulomb failure criterion.</td>
</tr>
<tr>
<td>C4</td>
<td>$\Delta \sigma'_{rd} = 2G \delta h / R$</td>
<td>Local radial effective stress Function of CPT resistance, free-field vertical effective stress (normalised by atmospheric pressure, $P_a = 100$ kPa) and $h/R$; $\tau_f$ is not corrected for OCR. $h/R$ is limited to a minimum of 8.</td>
</tr>
<tr>
<td>C5</td>
<td>$\delta_f = \delta_{ov}$</td>
<td>Dilatant increase in local radial effective stress during pile loading Related to sand shear stiffness, pile roughness $R_{ch}$ and radius. $G$ taken from Baldi et al.'s (1989) relationship with CPT $q_c$. $\delta h = 2R_{ch} = 2 \times 10^{-5} m$ for typical offshore piles.</td>
</tr>
<tr>
<td>C6</td>
<td>$\tau_f = (0.8 \sigma'<em>{rc} + \Delta \sigma'</em>{rd}) \tan \delta_f$</td>
<td>Tension loading Equation C6 should be used in place of Equation C2.</td>
</tr>
</tbody>
</table>

**OPEN-ENDED PILES**

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Formula</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>D3</td>
<td>$\sigma'<em>{re} = 0.029 q_c x (\sigma'</em>{vo}/P_a)^{0.13} x (h/R)^{0.38}$</td>
<td>Modified radius, $R^<em>$ Substituted into Equation C3 to give D3; $h/R^</em> &gt; 8$.</td>
</tr>
<tr>
<td></td>
<td>$R^* = (R_{outer}^2 - R_{inner}^2)^{1/3}$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\tau_f = 0.9 (0.8 \sigma'<em>{rc} + \Delta \sigma'</em>{rd}) x \tan \delta_f$</td>
<td>In tension Shear stresses in tension reduced by 10%.</td>
</tr>
</tbody>
</table>

Randolph et al. calculated the shaft friction acting on the pile from the equation:

$$\tau_f = K \sigma'_v \tan \delta$$
where $\delta$ is the angle of interface friction of the soil. In the absence of measured values of $\delta$, they suggest using $\delta = \phi_{cv} - 5^\circ$.

Randolph et al. assumed that $K$ is relatively insensitive to pile diameter and they do not take account of changes in radial effective stress during pile loading in the way that Lehane and Jardine did. However, they stated that the total shaft capacity available in tension ($Q_{tens}$) may be lower than that in compression ($Q_{comp}$) owing to radial contractions of the pile diameter and general reductions in the vertical loads acting in the soil. The following expression for the ratio of these two capacities was proposed:

$$\frac{Q_{tens}}{Q_{comp}} = \left( 1 - \log_{10} \left( \frac{200}{L} \right) \right) \left( 1 - 8.5 \eta + 30 \eta^2 \right)$$

where $L$ is the length of the pile; $R$ its radius; and $\eta$ is a non-dimensional group of parameters defined as follows:

$$\eta = v_p \tan \delta \left( \frac{L}{2R} \right) \left( \frac{G}{E_p} \right)$$

where $E_p$ is the pile's Young's modulus; $v_p$, its Poisson's ratio; $G$ is the soil's shear stiffness; and $\delta$ its angle of interface friction.

As noted above, Randolph et al.'s method starts by establishing the profile of closed-ended (or fully plugged) end-bearing resistance ($q_b$) against depth. They also suggested elasto-plastic cavity expansion theory can be used, which, for a fully-plugged pile, results in the equation:

$$q_b = P_{lim} \left( 1 + \tan \phi_{cv} \tan \left( 45 + \frac{\phi_{cv}}{2} \right) \right)$$

where $P_{lim}$ is the limit pressure for spherical cavity expansion; and $\phi_{cv}$ is the soil's angle of friction at constant volume. Randolph et al. discuss how $\phi$, $G$, and dilation angle might vary with silt content and relative density. Inevitably, the predictions for $q_b$ are sensitive to the way the soil behaviour is idealized and the various parameters are selected. However, they note that the above equation is equally valid for predicting cone penetration resistance ($q_c$), implying that, when cone penetration tests have been performed, $q_b$ can be taken as equal to $q_c$. It is assumed that there is no scale effect for end-bearing.

Randolph et al. used a database of 21 load tests, performed at 10 sites, to assess their new method. They found the mean ratio of calculated to measured shaft capacity ($Q/Q_m$) was 0.76, with a standard deviation of 0.41. One feature was that the method tends to over-predict base capacity in compression tests. These two opposing trends led to an average ratio ($Q/Q_m$) for total capacity of 0.99, with a standard deviation of only 0.16. Difficulties in allowing for post-driving residual stresses may have contributed to the apparent bias between base and shaft resistances.
5.4 FUTURE NEEDS

A recent review by Clare (1994) identified a number of research needs relating
to offshore piles in sand. In addition to those which the Dunkirk and
EURIPIDES projects are intended to help meet, the following issues are
considered to have a high priority:

- Effects of soil layering on shaft resistance and bearing capacity
- Effect of sand compressibility
- Behaviour under cyclic loading
- Behaviour in mixed soils
- Lateral loading
- Pile group effects
- Improvements in numerical analysis

Physical tests involving both model and full-scale piles were seen as playing
an important role in future research.

The recent trend towards light-weight jacket structures for North Sea fields, in
which pile loads resulting from environmental loading have become more
significant in comparison to the dead weight components, suggests that more
attention needs to be given to the effects of cyclic loading. In general sands can
be expected to be more susceptible than clays to sustained high level cyclic
loading (Geotechnical Consulting Group, 1994).

5.5 REFERENCES

Recommended practice for planning, designing and constructing fixed offshore
platforms.
Washington, DC.

BOLTON M.D. (1986).
The strength and dilatancy of sands.
Géotechnique, 36(1), 65-78.

Behaviour of pile plug in sandy soils during and after driving.

Cavity expansion in cohesive frictional soils.
Géotechnique, 36(3), 349-353.
*Field measurements of stress interactions between displacement piles in sand.*  
*Ground Engineering,* **28**(July/August), 36-40.

*Investigations into the behaviour of displacement piles for offshore foundations.*  
PhD thesis, University of London

*Research into the behaviour of displacement piles for offshore foundations.*  

*The effects of time on the capacity of pipe-piles in dense marine sand.*  

*Future research needs.*  

*Cavity expansion in sands under drained loading conditions.*  
Int. J. for Num. and Analytical Methods in Geomechanics, **16**(1), 3-23.

*Finite element analysis of laterally loaded piles.*  

GEOTECHNICAL CONSULTING GROUP LTD (1994).  
*Review of offshore pile design for cyclic loading: North Sea clays.*  
Report for Health and Safety Executive.

*Load tests on friction piles in loose sand.*  

*Evidence for scale effects in the end bearing capacity of open-ended piles in sand.*  
*Full displacement pressuremeter testing in sands.*  

*New pile design methods for offshore piles.*  
MTD Ltd, London.

*Axial capacity of an offshore pile driven in dense sand.*  

*Hutton tension leg platform foundations: prediction of driven pile behaviour.*  
Géotechnique, 38(2), 231-252.

*Magnus foundations: soil properties and predictions of field behaviour.*  

*Shaft capacity of driven piles in sand: a new design approach.*  
Proc. 7th Int. Conf. on Behaviour of Offshore Structures, Boston, 1, 23-36.

*Mechanisms of shaft friction in sand from instrumented pile tests.*  

*The skin friction of driven piles in sand.*  
MSc dissertation, Univ. of London (Imperial College).

*Axial capacity of driven piles in sand.*  
Report GEO:92110 to Amoco Production Company, Univ. of Western Australia, Dept Civ. Engng.

*Design of driven piles in sand.*  
Géotechnique, 44(3), 427-448.

*One-dimensional analysis of soil plugs in pipe piles.*  
Géotechnique 41(4), 587-598.
An appraisal of API RP2A recommendations for determining skin friction of piles in sand.

Tests on instrumented piles, Ogeechee River site.

Finite cavity expansion in dilatant soils: loading analysis.
Géotechnique, 41(2), 173-183.

EURIPIDES: tests on large scale driven piles in dense silica sands.
6. DESIGN PARAMETERS FOR NORTH SEA SANDS

6.1 PARAMETERS RELEVANT TO THE DESIGN OF PILES IN SAND

The description of the fundamental behaviour of driven piles in sand presented in Chapter 3 demonstrates that pile capacity is determined by the properties of the sand foundation and the pile, the method of pile installation, and the form of loading. This chapter begins by listing the relevant sand properties, pile installation details, and details of loading and then describes how the properties of the sands may be obtained. Typical values of parameters for North Sea sands are given, where available.

6.1.1 Pile parameters

The following characteristics of a pile, its installation details, and loading are relevant to its capacity:

Pile characteristics
- Geometry (penetration ratio, diameter, wall thickness)
- Tip detail (open-or closed-ended; driving shoe)
- Surface roughness and hardness

Installation details
- Rate of penetration, which (in conjunction with soil permeability) determines whether or not excess pore pressures remain after driving.
- Continuity of penetration, rest periods, ease of driving, jack stroke length (in model studies), etc.
- Mode of penetration, i.e. whether or not a plug forms in an open-ended pile.

Loading
- Whether tension or compression
- Whether static or cyclic
- Whether vertical or combined with significant horizontal components of load
- Delay before loading

6.1.2 Sand properties

The following characteristics and properties of the foundation affect the capacity and load-displacement behaviour of driven piles in sand:
- Soil stratigraphy
• Soil grading, in particular the silt and clay content of the sand
• Grain shape and crushability (which are functions of mineralogy)
• Fabric
• In situ density* (or void ratio)
• In situ stress state
• Stress history or overconsolidation ratio (OCR)
• Compressibility and shear and bulk stiffness characteristics
• Angle of shearing resistance at peak and constant volume
• Angle of interface friction at peak and constant volume
• Angle of dilation
• Permeability

Few of these parameters are taken into account explicitly in current design methods (see Chapter 4). However, they may be required for the new methods of analysis described in Chapter 5 and they may prove valuable in re-evaluations of pile capacity made during the life of a structure.

6.1.3 Obtaining the properties of offshore sands

Undisturbed sampling of sands for laboratory tests is extremely difficult and requires techniques (such as freezing) which are often not cost effective and are unlikely to be feasible offshore. As a result, design parameters for sands are usually determined from a combination of in situ and laboratory tests on reconstituted material. In situ tests are interpreted to provide information on the relative density and in situ stress state to which laboratory samples must be reconstituted.

In situ tests may be used directly in design or they may be interpreted on the basis of theory or empiricism. Theoretical solutions are available for in situ tests in which the surrounding soil elements undergo relatively similar and simple stress paths, e.g. the self boring pressuremeter test or seismic tests. Empirical correlations are necessary to interpret in situ tests in which a wide range of complex stress (or strain) paths are involved and the measured response is the sum of the response to these different stress (or strain) paths, e.g. the cone penetration test.

For cohesionless materials, empirical correlations between penetration resistance and design parameters have been developed largely on the basis of calibration chamber tests and the shortcomings of these need to be considered.

In situ tests used offshore are outlined in Appendix B. Detailed reviews of in situ testing are given by Lumne et al. (1989) and Lumne and Powell (1992). Relevant laboratory tests for obtaining the properties of offshore sands are also outlined in Appendix B.

*Usually expressed in terms of relative density, in which case it is necessary to determine values of maximum and minimum density
6.2 STRATIGRAPHY

Typical stratigraphies in the UK sector of the North Sea and the distribution of sands in general terms are described in Chapter 3. The detailed stratigraphy at a particular site is established using borings and in situ and laboratory tests, with in situ tests being used to interpolate between boreholes after their site-specific calibration. The piezocone is the most widely used tool for profiling at a site. However, thin layers (< 20mm thick) may not be detected with this device.

Various charts have been published that relate soil type to cone resistance and sleeve friction. An example is presented in Figure 51.* For classification purposes, penetration pore pressure is usually normalized in some way, for example as:

\[ B_q = \frac{\Delta u}{q_c - \sigma_{vo}} \]

where \( B_q \) = the (normalized) pore pressure parameter; \( \Delta u \) = excess pore pressure; \( q_c \) = cone resistance corrected for pore pressure effects; and \( \sigma_{vo} \) = total overburden pressure. A classification chart based on cone resistance and pore pressure ratio is shown in Figure 51(b).

