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This thesis has been accepted for the award of the degree in the Faculty of Engineering
Behaviour of Single Piles and Pile Groups in Calcareous Sediments

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SYNOPSIS

This thesis presents the results and analysis of an extensive study of the behaviour of model piles and pile groups in New North Rankin calcareous sand under static and cyclic loading, using different test boundary conditions. The results and experience gained from this study may be utilized in the theoretical analysis and design of offshore pile foundations.

To investigate soil strength and soil–metal interface behaviour, monotonic and cyclic shear box tests have been performed on different types of calcareous sands and one silica sand under constant normal load (CNL) and constant normal stiffness (CNS) conditions. The reduction in shear stress in the CNS tests may be predicted from the results of CNL tests. Thus CNL tests may be used to estimate the degradation of skin friction during cyclic pile loading.

Model pile tests on a single pile jacked into medium–dense and dense calcareous sand have been performed under various overburden pressures. Two different sizes of test vessel with different vessel base conditions have been used in these tests. There were two major aspects of these tests:

a) The effects of initial density, overburden pressure, vessel diameter, and vessel base conditions on the pile shaft friction under static loading, and the degradation factor for skin friction ($D_r$) under cyclic loading have been investigated. The influence of cyclic displacement amplitude and number of cycles on $D_r$ was also studied. The results showed that the initial density and overburden pressure have a significant effect on the jacking force required to install the pile, and on the static skin friction and the soil modulus obtained for static loading. It has also been found that the initial density, overburden pressure, cyclic displacement amplitude, and
number of cycles significantly influence the degradation factor for skin friction \( (D_\tau) \) under cyclic loading. The distribution of interface stress along the pile jacked into a sand consolidated by applying overburden pressure on both top and bottom ends of the sand was more uniform than that from applying overburden pressure only at the top surface of the sample.

b) The influences of different cyclic load levels, number of cycles and different types of cyclic loading (uniform and "storm") on the accumulation of cyclic displacement have been investigated. The results have shown that the accumulated displacement of the pile increases as the cyclic load level and number of cycles increase, but the increase depends on load level more than on number of cycles. For storm loading, the accumulated displacement in the first cycle appeared to be higher than for subsequent cycles in each parcel of load.

Model tests on pile groups jacked into calcareous sand consolidated in the large test vessel under various overburden pressures have been performed. The results of these tests have shown that, for medium–dense sand, the jacking force increases as the number of piles increases. The group efficiency \( (\eta) \) also increases as number of piles increases. The results have also shown that the degradation factor increases with increasing cyclic displacement and number of cycles. There is only a small effect of the number of piles on the skin friction degradation factor "\( D_\tau \)".

The accumulated displacement behaviour of a pile group was found to be similar to that of a single pile, with increases in displacement caused by increasing load level being more significant than increases arising from increasing numbers of cycles. The results showed a small influence of the number of piles on pile capacity and group settlement for both uniform and non–uniform cyclic loadings.

From the results of interaction tests between two piles, it has been found that there
is some interaction between two piles for small spacings and that sand density and overburden pressure both have an effect on the direction and magnitude of the deflection of an unloaded pile caused by the loaded pile.

The predicted and measured results for the residual load in a single pile and a pile in a group were found to be similar. The predicted and measured results for load–deflection response for a single pile and a pile group also showed good agreement.

The boundary element program "SCARP" has been used to analyse the response of a single pile and pile groups to static and cyclic loading. It was found that it was possible to predict, reasonably well, the residual load after jacking, and the static load–deflection response of the piles.

Provided that appropriate values of the input parameters are used, (especially Young's modulus \(E_y\)), the accumulated displacement for both uniform and non-uniform cyclic loading predicted by SCARP has been found to be in good agreement with the single pile results, although agreement for pile groups was less satisfactory. The differences between predicted and measured results increases with the number of piles in a group. This behaviour may be attributed to the assumption of soil homogeneity between the piles in the group, which is used in the program "SCARP". On the basis of these comparisons, suggestion are made for modifications of the SCARP program and also for the determination of the appropriate input parameters.
PREFACE

The candidate carried out the work described in this thesis during the period 1987–1992. All of the work was conducted in the School of Civil and Mining Engineering, at the University of Sydney.

The candidate was supervised by Professor H.G. Poulos, Professor of Civil Engineering, except for the period of Professor Poulos' sabbatical leave when he was supervised by Associate Professor J.C. Small.

The By-Laws of the University of Sydney require a candidate for degree of Doctor of Philosophy to indicate which sections of the thesis are original. Although many references have been used during the course of this research program, any information or ideas derived from these sources has been acknowledged in the text. In accordance with the above mentioned By-Laws the Author claims originality for the following work:

1) In Chapter 3, the modification of the shear direct box to perform static and cyclic shear tests on different types of carbonate and non-carbonate sands under the "CNL" condition, including the scanning electron micrographs of the metal surfaces and metal–soil interface.

2) In Chapter 4, the monotonic and cyclic "CNS" tests on dense samples of calcareous sand that were conducted, and the correlation between CNL and CNS tests made to estimate the degradation of pile skin friction during cyclic pile loading.

3) In Chapter 5, the modification and design of model single piles and pile groups, and the test apparatus and equipment (e.g sand rainer, vessel base, collars) for the preparation of reconstituted beds of sands; the method of applying the
overburden pressure at the top and also at the bottom surfaces; also planning, conducting and reporting of three series of experiments on a) raining sand into a vessel, b) the influence of vessel size on the vertical stress transfer to the base of the vessel, and c) the surface deflection of sand in the vessel during consolidation and the process of pile loading.

4) In Chapter 6, the planning, conduct and analysis of model single pile tests, together with comparisons with other model pile tests.

5) In Chapter 7, the planning, conduct and analysis of model pile group tests, together with comparisons with the model single pile tests results. Also planning, conduct and analysis of experiments involving interaction of two piles with spacing varied.

6) In Chapter 8, using program SCARP for pile and pile group analysis as well as the comparison of theoretical predictions with the model pile tests and model pile group tests from Chapter 6 and Chapter 7.

A number of research papers were prepared and published by the Author and others during the period of the Author's candidature. These are submitted in support of this thesis. They are:


Other Papers in preparation, but not submitted with this thesis, are:


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The work described in this thesis forms part of a continuing research project on the mechanics of calcareous sediments being carried out within the Centre for Geotechnical Research at the University of Sydney. I wish to thank Professor J.R. Booker and Professor J.P. Carter for their support and encouragement and also thanks to all the academic staff and my colleagues and fellow research students for their useful discussions, especially Drs C.Y. Lee, P.T. Brown, N. P. Balaam and L.W. Apperley.

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Finally I would like to thank my Mother, brothers and sisters and friends wherever they are for their support and encouragement throughout my postgraduate years at University.
NOTATION

A  Permanent displacement parameter
A_p  Pile cross-sectional area
A_{pi}  Cross-sectional area of pile
A_s  Soil-pile shaft contact area.
B  Soil parameter used in permanent settlement equation
C  Soil cohesion
C_I  Compression index
C_s  Swelling index
C_c'  Soil crushing coefficient
C.C  Soil carbonate content
C_r  Correction factor
CNL  Constant normal load
CNS  Constant normal stiffness
C_\alpha  Coefficient of secondary compression
C_v  Coefficient of consolidation
d  Pile diameter
D_{10}, D_{50}, and D_{60} = respective particle sizes at 10%, 50%, and 60% finer
D_E  Soil Young's modulus degradation factor
D_r  Relative density of soil
D_{r'}  Degradation factor for skin friction
D_f  Reduction factor for skin friction prior to the current cycle of loading
D_{r_{min}}  Minimum possible value of D_r
e  Void ratio
\epsilon_o  Initial void ratio
E  Elastic modulus of the steel spring beam
$E_s$  Young's modulus of soil

$E_t$  Tangent "soil modulus"

$E_{25}$  Secant soil Young's modulus at load level 25%

$E_{50}$  Secant soil Young's modulus at load level 50%

$E_p$  Young's modulus of pile

$f_b$  Pile end bearing capacity

$f_c$  Skin friction after cyclic loading

$f_{gc}$  Average post-cyclic skin friction of pile group

$f_{gs}$  Average static skin friction of pile group

$G$  Shear modulus of soil

$G_s$  Specific gravity

$h$  Height of the sample in shear box

$K$  Spring stiffness

$K_s$  Coefficient of subgrade shear reaction

$K_o$  Coefficient of earth pressure at rest

$l$  Length of pile element

$L$  Pile length

$m,n$  Permanent displacement parameters

$m_v$  Coefficient of volume compressibility,

$m_v = C/(2.303 \times (1+e) \sigma_n)$

where $C = C_c$ or $C_s$ depending on whether the stress is increasing or decreasing respectively.

$N$  Number of cycles or

Number of piles in pile group

$N_f$  Number of cycles to cause failure

$N_{eq}$  Equivalent number of uniform stress cycles

$N_f$  Number of cycles to cause failure

$N_q$  Bearing capacity factor
OCR  Overconsolidation ratio
P    Pile load
P_{av} Average load of pile group
P_{c} Cyclic load
P_{g} Ultimate load capacity of the pile group
P_{m} Mean load
P_{\text{max}} Maximum applied pile load
P_{\text{min}} Minimum applied pile load
P_s Ultimate static pile load capacity
Q_c Average cone resistance
R    Roughness, \( R = (D_{60} \, D_{10})/D_{50} \)
R_{s} Settlement ratio, \( R_s = \rho_g/\rho_s \)
R_{st} Roughness coefficient for the structure material
R_{soil} Roughness coefficient for soil
R_{\text{max}} Surface roughness
r_m An equivalent radius \( r_m = 2.5 \ell \, (1-\nu) \)
r_o Pile radius
S    Pile settlement
S_p Permanent displacement of pile
S_{pN} Permanent displacement of pile at cycle N
s/d Ratio of the spacing between two piles to pile diameter
t The gap thickness between two halves of shear box
X    Cyclic stress level \( X = (P_m + 0.5P_c)/P_s \)
z    Depth
\alpha The interaction factor or
Ratio of empirical permanent displacement
parameters, \( \alpha = n/m \)
\gamma Unit weight of soil
\( \gamma_d \)  
Dry density