6.3 GRADING, GRAIN SHAPE, AND CRUSHABILITY

6.3.1 Grading

Sands in the North Sea are generally described as silty fine (or fine to medium) sands. They frequently contain thin clay layers and shell fragments and occasionally contain gravel size rock fragments, the origin of which varies with location of the site. At some locations, the sands may contain a significant dispersed clay content which has an important bearing on their engineering behaviour; the Gullfaks C site (Tjelta et al., 1988) provides an example.

*The areas marked on Figure 51 correspond to the following soils: 1 sensitive fine-grained; 2 organic soils; 3 clay to silty clay; 4 clayey silt to silty clay; 5 silty sand to sandy silt; 6 clean sand to silty sand; 7 gravelly sand to sand; 8 very stiff sand to clayey sand (heavily overconsolidated or cemented); 9 very stiff, fine grained (heavily overconsolidated or cemented)
Figure 51
Classification charts based on CPT measurements of
(left) \( q_s \) and \( f_s \) (right) \( q_s \) and \( B_q \) (Robertson, 1990)

\[
\frac{q_s}{q_s^0 - f_s^0} \quad \text{NORMALIZED CONE RESISTANCE}
\]
An exercise has been carried out to establish grading envelopes for sands in the North Sea and the results are presented in Figure 52. Sands in the Northern North Sea tend to be finer and more consistent than sands in the Southern North Sea.

Figure 52
Grading envelopes for sands: (top) Southern North Sea; (bottom) Northern North Sea

The North Sea sands (D_{50} = 0.05-0.25mm) are consistently finer than those used in calibration chamber tests (D_{50} = 0.2-0.6mm).
In the absence of gradings, estimates of grain size may be made on the basis of the correlation presented in Figure 53.

![Figure 53](image)

**Figure 53**
Approximate grain size from cone resistance and sleeve friction
(Robertson and Campanella, 1983)

### 6.3.2 Mineralogy, angularity, and crushability

The mineralogy of sand grains determines the sand's hardness, or crushability, and has a strong influence on grain shape and compressibility.

Investigations of mineralogy are not routinely carried out and there is a lack of published data for the UK sector of the North Sea. (Some data for sands from the Norwegian sector is summarized in Table 8.) The sands are generally regarded as being quartzitic and are, therefore, relatively hard. Quartz grains tend to be more rounded than feldspar grains and North Sea sands are sub-rounded to rounded.

Sands of different mineralogy may occur in offshore locations: examples in the vicinity of the UK sector of the North Sea include the carbonate sands at Dunkerque and the glauconitic sands at Zebrugge.
Table 8
Properties of sands in Norwegian sector of North Sea

<table>
<thead>
<tr>
<th>Site</th>
<th>Uniformity</th>
<th>Mineralogy</th>
<th>Angularity</th>
<th>$e_{\text{max}}$</th>
<th>$e_{\text{min}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sleipner A4</td>
<td>0.14/0.09</td>
<td>92% quartz</td>
<td>Sub-rounded</td>
<td>1.00</td>
<td>0.54</td>
</tr>
<tr>
<td></td>
<td>= 1.56</td>
<td>8% feldspar</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gullfaks B</td>
<td>0.25/0.16</td>
<td>Mainly quartz</td>
<td>Sub-rounded</td>
<td>0.88</td>
<td>0.47</td>
</tr>
<tr>
<td></td>
<td>= 1.56</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gullfaks C</td>
<td>0.05/0.006</td>
<td>Mainly quartz</td>
<td>Sub-rounded</td>
<td>1.38</td>
<td>0.32</td>
</tr>
<tr>
<td></td>
<td>= 14.2</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Oseberg South</td>
<td>0.15/0.10</td>
<td>Mainly quartz</td>
<td>Sub-rounded</td>
<td>1.04</td>
<td>0.58</td>
</tr>
<tr>
<td></td>
<td>= 1.50</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Beryl A</td>
<td>0.11/0.07</td>
<td>75% quartz</td>
<td>Sub-rounded</td>
<td>1.02</td>
<td>0.53</td>
</tr>
<tr>
<td></td>
<td>= 1.57</td>
<td>15% feldspar</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>5% mica</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Data from Lunne (1991)

6.4 IN SITU DENSITY, STATE, AND STRESS HISTORY

6.4.1 Relative density

Tube sampling of sands results in a change in their density: loose sands tend to increase in density and dense sands to reduce. As a result, reliance must be placed on in situ tests to determine density. Absolute density may be determined using a nuclear density probe or electrical resistivity probe. It is, however, usual practice to estimate in situ relative density through calibrations with penetration resistance.

Lunne and Christoffersen (1983) proposed the correlation between relative density ($D_r$), cone resistance ($q_c$), and vertical effective stress ($\sigma'_v$) shown in Figure 54, for normally consolidated, pluviated, unaged and uncemented, clean, fine to medium, uniform, quartz (i.e. non-crushable) sands. Alternative correlations for normally consolidated sands have been presented by Schnertmann (1978) and Jamiołkowski et al. (1985). Use of these correlations for overconsolidated sands leads to an overestimate of relative density.
Figure 54
Correlation between $D_r$, $q_c$, and $\sigma'_c$ for normally consolidated sands
(Lunne & Christoffersen, 1983)

Baldi et al. (1986) proposed the correlation between relative density, cone resistance, and mean effective stress ($\sigma_m'$) shown in Figure 55, for both normally consolidated and overconsolidated sands. Use of this correlation requires an estimate to be made of horizontal effective stress ($\sigma'_h$) in order to obtain the mean effective stress ($\sigma'_m$) from:

$$\sigma'_m = \left( \frac{\sigma'_v + 2\sigma'_h}{3} \right)$$

$\sigma'_c$ may be estimated on the basis of dilatometer tests or through estimates of overconsolidation ratio (OCR), together with the correlation:

$$K_o^{oc} = K_o^{nc} \cdot OCR^m$$

where $K_o = \sigma'_c/\sigma'_v$ and $m$ is an index which takes the value:

$$m = 0.27 + 0.26 \cdot D_r$$

These correlations are based predominantly on the results of cone tests in calibration chambers and their application to the field needs to take account of the following:
Figure 55
Correlation between $D_r$, $q_c$, and $\sigma'_n$ (Baldi et al., 1986)

- Sand fabric in the field is likely to be different to that in the chamber because of differences in depositional mode and environment.
- Sands in situ are aged, whereas those in the calibration chamber are unaged. Ageing leads to an increase in stiffness and strength and, therefore, to an increase in cone resistance. As a result, use of these correlations could lead to an overestimate of relative density in aged sands.
- All calibration chamber results for dense sand (relative density > 80%) are affected by boundary conditions. For loose sands, calibration chamber results are thought to be independent of boundary conditions.
- For simplicity of handling, calibration chamber tests have been run on clean sands of medium grain size or larger. These sands are not representative of the silty and clayey fine sands encountered in the North Sea and the influence of grading is largely unknown.
- Sands in calibration chambers have not experienced the wave-
compaction stress-history of offshore sands.

- Sands used in calibration chambers tend to be more angular than those in the North Sea.
- The quartz sands used in calibration chambers are resistant to crushing. In more crushable and compressible sands, relative density may be underestimated on the basis of the calibration chamber results.

The correlations proposed by Lunne and Christoffersen (1983) — see Figure 54 — include a correction to cone resistance based on sample size and boundary effects.

Apart from the thin cover sands, which may be loose, the silty fine sands in the North Sea are generally medium dense to dense. At the Ekofisk site the sands are quoted as having a relative density of 100%. This high density is attributed to current and wave action (e.g. Bjerrum, 1973).

6.4.2 State parameter

The state parameter (Ψ) combines the influence of void ratio and stress level on behaviour, by relating them to an ultimate steady state; Ψ represents the difference in void ratio of soil at a given mean effective stress (p') and the void ratio on the steady state line at the same p' (see Figure 56). The state parameter has been correlated with angle of shearing resistance φ' and angle of dilation, as described in Sections 6.6.1 and 6.6.3.

![Figure 56](image)

Definition of state parameter (Been and Jefferies, 1985)
Been et al. (1986 and 1987) have proposed a method for determining the state parameter ($\psi$) from cone penetration tests, based on a study of drained cone penetration tests in calibration chambers. The relationships between $\psi$ and normalized penetration resistance ($= [q_c - p_o]/p'_o$, where $p_o$ and $p'_o$ are the mean total and effective stresses, respectively) are shown in Figure 57 for a range of sands.

![Figure 57](image)

**Figure 57**

Relationship between normalized cone penetration resistance and state parameter (Been et al., 1987)

The relationship for different sands has been expressed in terms of the slope of the line $m$ and its intercept $k$ with the $\psi = 0$ line. Both $m$ and $k$ have been related to the slope of the steady state line* ($\lambda_{ss}$), as shown in Figure 58. The value of $\psi$ therefore requires knowledge of the in situ stresses (to determine $p_o$ and $p'_o$) and $\lambda_{ss}$ (to determine $m$ and $k$). $\lambda_{ss}$ is determined from triaxial tests on the loose sand.

*Slope of steady state line $= \Delta e / \Delta (\log_{10} p')$
6.4.3 In situ horizontal stress and stress history

Both in situ horizontal total stress ($\sigma_{ho}$) and stress history are extremely difficult to determine in sand deposits. Some success is claimed in establishing $\sigma_{ho}$ from lift-off pressures in SBPM tests in sand (Buzzi et al., 1986), but extreme care is required during installation. A correlation has been proposed by Baldi et al. (1986) using the cone resistance ($q_c$) from CPTU tests and the horizontal stress index ($K_D$) from dilatometer tests:

$$K_D = 0.376 + 0.095 \frac{q_c}{\sigma_{vo}'} - 0.0046$$

Estimates of OCR in sand deposits may be made on the basis of the OCR of included clay layers, as interpreted from oedometer, CPTU, or dilatometer data. In the North Sea, this approach indicates OCRs of 1.5 to 2.5 in sand layers below 20m.

The difficulties with establishing $\sigma_{ho}$ and OCR reduce the reliability of correlations between in situ test data and design parameters which rely on these parameters.
6.5 DEFORMATION PARAMETERS

6.5.1 Shear stiffness characteristics

Prediction of the load-displacement behaviour of a vertically loaded pile in sand requires knowledge of the sand's stress-strain characteristics. Shaft load-displacement behaviour is likely to be determined by shear stiffness characteristic whereas base load-displacement behaviour will be influenced by both shear and bulk stiffness characteristics. These characteristics are determined by: grading; mineralogy; angularity; fabric; relative density stress level; strain level; stress path; stress (or strain) history; age and cementation; drainage conditions; and strain rate.

The influence of fabric, in this case due to different methods of reconstituting the sample of sand, on the overall stress-strain behaviour is shown in Figure 59, for the case of triaxial compression.

![Figure 59: Effect of method of reconstituting sand on its behaviour in drained triaxial compression (Mitchell et al., 1976)](image)

Figure 60 illustrates how the secant shear stiffness (G) of a clean reconstituted sub-angular quartz sand varies with mean effective stress (p'), shear strain level (γ), void ratio (e), and stress history (OCR) based on static torsional shear data for Ham River Sand (Porovic and Jardine, 1994). As discussed by Tatsuoka and Shibuya (1991), Jardine (1995), and others, the shear stiffness reduces with strain over the range of interest (typically 0.001% < γ < 10%) and increases with mean effective stress. The dependence on stress level varies with strain:
at very small strains, $G$ is proportional to the square root of $p'$, whereas at large shear strains it is proportional to $p'$.

![Graph showing composite shear stiffness vs strain curves for samples of Ham River Sand](image)

**Figure 60**

Composite shear stiffness vs strain curves for samples of Ham River Sand: (a) OCR = 1; (b) OCR = 3 (Porovic and Jardine, 1994)

The dependency of shear stiffness on relative density or void ratio has led to various expressions which link laboratory $G_{max}$ to void ratio ($e$). One example is the normalising function $f(e)$ proposed by Hardin and Richart (1963) for clean granular materials tested under similar conditions:

$$f(e) = \frac{(2.17 - e)^2}{1 + e}$$
Jamiolkowski et al. (1991) suggest another expression that is applicable to a wider range of geomaterials:

\[ f'(e) = \frac{1}{e^{\frac{1}{3}}} \]

Porovic and Jardine (1994) suggest that the dependence on void ratio may diminish at larger strains.