\( \gamma_{\text{max}} \)  
Maximum dry density

\( \gamma_{\text{min}} \)  
Minimum dry density

\( \gamma_{\text{bulk}} \)  
Bulk density

\( \delta \)  
Friction angle of pile-soil interface

\( \xi \)  
\( \ln \left( \frac{r_m}{r_o} \right) \)

\( \xi \)  
Arctan \( d/s \), in degrees, where 
\( s = \text{centre to centre spacing of piles} \)

\( \delta_p \)  
Friction angle of soil-metal interface at peak

\( \delta_{\text{u}} \)  
Ultimate friction angle of soil-metal interface at ultimate

\( \phi \)  
Internal friction angle of soil

\( \phi_p \)  
Internal friction angle at peak

\( \phi_{\text{u}} \)  
Internal friction angle at ultimate

\( \phi' \)  
Critical internal friction angle of soil

\( \phi_{\text{cv}}' \)  
Critical effective internal friction angle of soil

\( \lambda \)  
Degradation rate parameter

\( \nu \)  
Poisson's ratio of soil

\( \nu_s \)  
Poisson's ratio of soil

\( \rho \)  
Vertical displacement

\( \rho_b \)  
Pile base displacement

\( \rho_c \)  
Cyclic displacement

\( \rho_{\text{fs}} \)  
Displacement to cause static pile-soil slip

\( \rho_g \)  
Settlement of group

\( \rho_h \)  
Horizontal static shear displacement

\( \rho_s \)  
Soil displacement

\( \sigma_n \)  
Normal stress

\( \phi_c' \)  
Effective confining pressure in triaxial test

\( \sigma_{vo} \)  
Overburden pressure in pile test.
Chapter 1  INTRODUCTION 1

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Chapter 1

INTRODUCTION

1.1 INTRODUCTION

The design and construction of piles for offshore foundations in marine sediments has been studied extensively in the past two decades. This study has been accelerated by the rapid growth of offshore oil and gas exploration in that period. This production is expected to increase from 20% to 65% of total global production in the period 1982 to 2020 (Geer 1982).

Carbonate soils are found in the marine environments of tropical and temperate regions around the world, where many petroleum producing platforms have been built; for example those in the Gulf of Mexico, Southern California, Brazil, the Arabian Gulf, offshore India, Indonesia, and Bass Strait and the North West Shelf region of Australia. Carbonate soils cause engineering difficulties because they are composed of the skeletal remains of organisms, and have significantly higher in-situ void ratios than silica sands. The engineering behaviour of calcareous sands appears to be quite different from silica sands and this difference has been attributed in part
to the highly compressible and crushable nature of the carbonate particles (Datta et al., 1980 and Semple, 1988).

Previous experience from laboratory and field tests shows that the low capacity of driven and jacked piles in calcareous sand may result in problems in the design of driven pile foundations for offshore structures. Dramatic evidence of such problems materialized in 1982 when some driven piles at the site of the North Rankin platform on the North West shelf of Australia dropped as much as 60m (about 50% of design length) under their own weight while others exhibited a low driving resistance. Such experiences have stimulated investigators to perform laboratory work under a range of test boundary conditions, in order to reach a better understanding of the behaviour of piles and to develop means for making a reasonable estimation of pile capacity under both static and cyclic loading conditions.

Several programmes of model and field testing of piles in calcareous sands have been undertaken in the last decade, for example, an extensive model testing program at the University of Sydney, Australia by Poulos and his co-investigators, another model test program at the University of Western Australia by Randolph and his co-workers, a series of field and model pile tests in France by Nauroy, Le Tirant and Brucy, and the study of the North Rankin platform piles on the North West shelf of Australia by Khorshid (1990). However, the behaviour of pile groups, the interaction between piles in calcareous sands, and many aspects of the influence of boundary conditions on model test results, have not yet been investigated.

There is relatively little reliable data on the interaction between cyclically loaded piles in a group in non-calcereous soil. In-situ loading tests have been required to aid in the prediction of the behaviour of piles in these types of soil, but such tests are expensive and often time consuming and difficult to perform. However, in view
of the large pile sizes used recently, in-situ tests on offshore pile-groups may be impossible. Thus there is a need to perform a series of laboratory tests in order to help develop methods of analysis for the cyclic loading of piles. A general design procedure for a pile group, taking cyclic loading effects directly into account, is not yet fully developed.

A common method used to analyse the behaviour of a single axially loaded pile in an elastic soil is due to Poulos and Davis (1968), while extensions of this approach to analyse the loading of pile groups were made by Poulos and Mattes; 1971, Butterfield and Banerjee; 1971a and 1971b, Randolph and Wroth; 1979, O'Neill; 1983, El Sharnouby and Novak; 1985. Extensions to this approach to incorporate cyclic loading effects have been described by Poulos (1989a), Poulos and Lee (1989), and Hewitt (1988), but there have been few attempts to apply this theory to piles or pile groups in calcareous soils.

The most suitable types of piling for the support of offshore structures are driven piles, grouted piles, and driven and then grouted piles. Driven and then grouted piles are relatively new innovations, and have not yet been used widely in practice. Offshore grouted piles have been studied extensively (Nauroy, and Le Tirant; 1983, and Nauroy et al; 1985; 1986) and provide high static skin friction but may suffer from degradation of that skin friction during cyclic loading. Driven piles are still a favoured option because they are usually more economical, and less time-consuming and easier to install. They also suffer loss of skin friction due to cyclic loading, and this reduction of skin friction of the driven pile is attributed to the crushing of the soil particles during the driving procedure which reduces the lateral stress in the soil surrounding the piles. However, the end-bearing capacity of driven piles is greater than that of drilled and grouted piles. In this thesis, attention is focussed on the shaft resistance of jacked piles, and on the load-deflection behaviour of jacked piles.
under tensile loading. It is believed that the behaviour of jacked piles gives a reasonable indication of the behaviour of driven piles.

In the offshore environment, the major component of wave loading is in the vertical direction, resulting in axial loads being transferred to the foundation piles. Therefore, this study will look at the axial behaviour of both single piles and pile groups. One of the major design considerations for a pile group is the ultimate axial capacity and the load settlement response of the pile group after it has been subjected to prior cyclic loading.

The purpose of this thesis is to study experimentally the behaviour of model piles and pile groups subjected to static loading and various types of cyclic loading under different test boundary conditions, and to investigate some of the factors which affect this behaviour. In particular, the effects of sand density on the results of single pile and pile-soil-pile interaction are considered in this study. One part of this thesis will be dedicated to the design of a pile testing device considering different boundary conditions, and on the preparation of the reconstituted sand bed at the required density in the test vessel. Another portion of the effort of this study is devoted to a series of monotonic and cyclic shear box tests (CNL and CNS), using a modified conventional direct shear apparatus, to investigate the shear strength and stress deformation behaviour of the soil and how this behaviour influences soil-pile interface response. Attempts are also made to compare the measured model pile behaviour under static and cyclic loading with that predicted by a theoretical analysis using a modified form of boundary element analysis.
1.2 OUTLINE OF THE THESIS

Table 1.1 shows a summary of experimental work carried out in this thesis. Chapter 2 reviews literature on the properties of calcareous soil and the development of experimental and theoretical research on single piles and pile groups subjected to static and cyclic loading. The engineering properties of calcareous sand have been investigated by different researchers using triaxial tests and shear box tests under both conditions of constant normal load (CNL) and constant normal stress (CNS). The experimental work for piles includes laboratory and field tests. Efforts by a number of research workers over the last decade have been directed at obtaining a representative model of jacked and grouted pile foundations subjected to monotonic and cyclic loading. Some of these efforts are also reviewed.

Chapter 3 describes a modification of the experimental apparatus and test procedure for direct shear tests. This chapter also reports the results of over one hundred monotonic and cyclic shear box tests under constant normal load (CNL) for different types of calcareous sand and one type of silica sand. These tests have been performed to better understand the shear behaviour of soil and the pile-soil interface under static and cyclic loading, which is important for the design of offshore piles. For the interface tests, the surface roughness of aluminium and steel, $R_{\text{max}}$, has been measured using a scanning electron microscope. The influence of initial density, overburden pressure, cyclic displacement amplitude, number of cycles, water content, and particle size on the reduction of strength of soil-soil and soil-metal interfaces under cyclic loading has been investigated. Also, one series of tests has been performed to investigate the extent of particle crushing caused by cyclic loading.

In Chapter 4, the results from a series of static and cyclic shear box tests with either constant normal load (CNL) or constant normal stiffness (CNS) performed on
a very dense calcareous sand are reported. The purpose of these tests is to examine
the relationship between the degradation factor for skin friction in the CNS tests and
that in CNL tests during cyclic shearing. Consequently the results from CNL tests
may be used to estimate the degradation of skin friction of a pile due to cyclic
loading. The relationship between CNS and CNL tests requires consideration of the
change in volume occurring due to changing normal stress in the CNS tests. Thus,
one series of tests using an oedometer have been performed to measure the volume
compressibility of the soil.

Chapter 5 describes the experimental apparatus and procedure used for testing a
single pile and pile groups. The procedure for preparation of reconstituted beds of
sands, including details of measurements of density, are reported here. The results of
a series of tests investigating the vertical stress transfer to the base of the test vessel
are also reported. The effects of such factors as overburden pressure, initial void
ratio, dimensions of the vessel and roughness of the internal vessel wall are
considered. One series of tests has been carried out to investigate the surface
deflection of calcareous sand in the large vessel during consolidation and also during
the process of pile loading. The results of these tests show the variation of the
surface deflection with time of consolidation. The test results are compared with the
results from other publications on silica sand. The influence of applying pressure at
both the top and bottom ends of the sand surface, rather than only on the top, is
also examined.

Chapter 6 presents the results of a series of model tests on single piles jacked into
New North Rankin calcareous sand that had been consolidated at different initial
densities and under various overburden pressures. Two different sizes of test vessels
with different vessel base conditions have been used in these tests. In one series of
tests, the study is focussed on the influence of various parameters and test conditions
such as density, overburden pressure, cyclic displacement amplitude, moisture content, size of test vessel and type of vessel base on the pile shaft behaviour under static and cyclic loading. Another series of tests examines the influence of different types of cyclic loading (uniform and storm) on the accumulated displacement of the pile.