Sands may exhibit higher stiffness in their natural, intact states than when reconstituted, because of ageing, cementing, and other processes. However, the disturbance caused by installation may make these components of stiffness unavailable in the soil surrounding recently driven offshore piles.

6.5.2 Shear stiffness from in situ tests

The shear modulus at small strains (\(G_{\max}\)) can be derived from measurements of shear wave velocity (\(v_s\)) made in situ using the seismic cone (see Appendix B). To obtain a complete description of shear stiffness characteristics, \(G_{\max}\) may be combined with information on modulus decay, from laboratory tests on reconstituted soil, or from empirical rules (Hardin and Drnevich, 1972).

Deformation parameters may also be derived from in situ tests using:

- Empirical correlations between laboratory measurements on reconstituted specimens and the results of a penetration test on soil in the same state in a calibration chamber.
- Correlations between observed settlements in the field and the results of adjacent penetration tests; this provides an average deformation modulus.
- Theoretical interpretations of, for example, the pressuremeter test.

Difficulties arise when using penetration tests because penetration resistance is controlled largely by void ratio (or relative density) and effective stress state. The large strains involved in the test tend to remove the effects of stress and strain history which, however, influence stiffness characteristics.

Figure 61 illustrates the high sensitivity of deformation modulus (\(E'_s\)) and the low sensitivity of penetration resistance (\(q_c\)) to stress history, particularly at low relative density (i.e. low \(q_s/\sigma_{vn}'\)). It follows that no unique correlation exists between \(q_c\) and \(E'_s\) and that a knowledge of stress history is essential for interpretation. Robertson and Campanella (1983) suggest that \(E_s/\sigma_c\) varies between 1.5 and 3.0 for normally consolidated sand, where \(E_s\) is the secant modulus at 25% of the failure stress.

*\(E'_s\) is the drained secant modulus at an axial strain of 0.1% in triaxial compression
Figure 61  
Correlation between normalized secant Young's modulus and normalized cone penetration resistance (Baldi et al., 1989)

In contrast, the dynamic shear modulus ($G_{\text{max}}$) is determined largely by relative density and effective stress state, and does not appear to be strongly influenced by stress or strain history. Correlations of dynamic shear modulus with penetration resistance, such as that shown in Figure 62, are therefore reasonably reliable and can be used in the absence of shear wave measurements to determine $G_{\text{max}}$. 

120
Figure 62
Correlation between normalized dynamic shear modulus and normalized cone penetration resistance (Baldi et al., 1989)

In pressuremeter tests, deformation modulus is usually determined from load-unload loops. It is important that the modulus that is derived is related to the current stress level and cavity strain.

6.5.3 Shear stiffness from laboratory tests
Stress-strain characteristics of sands may be determined from laboratory tests on reconstituted samples which have been brought to their in situ state, in terms of density and fabric, and which have been reconsolidated following their recent stress history. (It is unlikely that reliable stiffness properties can be measured on intact samples, except when they are taken using freezing techniques.)

Measurements of $G_{\text{max}}$ may be made indirectly using the resonant column apparatus or bender elements (see Appendix B). Direct measurements of $G_{\text{max}}$ and of the full modulus decay curve may be made in either monotonic or cyclic loading tests using the torsional shear apparatus, or triaxial apparatus, both involving measuring systems which avoid errors due to bedding and compliance (see Appendix B).

6.5.4 Compressibility/crushability
The crushing of sand grains can make a significant contribution to the compressibility of a sand. Crushing can occur beneath the pile tip during installation and loading. Compressibility has an important influence on both shaft capacity (see Chapter 3) and end bearing capacity.
Crushing increases with grain size, uniformity, angularity, confining stress, deviator stress level. It is of major importance in carbonate sands, for which crushing index ($C_c$) has been defined by Datta et al. (1979) as:

$$C_c = \frac{\% \text{ passing } D_{10} \text{ after application of shear}}{10\%}$$

In connection with piles in sand, the compressibility of sand or limiting compressibility index ($C_{nl}$) has been taken as the slope of the void ratio vs log$_e$ p' curve, for p' of 800 kPa in an isotropic triaxial compression test.

### 6.6 FRICTIONAL RESISTANCE

#### 6.6.1 Angle of shearing resistance

The key factors determining the response of a soil to shear are:
- Drainage conditions (undrained, drained, or partially drained)
- Effective stress level
- Stress history
  - Shear direction (in terms of the direction of major principal stress to the vertical)
  - Relative magnitude of the intermediate principal stress
  - Initial soil state (i.e. relative density; fabric/state of packing; bonding; degree of saturation)
  - Constraints on shear zone formation/bifurcation

The response will vary with soil type, particularly at large displacements when turbulent shear in granular and low plasticity clays will lead to shear at critical state or constant volume ($\phi_{c}$), whereas sliding shear in medium to high plasticity clays will lead to residual conditions ($\phi_{r}$). Table 9 contrasts the factors that control $\phi$ at peak ($\phi_{p}$) and at constant volume ($\phi_{c}$) in a granular soil.

Angles of shearing resistance ($\phi$) for offshore sands may be derived by following one of several routes:
- Relative density ($D_r$) is estimated from information on cone resistance, vertical effective stress, and overconsolidation ratio or horizontal effective stress — see Section 6.4.1 — and then $\phi$ is either derived from relationships between $\phi$ and $D_r$ (such as that shown in Figure 63) or is measured in triaxial tests on samples reconstituted to the estimated in situ $D_r$, or derived from equations given by Bolton (1986).
Table 9
Factors controlling friction angles of granular soils

<table>
<thead>
<tr>
<th>Factor</th>
<th>Friction angle</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Peak</td>
</tr>
<tr>
<td><strong>Material characteristics</strong></td>
<td></td>
</tr>
<tr>
<td>Mineralogy</td>
<td>✓</td>
</tr>
<tr>
<td>Angularity</td>
<td>✓</td>
</tr>
<tr>
<td>Grading</td>
<td>✓</td>
</tr>
<tr>
<td><strong>Packing</strong></td>
<td></td>
</tr>
<tr>
<td>Density (i.e. void ratio and relative density)</td>
<td>✓</td>
</tr>
<tr>
<td>Fabric</td>
<td></td>
</tr>
<tr>
<td><strong>Stress state</strong></td>
<td></td>
</tr>
<tr>
<td>Mean effective stress</td>
<td>✓</td>
</tr>
<tr>
<td>Relative magnitude of intermediate principal stress</td>
<td>✓</td>
</tr>
<tr>
<td>Direction of shear</td>
<td>✓</td>
</tr>
<tr>
<td>Stress history</td>
<td></td>
</tr>
<tr>
<td><strong>Constraints</strong></td>
<td></td>
</tr>
<tr>
<td>Freedom to bifurcate</td>
<td></td>
</tr>
</tbody>
</table>

- $\phi$ is determined directly from measurements of cone resistance by assuming a bearing capacity theory and bearing capacity factor based on rigid plastic soil models. An example is shown in Figure 64.
- $\phi$ is derived from cone data using cavity expansion theory, which takes into account the compressibility of the sand by assuming elastic/perfectly-plastic behaviour.
- $\phi$ is derived from relationships with the state parameter $\psi$ (see Figure 65); this correlation applies to the peak value of $\phi$ in triaxial compression.
- $\phi$ is derived from SBPM tests (e.g. Hughes et al., 1977; Fahey and Randolph, 1984) or dilatometer tests (e.g. Marchetti, 1980).
Figure 63
Correlation between friction angle and relative density (after Schmertmann, 1978)

The interpretation needs to take account of the stress level dependence of $\phi$ (i.e., the curved failure envelope). The magnitude of any interpreted value (which is a secant value) is determined by the magnitude of the normal stress on the failure plane. Values for $\phi$ selected for design should take account of the presence of any clay inclusions.

Laboratory tests on sands from the Norwegian sector of the North Sea, which have been reconstituted to in situ relative densities, have been reported by Kjekstad and Lunne (1979) — see Figure 66. These angles of friction ($\phi$) are typical peak friction angles for quartz sand in triaxial compression.
Figure 64
Bearing capacity factors used to determine friction angle from CPTs
(Linne and Christoffersen, 1983)

Figure 65
Relationship between $\psi$ and friction angle (Been et al., 1987)
Figure 66
Angles of friction for North Sea sands in drained triaxial compression
(Kjekstad and Lunne, 1979)

6.6.2 Angle of interface friction
The behaviour of sand when sheared against steel interfaces is illustrated in Figure 67. The shear stress ratio ($\tau/\sigma$) increases with displacement until there is a well-defined yield point which increases with interface roughness.*

Dilation only begins when the yield point is reached and does not, in these experiments, occur for $R_{\text{max}} < 9.8\mu m$ (dilation does not occur in loose sands or silts, either). Up to yield, there is shear deformation within the sand, with little or no slip at the interface (except in loose sands); beyond yield, slip at the interface dominates.

Against a smooth interface, there is only slip at the interface beyond yield; against a rough interface, there is slip and particle rolling within a shear zone close to the interface. The thickness of this zone increases with interface roughness. For dense sand sheared against a rough interface, there are well-defined peak ($\tan \delta_{\text{max}}$) and residual ($\tan \delta_{\text{cv}}$) shear stress ratios; dense sands

*In these experiments, the roughness ($R_{\text{max}}$) was defined as the relative height between the highest peak and the lowest trough along a surface profile of length ($L$) of 0.2mm — see Figure 69

126
yield before they reach peak. Dilation ceases when the residual shear stress ratio (tan $\delta_n$) is reached; this requires only 2 to 4mm displacement.

![Graph showing shear stress ratio and volumetric strain](image)

**Figure 67**

*Behaviour of sand sheared against steel interfaces (after Uesugi and Kishida, 1986a)*

The value of $\delta_{\text{max}}$ varies with interface roughness and soil type (grading, grain size, hardness, and shape), as illustrated in Figure 68(a). The influence of relative density on $\delta_{\text{max}}$ is illustrated in Figure 68(b), using the results of shear box and ring shear interface tests on Labenne sand.* Increasing stress level suppresses dilation and reduces $\delta_{\text{max}}$.

The effect of particle size on $\delta_{\text{max}}$ can be eliminated for given grain shape, material type, and interface material by introducing normalized roughness ($R_n$) where:

$$R_n = \frac{R_{\text{max}}}{D_{50}}$$

where $D_{50}$ is the sand's median particle size.

*In these experiments, the centre-line average roughness ($R_{\text{CLA}}$) was measured and reported instead of maximum roughness ($R_{\text{max}}$)
Figure 68
Effect on interface friction of (top) surface roughness (Uesugi & Kishida, 1986a) and (bottom) initial void ratio (Jardine et al., 1992)
The relationship given above is illustrated in Figure 70, where it can be seen that $\delta_{\text{max}}$ tends towards $\phi_{\text{max}}$ for $\Re > 0.1$. It follows from this figure that, for a given pile roughness, $\delta$ will reduce as the mean particle size increases. (Note that, for offshore piles, typical values of $R_{\text{max}}$ for $L = 0.2\text{mm}$ are 10 to 20$\mu$m and $R_{\text{CLA}}$ lies between 5 and 10$\mu$m).

The LaBenne field tests showed that it is the constant volume interface friction angle ($\delta_{cv}$), and not the maximum angle ($\delta_{\text{max}}$), that controls shaft capacity. The interface tests shown in Figure 68(b) indicate that $\delta_{cv}$ is independent of relative density and $\delta_{cv}$ for LaBenne sand is approximately 28° for shear against steel interfaces of similar roughness to the stress transducers on the Imperial College pile. Figure 71(b) illustrates the variation of $\delta_{cv}$ with $D_{50}$ for a centre-line average roughness ($R_{\text{CLA}}$) of 10$\mu$m; and Figure 71(a) illustrates the variation of $\delta_{cv}$ with $R_{\text{CLA}}/D_{10}$.

Because of its relevance to pile capacity, $\delta_{cv}$ should be measured in interface shear tests (see Appendix B).

No systematic study of North Sea sands sheared against steel with roughnesses typical of offshore piles has been carried out. Guidance on likely values can be found in Jardine et al. (1992).
6.6.3 Angle of dilation

The dilation that occurs within the body of the sand and at the pile-shaft/sand interface has an influence on the load-displacement behaviour of a pile in sand. Its influence on end bearing capacity can be taken into account in more recent cavity expansion theories. Its effect on shaft friction and the scale effect that it introduces have been discussed in Chapter 3.