Chapter 7 presents the results of a series of tests on model pile groups jacked into calcareous sand consolidated in the large test vessel under various overburden pressures. Attention is focussed mainly on the influence of the number of piles on the load transfer and shear stress distribution along one pile in each group. The influence of number of piles on the degradation of skin friction along one pile in the group under uniform and non-uniform cyclic loading conditions is also included. The results of the tests on the interaction between two piles with different spacings are also presented here.

In Chapter 8, a numerical analysis is employed to predict the static and cyclic behaviour of a single pile and a pile-group as presented in Chapters 6 and 7. Attention is focussed mainly on the comparison between predicted and measured accumulated displacement for single pile and pile-group tests under conditions of uniform and non-uniform load-controlled cyclic loading.

Finally, the summary and conclusions, and recommendations for future research, are given in Chapter 9.
Table 1.1 Summary of Experimental Programme

Experimental Programme

Direct Shear Tests
  CNL Tests
  CNS Tests

Boundary Condition Tests
  Jacking Tests
  Static Tests
  Cyclic Tests

Single Pile Tests
  Load-Controlled Tests
  Disp.-Controlled Tests
  Individual Jacking
  Group Jacking

Pile Groups
  2 Pile-Groups & 4 Pile-Groups
  Jacking Tests
  Static Tests
  Cyclic Tests

Group Tests
  Interaction Tests
  Uniform Amplitude Tests
  Non-Uniform Amplitude Tests (Storm Load)
  Displ.-Controlled Tests
  Load-Controlled Tests
  Non-Uniform Amplitude (Storm Load)
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Chapter 2

LITERATURE REVIEW

2.1 INTRODUCTION

The literature review presented in this chapter covers two different aspects of research. The first deals with the characteristics of calcareous soil, while the second deals with the behaviour of offshore isolated single piles and pile groups. Most of these investigations are associated with the development of theoretical methods for use in pile design. Most of the research conducted on the behaviour of piles in calcareous sediments has only been carried out in the last 20 years. The main reason for this has been to aid the technical needs of companies in the installation of offshore structures. The viability of such structures is therefore highly dependant upon accurate analysis of the underlying soils.

Most geotechnical marine research has been concerned with piles in silica sand and clays which were discovered early in the North Sea and the Gulf of Mexico. However, the exploration for energy sources in regions consisting of calcareous soil has now grown to include many areas in the world including Australia, the Philippines, India, the Middle East, the Gulf of Mexico and Brazil. Many papers on
this topic have been published in several international journals, and a large number of these publications were presented to the International Conference on Calcareous Sediments, Perth, Australia, 1988.

In this chapter, background information on the general nature, classification and properties of calcareous sediments is reviewed as well as laboratory preparation of sand beds for model testing. The characteristics of the soil deposited, such as its density and grading are also outlined. A review is given of the shear strength characteristics of soils and the soil–pile interface response. Since some of the laboratory studies in this thesis were performed using a direct shearing device, this type of testing will be discussed in section 2.5. An overview is then given of the behaviour of piles installed in marine calcareous sediments, which will include the installation of piles, and the behaviour of single piles and pile groups under both monotonic and cyclic load conditions.

2.2 NATURE AND COMPOSITION OF CALCAREOUS SEDIMENTS

Calcareous or carbonate sediments are generally composed of calcium carbonate in the form of crushed shells and the skeletal remains of sea animals, coral, molluscs and algae. Noorany (1985) described calcareous sediments as being either biogenic in origin or arising from the mechanical erosion of coral ruins and shells by the action of ocean waves. Further descriptions have been made on the nature and origin of carbonate soils by Noorany (1989).

Carbonate sediments are found in the marine environment of the tropical and temperate regions of the world. Within this area between the latitudes of 30° North and 30° South, where large gas and oil platforms are constructed for example, the Gulf of Mexico, southern California, west Florida, Brazil, the coasts of Saudi Arabia and India, South China and Indonesia. Australian offshore platforms are situated in Bass Strait and on the North West Shelf of Western Australia. These sediments exist
in a wide variety of forms such as crystalline limestone, chalks, ooze, silts and carbonate muds.

The early exploration of calcareous sediments in Australia began in Bass Strait before 1970. The fields of Barracouta were discovered in 1964, and Halibut and Kingfish were discovered in 1967. The first geotechnical investigation undertaken of the North Rankin field on the North West Shelf took place in 1974 with the aim of providing information for the design of a gas-producing platform. The location of offshore platforms for gas and oil production in Australia are shown in Figure 2.1.

The proportions of calcium carbonate present in calcareous sediments varies according to the distance from the shore line or the presence of rivers that affect the composition of the sediments deposited on the seafloor. As a general definition, one may classify a soil as calcareous if it contains greater than 30% calcium carbonate. In some calcareous sands, the calcium carbonate content exceeds 90% by weight (Golightly and Hyde, 1988) and those calcareous soils are also known as carbonate sands.

Carbonate sediments originate from remains deposited on the seafloor, from shells and skeletons of marine organisms existing in the upper levels of ocean or from reefs and other bottom-dwelling organisms. Calcareous soils can also arise from the chemical precipitation of carbonate in solution, which can crystallise to form another compound, for example calcium-rich carbonates (like limestone) can be recrystallised to magnesium-rich carbonates (like dolomite). Chemical alteration may arise from small changes in carbon dioxide concentration and water temperature (Fookes and Higginbotham, 1975), or the activity of bacteria. A result of such an alteration is the cementation of sediment grains.

Inshore, the presence of rivers inhibits the formation of carbonate materials due to the decreased salinity of water (Semple, 1988) and the precipitating action of silica from the shore.
In the Australian context, the lack of major rivers favours the formation of calcareous soil with a high concentration of calcium carbonate on its shore lines. Higher concentrations of calcium carbonate may be found in the coasts of Saudi Arabia because that country has no rivers.

The particle sizes and shapes of calcareous sands can vary from being like those of silts to being like those of gravels and provides another distinctive feature to these soils. The finer grains are generally formed at a greater distance from the shore line in layered deposits, while the coarser ones are found in thick sediments closer to the continental shelf.

2.2.1 Comparison of Properties of Calcereous and Silica Sands

The size of soil particles, and the distribution of particles throughout the soil mass, are very important factors influencing the classification of non–carbonate sediments and probably important for carbonate sediments. Several systems of classification of calcareous soils have been based on grain size and geologic history of the deposit, e.g. Fookes and Higginbottom, 1975; Clark and Walker, 1977; King et al, 1980; Noorany, 1989. Semple (1988) consider particle shape, surface texture, presence of intra–particle voids and thin–walled particles, as important characteristics.

The nature of the particles in calcareous soil leads to a high friction angle and high in–situ void ratio, consequently, high compressibility results. Poulos et al (1982) showed that the reduction in friction angle of calcareous soil with increasing effective stress level is more significant than for silica sand. Datta et al (1980) state that the compressive volume strains which are observed during triaxial shear, result from the susceptibility to crushing of the sand grains. Once any cementation is broken down, the value of compression index for calcareous soil is much lower than for silica soils of the same particle size, at corresponding stress levels (Randolph, 1988a).

Detailed characteristics of calcareous soils such as shape, size of particle, void ratio,
porosity, cementation, specific gravity, and carbonate content are given by Demars et al (1976) and Poulos (1988a).

It was previously thought that low skin friction for piles in calcareous sands was due to susceptibility to crushing of the soil particles and random cementation (Datta et al, 1982; Dutt and Ingram, 1990). However according to Semple (1988) the initial void ratio (density) is a very important parameter in determining the behaviour of this type of soil. Semple also reported that if due account is taken of the initial density then the differences between the behaviour of calcareous and non-calcareous sands are less significant.

2.3 MICROSCOPIC EXAMINATION

A useful classification tool for calcareous soil is microscopic examination (Datta et al, 1982). The characteristics of various types of calcareous sediments which were discussed in previous sections have been examined microscopically by several researchers (Price, 1988; Allman, 1988; Lee, 1988; Boey, 1990) utilizing the scanning electron microscope technique. Microscopic examinations have been performed on the New North Rankin (N.N.R) calcareous sand by Allman (1988), while an examination of the other types of sediment has been carried out by Lee (1988) to make a comparison between the particle characteristics of different types of sediments.

Details of the component particles of N.N.R. calcareous sand presented by Allman (1988) are shown in Figure 2.2. Figure 2.2a shows a group of foraminifera which are highly variable in nature. They typically have a thin-walled structure, with high intra-particle void content, and are consequently highly susceptible to crushing. Very often, the foraminifera are partially shattered and infilled with debris. The foraminifera particle is well-rounded in shape and has a smooth surface texture. Figure 2.2b shows a fragment of bryozoan with a cellular structure, which is
partially infilled with debris and is angular in shape. Figure 2.2c shows a gastropod which has intra–particle voids and a relatively smooth surface texture. Figure 2.2d shows two coccoliths attached to a large particle. These coccolith particles are relatively small and composed of circular plates. Figure 2.2e shows a pteropod fragment which is of bioclastic origin. It has a delicate hollow rod–shape structure and smooth surface texture. The echinoderm fragment shown in figure 2.2f also has a hollow rod shape structure. This fragment has a rougher surface texture and stronger structure.

Further microscopic examinations have been carried out on different grading fractions of the N.N.R. sand by Boey (1990) and are shown in Figure 2.3. Figures 2.3a and 2.3b show small particles which are nearly round in shape and appear to have been obtained from the disintegration of larger particles. The large particles in Figures 2.3d, 2.3e and 2.3f are composed of hollow–tubed petropods showing the presence of intra–particle voids.

2.4 DEFORMATION & STRENGTH BEHAVIOUR OF CALCAREOUS SEDIMENTS

The deformation and the shear strength parameters for calcareous sediments are usually deduced from laboratory tests such as triaxial and direct shear tests which allow close control over test conditions. In this section, the compressibility, stress–strain behaviour, and shear strength properties for calcareous sands are reviewed. The objective of this section is to review the properties and behaviour of uncemented calcareous sands deduced from laboratory direct shear and triaxial tests.