Angle of dilation \((\sin^{-1} \frac{d\epsilon_a}{d\epsilon_e})\) or dilation rate \((d\epsilon_a/d\epsilon_e)\) can be measured in drained triaxial, shear box, or torsional shear tests on sand and in direct shear interface tests. A correlation between dilation rate in triaxial compression and state parameter has been proposed by Been et al. (1986) and is shown in Figure 72. Equations linking dilation angle to the sand's angle of friction, mean effective stress, and relative density are given by Bolton (1986).
Figure 71

Variation of $\delta_v$ with (top) $R_{CLA}/D_{50}$ and (btm) $D_{50}$ (Jardine et al., 1992)
6.7 PERMEABILITY

Values of permeability may be estimated from grading curves or, where the clay or silt content is high enough to cause piezocene penetration to be undrained, determined from the dissipation stage of such tests. The BAT Ground Water Monitoring System (Torstensson, 1984) has also been developed for offshore use (Rad, 1988) for the measurement of in situ permeability and the detection of gas.

6.8 REFERENCES


A state parameter for sands.
Géotechnique, 35(2), 99-112.

The cone penetration test in sands: part I, state parameter interpretation.
Géotechnique, 36(2), 239-249.

The cone penetration test in sands: part II, general inference of state.
Géotechnique, 37(3), 285-299.

Geotechnical problems involved in foundations of structures in the North Sea.
Géotechnique 3(3), 319-358.

BOLTON M.D. (1986).
The strength and dilatancy of sands.
Géotechnique, 36(1), 65-78.

Self-boring pressuremeter in Po River sand.

Crushing of calcareous sand during shear.

Effect of disturbance on parameters derived from self-boring pressuremeter tests in sand.
Géotechnique 34(1), 81-97.

HARDIN B.O. and DRNEVICH V.P. (1972).
Shear modulus and damping in soil: design equations and curves.

Elastic wave velocities in granular soils.
*Pressuremeter tests in sands.*  
Géotechnique, 27(4), 455-477.

*Soil parameters used for design of gravity platforms in the North Sea.*  
Proc. 2nd Int. Conf. on Behaviour of Offshore Structures (BOSS '79), London, 1, 175-192.

*New developments in field and laboratory testing of soils.*  

*New developments in field and laboratory testing of soils.*  

*One perspective of the pre-failure deformation characteristics of some geomaterials.*  
Proc. 1st Int. Conf. on Pre-failure Deformation Characteristics of Geomaterials, Sapporo, Japan, 2, 855-885.

*Friction coefficients for piles in sands and silts.*  

*Practical use of CPT correlations in sand based on calibration chamber tests.*  
Proc. 1st Int. Symp. on Calibration Chamber Testing, Clarkson Univ., Potsdam. (Also NGI Internal Report 521550-57.)

*Interpretation of cone penetrometer data for offshore sands.*  

*General report/discussion session 2: SPT, CPT, pressuremeter testing and recent developments in in-situ testing — Part 1: All tests except SPT.*  
*Recent developments in in situ testing in offshore soil investigations.*  

MARCHETTI S. (1980).  
*In situ tests by flat dilatometer.*  

*The influences of sand fabric on liquefaction behaviour.*  

*Some observations on the static and dynamic shear stiffness of Ham river sand.*  
Proc. 1st Int. Conf. on Pre-failure Deformation Characteristics of Geomaterials, Sapporo, Japan, 1, 25-30.

*Offshore BAT: equipment, testing and interpretation.*  
Internal report 52159-1, Norwegian Geotech. Inst., Oslo.

*Soil classification using the cone penetration test.*  

*Interpretation of cone penetration tests. Part I, Sand.*  

*Guidelines for cone penetration test: performance and design.*  

*Deformation characteristics of soils and rocks from field and laboratory tests.*  

*Foundation design for deepwater gravity base structure with long skirts on soft soils.*  
Proc. Int. Conf. on Behaviour of Offshore Structures (BOSS '88), Trondheim, 1, 173-192.
*A new system for groundwater monitoring.*

*Influential factors of friction between steel and dry sands.*

*Frictional resistance at yield between dry sand and mild steel.*
Soils and Foundations, 26(4), 139-149.
7. USING THE METHODS IN PRACTICE

7.1 INTRODUCTION
This chapter presents example calculations of shaft and end-bearing resistance for two sites, one (Site A) at the southern edge of the central North Sea and the other (Site B) in the southern North Sea. The purpose of the calculations is to illustrate the way in which the design methods discussed in Chapters 4 and 5 are used in practice and to highlight differences between the methods, particularly the newer design methods.

Ground conditions at Sites A and B are fairly typical of conditions in the areas of the North Sea where they are located. In both cases, the predominant soil type is sand. The piles considered for Site A were designed to support a gravity platform; whereas the piles considered for Site B were designed to anchor a light-weight offshore structure. The ground and pile details have been taken from consulting work undertaken by GCG during the past five years. A conductor pull-out test has been performed at Site B and serves as a benchmark with which to judge the relative merits of the various design methods.

7.1.1 Ground conditions
Example borehole logs from the two sites are shown on Figures 73 and 74.

The ground conditions at Site A are typical of those encountered in the central North Sea, consisting of 2.5m of loose to medium dense sand and gravel overlying 17.7m of stiff to very stiff slightly sandy clay (see Figure 73). Beneath the clay is an extensive (greater than 60m deep) deposit of medium dense to very dense sand. Traces from one of the piezocone tests performed at the site are presented on Figure 75.

The ground conditions at Site B are typical of many southern North Sea locations, consisting of 5m of fine dense sand and 4.6m of firm clay overlying more than 30m of dense fine sand (see Figure 74). Traces from a typical cone penetration test at the site are presented on Figure 76.
Loose to medium dense SAND and GRAVEL

Stiff to very stiff slightly sandy CLAY
with occasional pockets, bands, and thin layers of sand

Medium dense to very dense SAND
with pockets, bands, and thin layers of slightly sandy clay

Figure 73
Summary borehole log for Site A
Figure 74
Summary borehole log for Site B
Figure 75
Traces from one of the piezocone tests performed at Site A
Figure 76
Traces from a typical cone penetration test performed at Site B
7.1.2 Pile dimensions
The dimensions of the piles at the two sites are given in Table 10.

<table>
<thead>
<tr>
<th>Site</th>
<th>Outside diameter (m)</th>
<th>Wall thickness (mm)</th>
<th>Embedded length (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>1.524</td>
<td>50.8</td>
<td>65.0</td>
</tr>
<tr>
<td>B</td>
<td>0.660</td>
<td>31.75</td>
<td>38.1</td>
</tr>
</tbody>
</table>

7.1.3 Design methods considered
The design methods that have been used to conduct the worked examples are taken from:
- API RP2A
- Toolan, Lings, and Mirza (1990), method 1
- Imperial-College/GCG approach (1994-6)
- Randolph, Dolwin, and Beck (1994)

The joint Imperial-College/GCG approach involves using Lehane and Jardine's (1994) "method 2" to calculate shaft capacity and Jardine and Overy's (1996) method to calculate end-bearing capacity.

Details of these methods are given in Chapters 4 and 5, but are repeated below as necessary to explain the calculations.

7.2 INTERPRETATION OF PARAMETERS

7.2.1 Relative density
A key parameter in all of the design methods listed above is the relative density of the sand. This has been determined for the two sites using Lunne and Christoffersen's formulae (see Chapter 6) based on the measured cone penetration resistance ($q_c$). Many practising engineers would use older correlations between relative density and $q_c$, such as that published by Schmertmann (1978), which are independent of vertical effective stress. The use of Lunne and Christoffersen's formulae leads to more conservative estimates of pile capacity.

Figure 77 shows the variation in relative density ($D_r$) at Site A. The calculated
values of $D_r$ assume that the sand below 20m has an overconsolidation ratio (OCR) reducing with depth from 4.0 to 2.1 (based on the OCRs of the various clay marker bands). As the figure shows, the majority of the sand at Site A is in a very dense state.

![Relative density $D_r$ (%)](image)

**Figure 77**
Relative density inferred for Site A

Figure 78 shows the variation in relative density at Site B. The calculated values of $D_r$ assume that the sand below 10m has an OCR decreasing with depth from 4.2 to 1.8 (based on the removal of 35m of overburden, as interpreted from the regional surface geology). At this site, the sand is medium dense down to a depth of 26m and on the borderline between medium dense and dense below that depth.
7.2.2 Bulk unit weight
The bulk unit weight of the sand at Site A was taken as 18.9 kN/m$^3$ and at Site B as 19.3 kN/m$^3$, based on the available site investigation data.

7.3 API RP2A

7.3.1 Site A
The parameters adopted for the sand stratum at Site A are summarized in Table 11.

<table>
<thead>
<tr>
<th>Sand stratum</th>
<th>Depth (m)</th>
<th>Relative density</th>
<th>Angle of shaft friction $\delta$ ($^\circ$)</th>
<th>Limiting skin friction $\tau_{\text{max}}$ (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>20.0-28.5</td>
<td>Very dense</td>
<td>35.0</td>
<td>115.0</td>
</tr>
<tr>
<td></td>
<td>29.5-65.0</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 78
Relative density inferred for Site B
The shaft capacity of the pile (neglecting the contribution of the clay layer above a depth of 20m) is given in Table 12 for the two variants of API RP2A considered.

<table>
<thead>
<tr>
<th>Method</th>
<th>Loading direction</th>
<th>K</th>
<th>Average shaft friction (kPa)</th>
<th>Shaft capacity* (MN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>API RP2A</td>
<td>Compression or tension</td>
<td>0.8</td>
<td>114</td>
<td>24.0</td>
</tr>
<tr>
<td>UK practice</td>
<td>Compression</td>
<td>0.7</td>
<td>112</td>
<td>23.7</td>
</tr>
<tr>
<td></td>
<td>Tension</td>
<td>0.5</td>
<td>104</td>
<td>21.9</td>
</tr>
</tbody>
</table>

*Neglecting the contribution of the clay layer above a depth of 20m*

The end-bearing capacity of the pile based on its full cross-sectional area is 21.9MN, whereas that based on end-bearing on the pile annulus plus internal skin friction is approximately 25MN (with \( K = 0.7 \)). The first value is therefore critical.

### 7.3.2 Site B

The parameters adopted for the three main sand strata at Site B are summarized in Table 13.

<table>
<thead>
<tr>
<th>Sand stratum</th>
<th>Depth (m)</th>
<th>Relative density</th>
<th>Angle of shaft friction ( \delta ) (°)</th>
<th>Limiting skin friction ( \tau_{\text{max}} ) (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.0-5.0</td>
<td>Medium dense</td>
<td>25.0</td>
<td>81.0</td>
</tr>
<tr>
<td>2</td>
<td>9.6-26.0</td>
<td>Medium dense</td>
<td>25.0</td>
<td>81.0</td>
</tr>
<tr>
<td>3</td>
<td>26.0-38.1</td>
<td>Medium dense to dense</td>
<td>27.5</td>
<td>88.5</td>
</tr>
</tbody>
</table>

The shaft capacity of the pile (neglecting the contribution of the clay layer) is given in Table 14 for the two variants of API RP2A considered.
Table 14
 Shaft capacities for Site B based on API RP2A

<table>
<thead>
<tr>
<th>Method</th>
<th>Loading direction</th>
<th>K</th>
<th>Average shaft friction (kPa)</th>
<th>Shaft capacity* (MN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>API RP2A</td>
<td>Compression or tension</td>
<td>0.8</td>
<td>63.6</td>
<td>4.42</td>
</tr>
<tr>
<td>UK practice</td>
<td>Compression</td>
<td>0.7</td>
<td>60.1</td>
<td>4.18</td>
</tr>
<tr>
<td></td>
<td>Tension</td>
<td>0.5</td>
<td>48.4</td>
<td>3.36</td>
</tr>
</tbody>
</table>

*Neglecting the contribution of the clay layer

The end-bearing capacity of the pile based on its full cross-sectional area is 2.46MN, whereas that based on end-bearing on the pile annulus plus internal skin friction is more than 4MN (with K = 0.7). The first value is therefore critical.