2.4.1 Compressibility

When soil is subjected to load, the individual soil grains are compressed and the intergranular voids are forced to change shape and volume. For non–carbonate sands the deformation of the mineral grains themselves is small enough to be neglected.
The amount of compression from this process depends on the amount of pore fluid (voids) existing in the soil mass and the rigidity of the soil skeleton, both of which are functions of particle size, shape and texture.

In general the compressibility of granular soil is due to the following factors:

a) the orientation and distribution of the soil particles,

b) the rearrangement of soil particles and the interaction due to loading,

c) the elastic deformation of the soil,

d) the soil void ratio,

e) the crushing of the particles, and

f) the breaking of cohesive bonds.

The last three factors are more significant than the others when studying calcareous soils as opposed to non-carbonate sands (e.g. Nauroy and Le Tirant, 1983; Semple, 1988; Murff, 1987; Golightly and Hyde, 1988). This is because calcareous sediments exist at a high void ratio, have high intra-particle voids, and have grains which are easily crushed under relatively low pressures. The cohesive bonds between grains depend on the degree of cementation. Allman (1988) conducted several static and cyclic loading tests on artificially cemented soils, and observed that the cohesion due to cementation affects the rate of volume change during shear tests. Partial soil cementation can also reduce lateral stresses (Noorany 1985) as pile vibration could create gaps between the pile and soil, producing lower interface stresses.

The conventional analysis of the bearing capacity of foundations is based on the assumption of the incompressibility of soil, but it has been recognized that the theory should be altered to consider the compressibility of soil. Terzaghi in 1943 suggested the use of the same bearing capacity equation and factors but with the
reduced strength parameters $C^*$ and $\varphi^*$, These were defined as:

$$C^* = 0.67C,$$  \hspace{1cm} (2.1)

and $$\varphi^* = \tan^{-1} (0.7\tan\varphi)$$  \hspace{1cm} (2.2)

where $C$, $\varphi$ are the cohesion and angle of internal friction of the soil.
Vesic (1970) extended this approach to include the effect of relative density and recommended that the reduction factor of 0.67 should be replaced by a correlation factor depending on the relative density $D_r$:

when $0 < D_r < 0.67$

Correction Factor = $0.67 + D_r - 0.75D_r^2$  \hspace{1cm} (2.3)

The last decade has seen several studies of the compressibility of carbonate soil and its effects on the bearing capacity of offshore foundations (Nauroy and Le Tirant (1983 and 1985); Nauroy, Bruy and Le Tirant (1985, 1987 and 1988); Murff (1987); Poulos (1988a); Poulos and Chua (1985); Semple (1988)).

Vesic (1972) proposed that the bearing capacity of a pile could be predicted using Vesic's spherical expansion theory. Poulos and Chua (1985) found some merit in the theory which incorporates the friction angle ($\varphi$) and plastic volume strain characteristics of soil.

The compression index of calcareous sediments may be determined by using a one-dimensional compression test, or isotropic compression tests. One-dimensional compression tests include the conventional one-dimensional oedometer test, and the $K_0$ triaxial consolidation test. Poulos et al (1984) examined the compressibility, coefficient of consolidation and the creep behaviour of calcareous soil from Bass Strait sand. The results indicated that the compression index $C_T$ increases with increasing void ratio for the same level of effective stress as shown in Figure 2.4. The values of the compression index of calcareous soil are much greater than those
of silica sands. They consider that the values of $C_1$ are similar to the values for many clay soils.

The coefficient of secondary compression $C_\alpha$ is defined as the rate of change of void ratio per log cycle of time. The $C_\alpha$ value of Bass Strait calcareous soil is generally of the order of 0.003, and is found to increase with increasing mean effective stress. Poulos (1988a) derives a description of creep characteristic, an empirical expression for the relationship between $C_\alpha$ and $C_C$, roughly estimating the value of $C_\alpha$ as:

$$C_\alpha = 0.00077 \log_{10} \sigma'_v - 1$$  \hspace{1cm} (2.4)

where $\sigma'_v$ is effective stress.

Consequently the ratio of $C_\alpha/C_C$ for these sediments ranges between 0.01 and 0.03 and this proportion tends to decrease with increasing initial void ratio.

The tests conducted by Young (1983) indicated that the coefficient of consolidation $c_v$ of Bass Strait calcareous sand decreases as effective stress increases. The values of $c_v$ are much less than those for silica sand and lie within the range 0.15 mm/min to 0.020 mm/min.

Data from the $K_0$ apparatus described by Davis and Poulos (1963) shows that the coefficient of earth pressure at rest $K_0$ is about 0.3 for the soil in its normally consolidated state. This $K_0$ value can be compared with values of $K_0$ obtained from the early proposals made by Jaky (1944) based upon the angle of internal friction ($\varphi$) of the soil:

$$K_0 = 1 - \sin \varphi$$  \hspace{1cm} (2.5)

The values of $\varphi$ depend on the initial density range, between 30° to 50°, and are obtained through several direct shear tests. The values of $K_0$ from the above
equation lie between 0.23 and 0.43, and these values show reasonable agreement with those from the $K_0$ consolidation test (Poulos, Uesugi and Young, 1982).