7.4 TOOLAN, LINGS, AND MIRZA (1990)

Toolan et al.'s "method 1" is described in detail in Chapter 5. It assumes that the local skin friction acting on the pile shaft increases linearly from zero at the ground surface to a maximum at the pile tip that depends on the relative density (D_r) of the sand at that level. The maximum value is read from the chart given in Chapter 4.

If relative density varies with depth, then each stratum is analyzed separately and the resulting profiles of skin friction are combined in piecemeal fashion (see Figures 79 and 80 below).

7.4.1 Site A

The variation of relative density (D_r) with depth at Site A is assumed to follow the design line given on Figure 77.

Values of local skin friction at the pile toe for the various sand strata at Site A are given in Table 15. These have been determined from Toolan et al.'s chart, which covers relative densities up to 90%. The values given for strata 1 and 3 are therefore based on a relative density of 90% and not 95%.
Table 15
Parameters adopted for Toolan et al.'s method for Site A

<table>
<thead>
<tr>
<th>Stratum</th>
<th>Depth (m)</th>
<th>Relative density D_r (%)</th>
<th>Skin friction at pile toe ( \tau_{toe} ) (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>20.0-28.5</td>
<td>95</td>
<td>156</td>
</tr>
<tr>
<td>2</td>
<td>29.5-35.0</td>
<td>85</td>
<td>135</td>
</tr>
<tr>
<td>3</td>
<td>35.0-38.0</td>
<td>95</td>
<td>156</td>
</tr>
<tr>
<td>4</td>
<td>38.0-44.0</td>
<td>82.5</td>
<td>115</td>
</tr>
<tr>
<td>5</td>
<td>44.0-65.0</td>
<td>90</td>
<td>156</td>
</tr>
</tbody>
</table>

Figure 79 shows the variation of local skin friction with depth along the pile shaft based on these assumptions.

Figure 79
Skin friction inferred from Toolan et al.'s method for Site A
The shaft capacity of the pile (neglecting the contribution of the clay layers) is 20.6MN both in compression and in tension. The end-bearing capacity is assumed to be the same as that calculated using API RP2A.

7.4.2 Site B
The variation of relative density \(D_r\) with depth at Site B is assumed to follow the design line given on Figure 78.

Using Toolan et al.'s chart, the local skin friction at the pile tip for a sand with \(D_r = 52.5\%\) is 44kPa and for \(D_r = 62.5\%\) it is 61kPa. Based on these values, Figure 80 shows the variation of local skin friction with depth along the pile shaft according to Toolan et al.

![Figure 80](image)

**Figure 80**
Skin friction inferred from Toolan et al.'s method for Site B

The shaft capacity of the pile (neglecting the contribution of the clay layer) is 2.02MN both in compression and in tension. The end-bearing capacity is assumed to be the same as that calculated using API RP2A.

7.5 IMPERIAL-COLLEGE/GCG APPROACH
The Imperial-College/GCG approach involves using Lehane and Jardine's "method 2" (see Chapter 5) to estimate shaft capacity and Jardine and Overy's method to estimate end-bearing resistance (also see Chapter 5).
Lehane and Jardine's method calculates the shaft friction acting along the pile from the equation:

$$\tau_f = \sigma'_f \tan \delta$$

where, for compression loading:

$$\sigma'_f = \sigma'_{rc} + \Delta \sigma'_r$$

and for tension loading:

$$\sigma'_f = 0.8 \sigma'_{rc} + \Delta \sigma'_r$$

The $\Delta \sigma'_r$ term in these equations is small for the two sites considered, contributing about 1% to the capacity for Site A and 2% for Site B.

The radial effective stress acting at the end of equalization ($\sigma'_{rc}$) has been determined from the equation:

$$\sigma'_{rc} = 0.0286 \ q_b \ \left( \frac{\sigma'_{vo}}{P_A} \right)^{0.2} \left( \frac{h}{R^*} \right)^{-0.3}$$

where $\sigma'_{vo}$ is the original vertical effective stress in the soil, $P_A$ is standard atmospheric pressure, and $R^*$ is the radius of the pile (corrected for the fact that the pile is open-ended — see Chapter 5 for details). A minimum value of 8 was used for $h/R^*$ and the resulting shaft capacity was reduced by 15%, as recommended by Jardine and Overy (1996).

Following Jardine and Overy (1996), end-bearing has been calculated as the product of the cone resistance at the pile tip, the annular area of the pile, and a shape factor (which has been taken as 1.0). Slightly different results would have been obtained if the latest method described by Jardine and Chow (1996) had been used.

### 7.5.1 Site A

The angle of interface friction for Site A was assumed to be 22° for the vast majority of the sand, which takes account of the high (> 400kPa) predicted radial effective stresses acting along the pile shaft. The value of 22° was selected by reference to Jardine et al. (1992), combined with the limiting value of $\delta$ suggested by Uesugi and Kishida (1986). Reduced values were adopted immediately below any clay layers to allow for interface down-drag. $R^*$ was 0.274m.

The calculated shaft capacities were 56.4MN in compression and 45.3MN in tension. The end-bearing capacity was 9.57MN.
7.5.2 Site B
The angle of interface friction for Site B was assumed to be 24.5°, with reduced values being adopted immediately below any clay layers and where the predicted radial effective was greater than 400kPa (as for Site B). R* was 0.141m.

The calculated shaft capacities were 6.16MN in compression and 4.93MN in tension. The end-bearing capacity was 2.50MN.

7.6 RANDOLPH, DOLWIN, AND BECK (1994)
In Randolph, Dolwin, and Beck's method, the radial effective stress acting at any point along the pile shaft at failure ($\sigma_{th}'$) is calculated from the end-bearing resistance at that depth ($q_b$), using the formula:

$$\sigma_{th}' = a \ q_b \ e^{-b \ \tan \ \phi_v}$$

where $a = 2$, $b = 7$, and $\phi_v$ is the constant volume angle of friction of the sand. A value of $\phi_v = 32^\circ$ has been assumed for Sites A and B, and hence this equation reduces to $\sigma_{th}' = 0.0252 q_b$. For both sites, $q_b$ was taken as the cone penetration resistance. Different results might be obtained if $q_b$ was calculated from cavity expansion theory.

Shaft friction is determined from the equation:

$$\tau_f = K \ \sigma_{vo}' \ \tan \ \delta$$

where $\delta$ is the angle of interface friction of the soil, which Randolph et al. suggest should be taken as $\delta = \phi_v - 5^\circ$ (i.e. 27° for both sites).

In the last equation, the earth pressure coefficient $K$ is given by:

$$K = K_{\min} + (K_{\max} - K_{\min}) \ e^{\frac{q_b}{2 R}}$$

where $\mu$ is 0.05. $K_{\min}$ has been taken as the active earth pressure coefficient ($K_a$), which, for $\phi_v = 32^\circ$, is 0.31. The profiles of $K_{\max}$ differ between the two sites.

Randolph et al.'s method reduces the shaft capacity of tension piles by a factor which depends on the sand's Poisson's ratio, the pile's Young's Modulus, and the angle of interface friction (see Chapter 5 for more details). The value of these parameters were assumed to be 0.15, 30GPa, and 27° respectively.

According to the theory put forward by Randolph, Leong, and Houltsby (1991) — as described in Chapter 5 — a plug should form at the bottom of both piles and hence the end-bearing resistances were calculated as the product of the cone resistance at the pile tip and the total cross-sectional area of the pile. This
approach treats the soil mass beneath the pile as a continuum. As explained in Chapter 3, it is likely that pile diameter will have a strong effect on the available end-bearing resistance. Recent model tests described by Randolph (1996) suggest that \( q_e \) is likely to be far smaller than \( q_b \) for open-ended piles. Future development of the Randolph, Dolwin, and Beck method is likely to modify the \( q_b = q_e \) assumption.

### 7.6.1 Site A

The value of \( K_{max} \) adopted for Site A varied from 6.2 at a depth of 20m to 3.4 at the pile toe. The shear modulus for the sand was taken as 140MPa, giving a correction factor for tension loading of 0.55.

The calculated shaft capacities were 106.9MN in compression and 59.1MN in tension. The end-bearing capacity was 146.3MN.

### 7.6.2 Site B

The value of \( K_{max} \) adopted for Site B varied from 15 near the ground surface to 2.8 at the pile toe. The shear modulus for the sand was taken as 80MPa, giving a correction factor for tension loading of 0.69. The end-bearing capacity was 13.7MN.

The calculated shaft capacities were 11.7MN in compression and 8.0MN in tension.

### 7.7 COMPARISON OF RESULTS

#### 7.7.1 Shaft capacity

Table 16 below summarizes the shaft capacities that the various design methods predict for the two sites, for both compression and tension loading.

Current UK practice and Toolan et al.'s method give more conservative designs than required by the current edition of API RP2A.

The two newer methods suggest that the recommendations of API RP2A may also be conservative. The very high predictions given by Randolph et al.'s method reflects its sensitivity to the basic assumptions concerning the radial effective stress acting on the pile. Reasons why Randolph et al.'s method gives larger capacities than Lehane and Jardine's are the higher angle of interface friction (27° versus 22-24.5°) and the fact that no allowance is made for the open-ended nature of the pile when evaluating radial effective stresses.

It is interesting to note that the observed tension capacity at Site B was close to 5MN (Jardine and Overy, 1996) and matches best the Lehane and Jardine prediction.
Table 16
Comparison of shaft capacities predicted by various design methods

<table>
<thead>
<tr>
<th>Design method</th>
<th>Site A</th>
<th>Site B</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Comp'n (MN)</td>
<td>Tension (MN)</td>
</tr>
<tr>
<td>API RP2A</td>
<td>24.0</td>
<td>24.0</td>
</tr>
<tr>
<td>API RP2A (UK practice)</td>
<td>23.7</td>
<td>21.9</td>
</tr>
<tr>
<td>Toolan et al. (1990)</td>
<td>20.6</td>
<td>20.6</td>
</tr>
<tr>
<td>Imperial-College/GCG</td>
<td>56.4</td>
<td>45.3</td>
</tr>
<tr>
<td>(Lehane &amp; Jardine, 1994)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Randolph et al. (1994)</td>
<td>106.9</td>
<td>59.1</td>
</tr>
</tbody>
</table>

*Neglecting the contribution from any clay layers*

7.7.2 End-bearing capacity

Table 17 below summarizes the end-bearing capacities that the various design methods predict for the two sites.

Table 17
Comparison of end-bearing capacities predicted by various design methods

<table>
<thead>
<tr>
<th>Design method</th>
<th>Site A (MN)</th>
<th>Site B (MN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>API RP2A</td>
<td>21.9</td>
<td>2.46</td>
</tr>
<tr>
<td>Toolan et al. (1990)</td>
<td>Same as API RP2A</td>
<td></td>
</tr>
<tr>
<td>Imperial-College/GCG</td>
<td>9.57</td>
<td>2.50</td>
</tr>
<tr>
<td>(Jardine &amp; Overy, 1996)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Randolph et al. (1994)</td>
<td>146.3</td>
<td>13.7</td>
</tr>
</tbody>
</table>

This end-bearing predicted by Jardine and Overy's method for Site A is less than 50% of that calculated by API RP2A. This reflects the influence of pile size (both diameter and length): full end-bearing capacity is unlikely to be mobilized within acceptable settlements (say 10% of the pile's diameter).

The predictions obtained by Randolph et al.'s method are extreme, which
mainly reflects the assumption that the end-bearing \( (q_e) \) equals the cone resistance at the pile tip \( (q_c) \) and acts over the full cross-sectional area of the pile. Randolph et al.'s method was verified using tests on piles less than 35m in length and 0.9m in diameter. It seems that this assumption does not extrapolate well to much larger piles.

### 7.7.3 Total capacity

Table 18 below summarizes the total capacities that the various design methods predict for the two sites.

<table>
<thead>
<tr>
<th>Design method</th>
<th>Site A</th>
<th>Site B</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Comp'n (MN)</td>
<td>Tension (MN)</td>
</tr>
<tr>
<td>API RP2A</td>
<td>45.9</td>
<td>24.0</td>
</tr>
<tr>
<td>API RP2A (UK practice)</td>
<td>45.6</td>
<td>21.9</td>
</tr>
<tr>
<td>Toolan et al. (1990)</td>
<td>42.5</td>
<td>20.6</td>
</tr>
<tr>
<td>Lehane &amp; Jardine (1994)</td>
<td>66.0</td>
<td>45.3</td>
</tr>
<tr>
<td>Randolph et al. (1994)</td>
<td>253.2</td>
<td>59.1</td>
</tr>
</tbody>
</table>

*Neglecting the contribution from any clay layers*

As this table shows, the various methods give a very wide range of predictions! Reasons for these differences have already been discussed above.

### 7.8 CONCLUSIONS

This chapter has presented two worked examples for typical sand sites in the North Sea. Predictions of shaft and end-bearing capacity have been made using five different methods and have been compared. The methods give a very wide range of calculated capacities.

In considering the relative merits of the various methods, it is first worth noting that, in the light of the improved understanding of pile behaviour described in Chapter 3, it is no longer logical to apply limiting values to local skin friction acting on offshore piles.

The removal of limiting skin friction values has a strong influence on predicted shaft capacities, as does the "h/R effect" described in Chapter 3. The combination of these effects will vary from site to site depending on the soils encountered.
Although the new methods of designing offshore piles are based on fundamental principles, careful checks are needed to ensure they are fully applicable to offshore design. Chow (1996) describes a study that examines the validity of the Imperial College methods. The EURIPIDES and other current research projects share the same aim (see Chapter 5).

It appears from this study that Toolan et al.'s method underpredicts pile capacity, a conclusion supported by a separate study by Gavin and Lehanes (1996). According to these authors, the API method has a tendency to underpredict the capacity of piles in dense and very dense sands, but overpredicts the capacities of long piles.

Randolph et al.'s method appears to be ill-conditioned for large offshore piles if the assumption is made that the pile's end-bearing ($q_e$) equals the cone resistance at the pile tip ($q_c$). It is likely that future development of the method will modify the assumed $q_e$-$q_c$ relationship (Randolph, 1996).

The systematic assessment of 65 pile load tests by Jardine and Chow (1996) confirms that the new Imperial College method currently offers the most promising improvements in predictive capability.

7.9 REFERENCES

AMERICAN PETROLEUM INSTITUTE (1993),
Recommended practice for planning, designing and constructing fixed offshore platforms — working stress design.

Investigations into the behaviour of displacement piles for offshore foundations.
PhD thesis, University of London

The reliability of conventional design methods for driven piles in sand.
Proc. 6th Int. Conf. and Exhibition on Piling and Deep Foundations, Bombay, India, Deep Foundation Institute.

New pile design methods for offshore piles.
MTD Ltd, London.

Axial capacity of an offshore pile driven in dense sand.
*Shaft capacity of driven piles in sand: a new design approach.*  
Proc. 7th Int. Conf. on Behaviour of Offshore Structures, Boston, 1, 23-36.

*Interpretation of cone penetrometer data for offshore sands.*  

*Personal communication with RJ Jardine.*

*Design of driven piles in sand.*  
Géotechnique, 44(3), 427-448.

*One-dimensional analysis of soil plugs in pipe piles.*  
Géotechnique 41(4), 587-598.

*An appraisal of API RP2A recommendations for determining skin friction of piles in sand.*  

*Frictional resistance at yield between dry sand and mild steel.*  
Soils and Foundations, 26(4), 139-149.
8. CONCLUSIONS

8.1 INTRODUCTION

Over 90 per cent of all structures founded in the UK sector of the North Sea are pile-supported jacket structures. The piles are typically open-ended steel pipes up to 2m in diameter and 80m or more in length. Their size is such that they dwarf their counterparts onshore and yet they are designed mainly on the basis of experience gained from testing onshore piles.

8.2 GEOLOGY AND GEOGRAPHY OF NORTH SEA SANDS

Sands are present in almost all parts of the North Sea, either as a thin surface layer, interbedded with clays, or as the predominant soil type. In the northern North Sea, stiff to very stiff overconsolidated clays are in abundance, although in many areas they are interbedded with dense fine sand. In the central North Sea, interbedded clays and sands predominate, whereas in the south there is a large tract of mainly fine to coarse sand.

The sands of the North Sea were laid down under a variety of depositional environments, mainly marine in the north, mixed marine and deltaic in the centre, and deltaic in the south.

8.3 FUNDAMENTAL BEHAVIOUR OF DRIVEN PILES IN SAND

Recent research into the behaviour of displacement piles in clays and sand has led to an improved understanding of the mechanisms that control the axial capacity of piles, and in particular the mobilization of shaft resistance.

When a pile is driven into the ground, its tip "punches" a hole in the soil which its shaft prevents from closing up. The shear stresses that act along the pile shaft vary enormously over the embedded length of the pile. In sands, the shear stress mobilized in any one soil horizon varies primarily with:

- Vertical effective stress
- Relative density
- Height of the horizon above the pile tip
The combination of these factors leads to a relationship between the local shear stress and depth below ground level that is approximately exponential in nature, with the greatest stress occurring near the pile tip. The average shear stress acting on the pile, however, appears to reach a constant value with increasing length of pile — the so-called critical depth phenomenon.

Excess pore pressures that are set up during pile installation dissipate during the subsequent equalization period and lead to a change in radial effective stress acting on the pile.

Further changes in radial effective stress take place during pile loading, owing to dilation at the pile/soil interface. The magnitude of these changes is inversely proportional to the radius of the pile.

The shear stress that is mobilized at failure is determined by the radial effective stress and the angle of friction at the pile/soil interface. The latter depends on the roughness of the pile wall and the characteristics of the sand.

8.4 EXISTING DESIGN PROCEDURES

The design of piles in the North Sea is, with minor variations, currently based on the 15th Edition of the American Petroleum Institute's Recommended practice for planning, designing and constructing fixed offshore platforms (API RP2A).

8.4.1 Shaft capacity of single piles

The shaft capacity of a single pile is calculated by integrating the pile's unit shaft resistance ($\tau_s$) over the full surface area of the pile. The shaft resistance is given by the following equation:

$$\tau_s = K \sigma'_y \tan \delta \cdot \tau_{\text{max}}$$

where $K$ is a dimensionless earth pressure coefficient; $\sigma'_y$ is the original vertical effective stress in the ground; $\delta$ is the angle of interface friction between the sand and the pile wall; and $\tau_{\text{max}}$ is the limiting value of shaft friction.

Typical values of these parameters for the dense to very dense sands that are encountered in the North Sea are:

- $K = 0.7$ in compression or 0.5 in tension
- $\delta = 30-35^\circ$
- $\tau_{\text{max}} = 96-115$ kPa

8.4.2 End-bearing capacity of single piles

The end-bearing capacity of a single pile is calculated by integrating
the pile's end-bearing resistance \( q_b \) over the full cross-sectional area of the pile. The end-bearing resistance is given by the following equation:

\[
q_b = \alpha \sigma' v_b N_q + q_{\text{max}}
\]

where \( \alpha \sigma' v_b \) is the original vertical effective stress in the ground at the pile toe; \( N_q \) is a dimensionless bearing capacity factor; and \( q_{\text{max}} \) is the limiting value of end-bearing.

Typical values of these parameters for the dense to very dense sands that are encountered in the North Sea are:

- \( N_q = 40-50 \)
- \( q_{\text{max}} = 9.6-12.0 \text{MPa} \)

### 8.5 Recent Developments in Design Methods for Driven Piles in Sand

Recent research into the behaviour of displacement piles in clays and sands has led to a re-appraisal of existing design methods and a questioning of some of the fundamental assumptions that are built into them. In particular, the concept of a *critical depth*, below which the unit shaft resistance of a pile in sand no longer increases with increasing depth, is now thought to be in error. Current thinking is that the shear stress that can be mobilized in any one soil horizon decreases with continued penetration of the pile tip below that horizon. This so-called *friction-fatigue* or \( h/R \) *effect* can be used to explain the tendency for shaft resistance to reach an apparently constant value at some depth and hence is consistent with past experience from pile loading tests in sand.

Arising from the recent research are a number of new design methods that take account of friction-fatigue, the influence of the sand's relative density and interface shearing characteristics, and changes in radial effective stress during pile loading. The new methods provide improved correlations with the results of existing pile tests in sand.

### 8.6 Design Parameters for North Sea Sands

Design parameters for offshore sands are usually determined from a combination of *in situ* tests and laboratory tests on reconstituted material. *In situ* tests provide information about relative density and the *in situ* stress state to which laboratory sample must be reconstituted before testing.

The primary *in situ* test used offshore is the cone penetration test, which provides measurements of cone resistance and sleeve friction.
Recent modern practice includes the measurement of pore pressure on or just behind the cone face.

The cone penetration test (CPT) can be used to determine the stratigraphy of the soils encountered at a particular site. Various classification charts exist for this purpose. In addition, the CPT can be used to estimate the relative density of the in situ sand and, indirectly, its angle of shearing resistance.

The vast majority of published information about the properties of North Sea sands comprises grading curves and the results of cone penetration tests. Investigations of mineralogy are normally carried out.

8.7 USING THE METHODS IN PRACTICE

Predictions of shaft and end-bearing capacity using five of the methods described in Chapters 4 and 5 give a very wide range of calculated capacities:

- The API method (used verbatim and as modified for UK practice) tends to underpredict the capacity of piles in dense and very dense sands, but overpredicts the capacities of long piles
- Tuolan et al.'s (1990) method underpredicts pile capacity
- Randolph et al.'s (1994) method appears to be ill-conditioned for large offshore piles if the assumption is made that the pile's end-bearing equals the cone resistance at the pile tip
- The new Imperial College method (Lehane and Jardine, 1994) currently offers the most promising improvements in predictive capability

Although the new methods of designing offshore piles are based on fundamental principles, careful checks are needed to ensure they are fully applicable to offshore design.

It is no longer logical to apply limiting values to local skin friction acting on offshore piles. The removal of limiting skin friction values has a strong influence on predicted shaft capacities, as does the "h/R effect". The combination of these effects will vary from site to site depending on the soils encountered.

8.8 REFERENCES

Recommended practice for planning, designing and constructing fixed offshore platforms — working stress design.
API Recommended Practice 2A-WSD (RP 2A-WSD), 20th edition,
Am. Petroleum Inst., Washington, DC.

*Shaft capacity of driven piles in sand: a new design approach.*  
Proc. 7th Int. Conf. on Behaviour of Offshore Structures, Boston, 1, 23-36.

*Design of driven piles in sand.*  
Géotechnique, 44(3), 427-448.

*An appraisal of API RP2A recommendations for determining skin friction of piles in sand.*  
A. PILE TESTS IN SAND – EXISTING DATABASES

A.1 INTRODUCTION

The following sections give details of the main databases of pile tests in sand that have been published in the geotechnical literature.

A.2 DENNIS AND OLSEN'S DATABASE

As part of a project sponsored by the American Petroleum Institute (API), Dennis and Olsen (1983) assembled a database of 1004 pile load tests through a survey of the literature and correspondence with government agencies, oil companies, and consulting firms. Of these, 66 were credible tests on pipe piles driven in sands at sites where at least some features of the soil profile had been recorded. Six of the tests were in carbonate sands; supplementary information on residual driving stresses was obtained from 7 tests on H-section steel piles. Dennis and Olsen did not identify any of the cases entered into their database, but the principal statistics of the tests are set out in Table 19.

Dennis and Olsen used their database to test the pre-14th edition API RP2A recommendations for piles in sand. Their main conclusions were that the design rules were marginally conservative, on average, but gave a remarkably wide spread between predicted capacity ($Q_c$) and measured capacity ($Q_m$). $Q_c/Q_m$ ranged between 0.2 and 11.0 and a strong bias was found for the capacity of long piles to be over-predicted. They proposed that the API method could be improved by:

- Extending the soil classification system
- Varying the parameters $\delta$ and $N_q$ with relative density as well as particle size (whilst leaving the earth pressure coefficient $K$ independent of any soil parameter)
- Treating the shaft capacity of tension and compression piles equally
- Excluding piles in calcareous sands and applying depth correction factors to both the base and shaft resistance
Table 19
Principal statistics of the API database of pile test results

<table>
<thead>
<tr>
<th>Property</th>
<th>Compression tests</th>
<th>Tension tests</th>
<th>Strain gauged tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total number</td>
<td>46</td>
<td>20</td>
<td>21</td>
</tr>
<tr>
<td>Range of capacities (kN)</td>
<td>450-9000</td>
<td>60-2000</td>
<td></td>
</tr>
<tr>
<td><strong>Lengths</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>&lt; 15m</td>
<td>28%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>15-30m</td>
<td>52%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>30-46m</td>
<td>15%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>&gt; 46m</td>
<td>5%</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Soil type &amp; pile lengths</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Silica sand</td>
<td>100%</td>
<td>65% (6.5-22m)</td>
<td></td>
</tr>
<tr>
<td>Carbonate sand</td>
<td>0%</td>
<td>30% (45-83m)</td>
<td></td>
</tr>
<tr>
<td>Gravel</td>
<td>0%</td>
<td>5%</td>
<td></td>
</tr>
<tr>
<td><strong>Pile type</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Steel pipe</td>
<td>53%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Concrete pipe</td>
<td>14%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Steel H</td>
<td>33%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Proportion in tension</td>
<td>29% (3-30m)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*Data from Dennis and Olsen (1983)*

Their proposals were developed from statistical analysis, rather than an investigation of the basic mechanics involved, and resulted in the range for $Q_c/Q_m$ narrowing to 0.49-2.17 and the standard deviation in $Q_c/Q_m$ falling from 1.68 to 0.38. They also investigated the possible effects of placing absolute upper limits on the end bearing capacity and local shaft resistance and suggested values that made a marginal improvement to the reliability of their design method. With the exception of the depth reduction factors referred to above, Dennis and Olsen's conclusions formed the basis of the revisions made in the 15th edition of API RP2A.
A.3 BRIAUD AND TUCKER'S DATABASE

In 1984, Briaud and Tucker published a method for analysing driven piles in sand in which hyperbolic load transfer functions could be developed for end bearing and shaft resistance on the basis of standard penetration tests (see Chapter 4); the method attempted to account for the residual stresses set up by pile driving. They developed and calibrated their method through a series of correlations drawn from a database of 32 instrumented tests, performed at nine sites, on steel and concrete piles. The tests that were included in their database are summarized in Table 20.

A.4 LINGS' DATABASE

The publication in 1984 of the 15th edition of API RP2A led to concern in the UK that the new recommendations might be unconservative, particularly for long tension piles in sand. Lings (1985) investigated the problem by re-examining the available field database for piles. Early on in his study, Lings concluded that residual stress corrections were likely to be unreliable in most cases and concentrated his attention on tension tests performed at dominantly granular sites, where end resistance could be discounted. The main features of Lings' study were the collation of data from several other sites and the care with which he analysed the source references in order to:

- Identify the soils profile and exclude any sites with excessive variability or inappropriate clay layers
- Establish the soils' relative density and the initial vertical effective stress distribution at each site
- Assess the quality of the load tests and allow for differences between open-ended and closed-ended piles

The tension tests which Lings considered to be reliable are tabulated in Table 20. Comparing his database with the recommendations of API RP2A (15th edition), Lings found surprisingly good predictions, on average. But the bias noted by Dennis and Olsen (1983) — for the capacity of long piles to be over predicted — remained evident, probably because the depth corrections proposed by Dennis and Olsen had not been incorporated into the 15th edition of RP2A. Lings drew attention to the trends with depth discussed by Vesic (1970) and noted that the shaft capacity developed at any given level in the soil profile might be dependent on the relative position of the pile tip, a factor that had not been addressed explicitly in any of the earlier analyses referred to above.

The practical implications of Lings' study were discussed by Toolan and Ims (1988) and the work was further extended by Toolan et al. (1990), who added three further tension test case histories (see Table 20).
Table 20
Pile tests included in Lings' (1985) database

<table>
<thead>
<tr>
<th>#</th>
<th>Site</th>
<th>Pile type</th>
<th>Length (m)</th>
<th>Diameter (mm)</th>
<th>B&amp;T</th>
<th>Li</th>
<th>T+</th>
<th>L+</th>
<th>R+</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Arkansas River</td>
<td>Steel pipe &amp; H</td>
<td>12-16</td>
<td>360-520</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>2</td>
<td>Low-sill, Louisiana</td>
<td>Steel pipe &amp; H</td>
<td>13.7-24.5</td>
<td>400-530</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>3</td>
<td>Ogeechee River, Georgia</td>
<td>Steel pipe</td>
<td>3.0-10.0</td>
<td>457</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>4</td>
<td>Lock &amp; Dam 26</td>
<td>Steel H</td>
<td>16.4-24.5</td>
<td>406</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>5</td>
<td>West Seattle Bridge</td>
<td>Steel H</td>
<td>25.6-30.0</td>
<td>625</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>6</td>
<td>Quebec</td>
<td>Steel H &amp; concrete</td>
<td>18.3</td>
<td>225</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>7</td>
<td>Drammen, Norway</td>
<td>Circular concrete</td>
<td>8.0-23</td>
<td>280</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>8</td>
<td>Corpus Christi, Texas</td>
<td>Square concrete</td>
<td>10.2</td>
<td>457</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>9</td>
<td>Sweden?</td>
<td>Square concrete</td>
<td>10.8</td>
<td>305</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>10</td>
<td>Lower Arrow Lake, British Columbia</td>
<td>Steel pipe</td>
<td>45.4</td>
<td>610</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>11</td>
<td>Mustang Island, Texas</td>
<td>Steel pipe</td>
<td>21.0</td>
<td>610</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>12</td>
<td>Hoogezand, Holland</td>
<td>Steel pipe</td>
<td>6.75</td>
<td>356</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>13</td>
<td>North Sea</td>
<td>Steel pipe conductor</td>
<td>30.5-38.2</td>
<td>610-669</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>14</td>
<td>Padre Island, Texas</td>
<td>Steel pipe conductor</td>
<td>14.6-17.1</td>
<td>598</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>15</td>
<td>Las Barrion, Cadiz, Spain</td>
<td>Concrete pipe</td>
<td>18.5</td>
<td>914</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>16</td>
<td>Blount Island, Florida</td>
<td>Steel pipe</td>
<td>22.6</td>
<td>273</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>17</td>
<td>San Francisco</td>
<td>Steel pipe</td>
<td>9.15</td>
<td>273</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>18</td>
<td>Dunkerque</td>
<td>Steel pipe</td>
<td>11.6</td>
<td>324</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>19</td>
<td>Offshore Isreal</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>Ras Tanahib, Saudi Arabia</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>21</td>
<td>Labenne, SW France</td>
<td>Closed-ended steel pipe</td>
<td></td>
<td>102</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>22</td>
<td>Holi-Ts</td>
<td>Steel pipe</td>
<td>34.3</td>
<td>610</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
</tbody>
</table>

B&T = Briaud and Tucker (1984); Li = Lings (1983); T+ = Toolan et al. (1990); L+ = Lehane (1992); R+ = Randolph et al. (1994)
Comparing their combined database of tension pile capacities with those computed using the 15th edition of API RP2A, Toolan et al. (1990) note a mean $Q_c/Q_m$ value of 1.12. In addition to the average ratio being significantly non-conservative, the standard deviation in $Q_c/Q_m$ was also relatively large, at 0.67, and the data continued to show a strong bias for the capacity of longer piles to be over-predicted, as illustrated in Figure 81. The alternative method proposed by Toolan et al. (1990) for assessing pile shaft capacity in sand is discussed later, but we should note here that Toolan et al. considered a length effect associated with "friction fatigue", or the dependence of shaft resistance on the relative position of the pile tip, to be a highly significant factor.

**Figure 81**

Trends for $Q_c/Q_m$ against pile penetration

(after Toolan et al., 1990)

### A.5 OTHER DATABASES

The following paragraphs summarise some additional recent contributions to the database for piles driven in sand.

Briaud and Tucker (1988) presented a broad summary of the performance of 98 piles which were tested to failure under compressive axial loading in a range of soils by the Mississippi State Highways Department. The authors did not identify the individual tests, or give many details. But it seems from their summary plots that eight, mostly box section concrete or H-section steel piles, had been driven into sand and that their typical dimensions were 12m in length.
and 400mm in equivalent diameter. The measured capacities of these relatively small piles fell between 500 and 2500kN and were typically twice those computed by the 15th edition of API RP2A. These observations are compatible with the trend shown by the data plotted on Figure 81.

Briaud et al. (1989) presented details of tests on 273mm diameter, 9.15m long, closed-ended steel pipe piles that had been driven into clean hydraulic sand fill in San Francisco.

Some results of the CLAROM research project into piles in sand were published in 1991 by Bruyé et al. (1991) and by Le Tirant et al. (1991). The first paper described driving trials, model tests, and full scale tests performed at the Dunkerque dense sand site: the load tests to failure were performed on steel pipe piles which were 324mm in diameter and 11.6m long.

In their companion paper, Le Tirant et al. (1991) returned to the concept of limiting values for unit shaft and base resistances, as proposed by Kerisel (1964) and Vesic (1964) for piles driven in sand. They collated data from some of Lings' (1985) sites with additional tests from the Dunkerque programme and piles driven in San Francisco hydraulic fill (see Briaud et al., 1989), a mixed sand/clay/calcerenite profile from offshore Israel (from Komornik et al. 1989), and the Ras Tanabib case history from Saudi Arabia (see Helfrich et al. 1985). Their analysis suggested that the API RP2A 15th edition rules might be generally conservative in very dense sands and possibly non-conservative in loose to very loose deposits. They also concurred with a suggestion by Lings (1985) that tension and compression shaft capacities might be comparable in dense sands, but that tension capacities could be relatively low in loose deposits. Their main point, however, was that the subject was poorly understood and that large scale tests on fully instrumented piles were needed urgently.

Lehane (1992) performed field research on a range of soil types using highly instrumented piles. Included in his programme were three tests with a 102mm diameter, 6m long steel pipe pile in the loose to medium dense dune sand found at Labenne, south west France. The Labenne results were reported by Lehane et al. (1993) and showed for the first time how the local effective stresses develop during installation, equalization, and load testing. In particular, the tests identified an important dependence of the earth pressure coefficient acting at failure ($K_F$) on relative density, the relative position of the pile tip, and dilation developing at the pile-soil interface. The results are discussed in more detail in Appendix B; we note here that the programme led to a new design approach which Lehane (1992) tested against a selection of larger experiments. He considered only those tests where reliable measurements had been made of shaft resistance; the cases that he re-evaluated were the Drammen, Ogechee, Hoogzand, and San Francisco tests (see Table 20).
Holm presented a summary paper in 1992 which referred briefly to the long
term behaviour of piles driven in sand in Sweden. The Swedish Geotechnical
Institute had collated a series of stress-wave measurements made at various
sites which he interpreted as showing a steady build up of shaft capacity over
the first six months after they were driven; no details were given of the piles or
site conditions. Tavenas and Audy (1972) also reported an unexpected
tendency for piles driven in sand to show increases in capacity of 50 to 90%
over the first 20 days of their lives. Mitchell and Solymar (1984) discuss the
possible mechanisms by which sands might gain strength and stiffness through
ageing.

After reviewing recent research, Randolph et al. (1994) proposed a design
method for piles driven in sand which synthesized some of the developments
discussed above, focusing particularly on end-bearing, the "friction fatigue" of
shaft friction, and differences between tension and compression loading. The
new approach was tested against a database which consisted of 36 tests
performed at ten sites. The cases considered were:

- The Drammen, Low Sill, Oggeechee River, Hoogzand, Seattle and Los
  Barrios tests (see tables above for Briaud and Tucker, 1984; Lings, 1985;
  Toolan et al., 1990).
- The San Francisco, Dunkerque and Labenne tests described above.
- Tests at the Hsin-Ta inter-layered clay and sand site involving steel pipe
  piles which were 34.3m long and 610mm in diameter; these experiments
  had been reported by Yen et al. (1989).

Randolph et al. considered both compression and tension loading, and noted
that uncertainty was inevitable concerning the definitions of failure and the
separation of base and shaft loads at failure.

Recently, Chow (1996) has assembled a new database of 65 high quality pile
load tests in sand. This has been used to validate the improved design approach
described by Jardine and Chow (1996).

A.6 REFERENCES

Recommended practice for planning, designing and constructing fixed offshore
platforms.
Washington, DC.

Piles in sand: a method including residual stresses.
*Measured and predicted axial response of 98 piles.*  

*Axially loaded 5 pile group and single pile in sand.*  

*Behaviour of pile plug in sandy soils during and after driving.*  

*Investigations into the behaviour of displacement piles for offshore foundations.*  
PhD thesis, University of London

*Axial capacity of steel pipe piles in clay.*  
Proc. Conf. on Geotech. Practice on Offshore Enng, Austin, Texas, 370-388.

*Pile load tests in dense sand: planning, instrumentation and results.*  

*Piling in Sweden, Denmark, Norway and Finland.*  

*New pile design methods for offshore piles.*  
MTD Ltd, London.

KERISEL J. (1964).  
*Deep foundations — basic experimental facts.*  
Proc. Conf. on Deep Foundations, Mexico, **1**, 5-44.

*Piles for an offshore unloading terminal.*  

*Experimental investigations of pile behaviour using instrumented field tests.*  


*Interpretation of instrumented driven steel pipe piles.*
B. METHODS USED TO OBTAIN THE PROPERTIES OF NORTH SEA SANDS

B.1 IN SITU TESTS

B.1.1 Cone penetration test

The in situ test that is used most widely offshore is the cone penetration test with pore pressure measurement (CPTU or piezocene test), in which continuous measurements are made of:

- Cone tip resistance ($q_c$)
- Sleeve friction ($f_s$)
- Penetration pore water pressure* ($u$)

The standard cone has a 60° apex angle, a cross-sectional area of 10cm$^2$, and a friction sleeve area of 150cm$^2$ (see Figure 82). The cone penetrates the ground at a rate of 2cm/sec. In the North Sea, 15cm$^2$ cones are sometimes used (de Ruiter, 1982). The cone may be operated from the sea-bed or within a borehole ("downhole").

Measurements of cone resistance and sleeve friction may be influenced by pore water pressures that are generated behind the cone tip and at each end of the friction sleeve and which exert a net force on the tip and sleeve; when cone resistance and sleeve friction are corrected for this effect, they are given the symbols $q_t$ and $f_t$.

In sands, CPTU tests are interpreted to provide information on stratigraphy, grain size, relative density, angle of shearing resistance, and shear modulus. Where excess pore pressures are generated, their dissipation may be interpreted to yield data on permeability. Measurements of $q_c$ (or $q_t$) are sometimes directly related to pile end-bearing resistance in closed-end piles (which requires scale effects to be considered), whereas measurements of $f_s$ (or $f_t$) are related to shaft resistance (although there are dangers in such relationships, because of the h/R effect described in Chapter 3).**

*Measured on the face or shoulder of the cone

**Values of $f_s$ are, in fact, only relevant to the shaft friction close to the pile tip
B.1.2 Dilatometer test

The dilatometer test measures soil contact pressure \( (p_0) \) and a pressure on a plate \( (p_1) \), when it has been expanded 1mm laterally into the soil — see Figure 83. In offshore use, the plate is 77mm wide and 16mm thick. The expansion takes approximately 15 seconds and the test is carried out at vertical intervals of 20cm. Empirical correlations have been proposed for deriving in situ horizontal stress, stress history, constrained modulus, and friction angle (Marchetti, 1980; Schmertmann, 1982 and 1986; and Lunne et al., 1989).
B.1.3 Pressuremeter test

In the pressuremeter test, the *in situ* strength and deformation characteristics of the ground are determined by expanding a cylindrical cavity and measuring the relationship between the applied pressure and the expansion of the cavity. A test may be carried out:

- In a pre-drilled borehole (Ménard type) — see Foldout 6(a).
- With a self-boring pressuremeter (SBPM) (Baguelin et al., 1972; Wroth and Hughes, 1973) — see Foldout 6(b).
- With a push-in pressuremeter (PIP) (Reid et al., 1982; Pyffe et al., 1986) — see Foldout 6(c).
- With a full-displacement cone-pressuremeter (Hughes and Robertson, 1985; Withers et al., 1986) — see Foldout 6(d).

Both the push-in pressuremeter and the cone-pressuremeter are suitable for offshore use because they are robust. The SBPM has also been used offshore (e.g. Fay et al., 1985; Fay and Le Tirant, 1990). Foldout 6(e) shows a wireline SBPM, but this is still not commonly deployed.

A review of pressuremeter testing and its interpretation is given by Mair and Wood (1987). Interpretation of the cone pressuremeter test in sand is discussed by Houlsby and Schnaid (1994).
B.1.4 Seismic cone penetrometer test

The seismic cone penetrometer test (SCPT) combines a piezocone with one or two sets of geophones built into the cone (Campanella and Robertson, 1984; Campanella et al., 1986). The downhole shear wave velocity ($V_s$) is measured at depth intervals of 1m. The small strain shear modulus ($G_{max}$) is derived from $V_s$ and bulk density. This device is being used increasingly offshore.

B.1.5 In situ density probes

The Nuclear Density Probe (Tjelta et al., 1985) combines a radioactive source and detector inside a 15cm$^2$ cone. The bulk density of the soil in situ is determined by calibrating the device in fluids of known density.

The Electrical Resistivity Probe (Kroezen, 1981) measures the resistivity of the pore water and this is correlated to density through laboratory calibrations.

B.1.6 Standard penetration test

The standard penetration test (SPT) is not widely used offshore and there is no established method for correcting the SPT blow count for energy loss using a wire-line (down-the-hole) tool.

B.2 LABORATORY TESTS

Laboratory tests on sands can be used to establish grading, grain shape, crushability, maximum and minimum void ratios, angles of shearing resistance and interface friction, angles of dilation, and shear and bulk stiffness characteristics. Laboratory, rather than in situ, tests are also used to assess response to cyclic loading.

Laboratory tests to measure stress-strain-strength properties are usually run on reconstituted samples. The method of forming the samples determines the fabric of the sand and its response at small and intermediate strains (see Figure 84). This needs to be taken into account when reconstituting samples for laboratory tests and a method appropriate to the depositional mode of the in situ sand should be selected.
B.2.1 Measurement of stress-strain properties

Measurements of soil deformation made remotely from the soil sample, as in the external measurement of axial displacement in the triaxial apparatus, are subject to significant errors resulting from, *inter alia*:

- Bedding at the sample/loading platten interface
- Compliance in the load measuring system
- Tilting of the sample

Stiffness and compressibility determined from external measurements in the triaxial apparatus and oedometer are, therefore, often unreliable.

In the case of the direct simple shear or torsional shear tests, slip may occur at the sample/loading platten interface and introduce errors in the determination of shear stiffness.

In triaxial and torsional shear apparatuses, there is space in the pressure chamber to incorporate instrumentation that can be attached over a gauge length on the sample and so avoid the potential measurement errors. Reliable determinations of small strain stiffness characteristics of sands require the use of this local instrumentation in stress path tests in which the recent stress history of the sand can be followed. Where these devices have been used there has been a tendency to concentrate on the measurement of shear stiffness and
its variation with strain level. Measurements of bulk stiffness at small strains are also important for more sophisticated analyses of pile behaviour. Reference may be made to the following publications for more information.


B.2.2. Measurement of $G_{\text{max}}$

The dynamic shear modulus of laboratory samples of sand can be determined using the resonant column apparatus (e.g. Hardin and Drnevich, 1972) or using bender elements (e.g. Schulteiss, 1980; and Dyvik and Madshus, 1985).

In the resonant column apparatus, solid or hollow cylindrical samples of sand may be tested, usually in a fixed-free configuration. After reconsolidation to the estimated in situ stresses, a coil-magnet drive system at the top of the sample is used to vibrate the sample at first-mode resonance in torsional motion. Shear modulus is determined from the resonant frequency combined with system constants and sample density.

Bender elements are bimorph piezo-ceramic crystals which are embedded in the ends of triaxial or oedometer samples. Seismic body waves are generated by the bender element at one end of the sample and received by the bender element at the other end. The travel time and hence the shear wave velocity ($v_s$) are determined and can be converted to dynamic shear modulus ($G_{\text{max}}$) by the equation:

$$G_{\text{max}} = \rho v_s^2$$

where $\rho$ is the soil's density.

B.2.3 Measurement of angles of friction and dilation

The strength and volume-change characteristics of sand are conventionally measured in the triaxial apparatus, the direct shear box, or the direct simple shear apparatus. More recently the torsional shear apparatus has been introduced. This test enables some of the uncertainties involved in the interpretation of shear box and simple shear tests to be removed.

These tests provide data on strength and dilation for the particular shearing modes listed in Table 3.

A more complete picture of the anisotropy of strength and dilation requires tests in the hollow cylinder apparatus (e.g. Hight et al., 1983) or on tilted samples in the triaxial or plane strain apparatus (e.g. Tatsuoka et al., 1988)
Table 21
Typical parameters for various shearing modes

<table>
<thead>
<tr>
<th>Shearing mode</th>
<th>b = ( \frac{\sigma_2 - \sigma_3}{\sigma_1 - \sigma_3} )</th>
<th>Direction of the major principal stress to the vertical (( \alpha ))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Triaxial compression</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Triaxial extension</td>
<td>1</td>
<td>90°</td>
</tr>
<tr>
<td>Torsional shear</td>
<td>0.3-0.5</td>
<td>approx. 45°</td>
</tr>
<tr>
<td>(shear box, simple shear)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

B.2.4 Measurement of interface friction

The angle of interface shearing resistance (\( \delta \)) can be measured in direct shear interface tests, using sand from the site, sheared against a steel interface having the same roughness and hardness as the prototype piles. The tests need to be conducted with the sand at its in situ relative density and with constant normal stress levels appropriate to the pile depth. The tests should involve the measurement of volume change and need to be continued until constant volume conditions are reached, since \( \delta_{cv} \) and not \( \delta_{max} \) controls shaft capacity.

Although the direct shear box has imperfect boundary conditions, \( \delta_{cv} \) from such tests was found by Jardine et al. (1992) to be within 1° of values measured in equivalent tests in the ring shear apparatus.

To investigate interactions between dilation in the shear zone and the stiffness of the surrounding ground, Boulon and Foray (1986) advocate controlled normal stiffness interface shear tests.

B.3 REFERENCES


*Physical and numerical simulation of lateral shaft frictions along offshore piles in sand.*  
Proc. 3rd Int. Conf. on Numerical Methods in Offshore Piling, Nantes, 127-147.

*A seismic cone penetrometer to measure engineering properties of soil.*  
Society of Exploration Geophysicists' 54th Annual Meeting, Atlanta.

*Seismic cone penetration test.*  

DE RUITER J. (1982).  
*The static cone penetration test.*  

*Laboratory measurements of G,max using bender elements.*  

*Use of the PAM self-boring pressuremeter and the STACOR large-size fixed-piston corer for deep seabed surveying.*  

*Offshore wireline self-boring pressuremeter.*  
Proc. 3rd Int. Symp. on Pressuremeters, Oxford, 55-64.

*The push-in pressuremeter: 5 years off-shore experience.*  

HARDIN B.O. and DRNEVICH V.P. (1972).  
*Shear modulus and damping in soil: design equations and curves.*  
*The development of a new hollow cylinder apparatus for investigating the effects of principal stress rotation in soils.*

*Interpretation of shear moduli from cone pressuremeter tests in sand.*
Geotechnique 44(1), 147-164.

*Full displacement pressuremeter testing in sands.*

*Friction coefficients for piles in sands and silts.*

*Measurement on in situ density in sandy/silty soils.*

*General report/discussion session 2: SPT, CPT, pressuremeter testing and recent developments in in-situ testing — Part I: All tests except SPT.*

*Pressuremeter testing: methods and interpretation.*

MARCHETTI S. (1980).
*In situ tests by flat dilatometer.*

*The influences of sand fabric on liquefaction behaviour.*
Waterways Experimental Station, Vicksburg, Miss., Contract Report 5-76-5, 38pp.

*The push-in pressuremeter.*
_A method for determining the friction angle in sands from the Marchetti dilatometer tests._  

_Dilatometer to compute foundation settlement._  
Proc. Conf. on Use of In Situ Tests in Geotechnical Engineering (IN SITU '86), Blacksburg, Virginia, Am. Soc. Civ. Engrs.

SCHULTEISS P.J. (1980).  
_Simultaneous measurement of P and S wave velocities during conventional laboratory soil testing procedures._  
Marine Geotechnology 4(4).

_Discussion on paper by Jewell and Wroth._  

_In situ density measurements by nuclear backscatter for an offshore soil investigation._  

_The development of a full displacement pressuremeter._  

_An instrument for the in-situ measurement of the properties of soft clays._  
Proc. 8th Int. Conf. Soil Mech. & Fdn Engng, Moscow, 1,2, 487-494.