2.4.2 Deformation and Strength from Direct Shear Tests

The characteristics of soil subjected to static and repeated loading have received significant attention in experimental and theoretical studies. One experimental method of obtaining those characteristics is to perform static and cyclic shear tests by using the direct shear apparatus under both constant normal stress conditions (Potyondy, 1961; Suklje and Brodnik, 1963; Kulhawy and Peterson, 1979) and constant normal stiffness conditions (Williams, 1980; Lam and Johnston, 1982; Ooi and Carter, 1987; Boey, 1990). It has been suggested by several of the latter investigators that direct shear tests can be used to evaluate the development of soil–pile interface stress.

Potyondy (1961) conducted an extensive number of static skin friction tests with construction materials (steel, concrete and wood) on various soils. He found that the change of the skin friction is a function of soil grain distribution, moisture content, normal load, type of construction materials and the characteristics of the surface finish.

Data from static friction tests with smooth and rough concrete blocks against sand and gravel (Suklje and Brodnik, 1963), showed that the coefficient of skin friction $\mu_s$ between a smooth concrete surface and gravel is less than that between the same concrete surface and sand, and $\mu_s$ for gravel sheared on the rough surface is greater than for the same soil sheared on the smooth concrete surface. Data obtained by Yoshimi and Kishida (1981a and 1981b) revealed that roughness correlates well with the coefficient of friction $\mu_s$, and that $\mu_s$ was independent of the sand density. The results of the tests conducted by Uesugi (1987) showed that the coefficient of friction is affected by the roughness of the construction material surfaces, the roundness of the sand particles and the angle of internal friction of the soil.
In calcareous sands, the results of static shear box tests reported by Datta et al (1979) showed that the crushing of the particles of calcareous sediments was greater than for silica sand and that the amount of crushing increases with increasing mean effective stress.

Hardin (1985) and Noorany (1985) indicated that the stress–strain behaviour of carbonate soil is highly influenced by the degree of crushing which occurs during loading and deformation.

For cyclic tests, a new direct shear device for static and cyclic shearing of the soil–structure material interface has been developed by Desai et al (1985). Tests have been conducted on loose silica sand ($D_v=15\%$) and dense silica sand ($D_v=80\%$) at various values of, normal stress, rate of loading, displacement amplitude ($\rho_o$), and number of cycles ($N$). For both densities, the shear stress increases with increasing normal stress and with increasing number of cycles. For various values of ($\rho_o$), the results showed that there was no significant difference in static responses, but the cyclic shear stress was influenced by the higher displacement amplitudes and rates.

Repeated loading can produce a greater deformation than that produced by an equivalent static shear stress applied to soil deposits. The deformation increases with increasing number of cycles. However, the normal displacement of calcareous sands is much larger than that of silica sands for the same load condition. Poulos et al (1982) have found that calcareous soils are significantly more compressible than silica sand.

The particles of calcareous soil are prone to crushing and are highly compressible during shear. This has induced several investigators to study the effects of particle damage of carbonate soil on strength and shear strain behaviour (e.g. Datta et al, 1979; Semple, 1988; Hardin, 1985). Finn et al (1982) attribute the increasing shear strength during cyclic loading to the strain–hardening that is accompanied by dilation. In contrast to the behaviour of silica sand, the results from cyclic simple shear tests
conducted by Morrison et al (1988) show that the shear resistance of carbonate and calcareous soils reduced rapidly after the first cycle and then reached a fairly steady state after a few cycles.

2.4.3 Deformation and Strength from Triaxial Tests

Triaxial tests have been used widely to determine the mechanical properties of soil, for example the stress–strain relations, the volume change, the shear strength and pore pressure behaviour, under static load conditions. However, triaxial tests can also be used to determine the soil properties under cyclic loading (e.g. Seed and Lee, 1966). The purpose of this section is to review the static and cyclic behaviour of carbonate sediments under triaxial test conditions.

For static tests, the soil behaviour changes from that of a dilatant material to that of a more plastic material exhibiting volume reduction during shearing, as the confining pressure increases. Allman and Poulos (1988) found that the transition from brittle to ductile behaviour of uncemented N.N.R calcareous sand occurred with increasing confining pressure. They also found that the peak friction angles for the same sand were 35.2° and 34.6° for medium–dense and dense samples. However, the peak friction angle of uncemented calcareous sands has been studied by (Datta et al, 1980; Noorany, 1985; Airey et al, 1988; Hull et al, 1988) and was found to lie in the range 40° to 50°. Airey et al (1988) and Hull et al (1988) show that the stiffness of the different types of calcareous sediments increases with increasing confining pressure.

Published data on stress–strain behaviour of uncemented calcareous sands were summarized by Semple (1988) and are shown in Figure 2.5. These results show a tendency for increasing volumetric strain with increasing initial void ratio. Similar trends have been noted for cemented calcareous sediments as well as for silica sand.

The secant soil modulus at a load level 25% of peak is represented as $E_{25}$. The
normalized drained Young's modulus $E_2/\sigma_c$ variation with initial void ratio for both carbonate and silica sands can be obtained from the data by Hull et al (1988) and is shown in Figure 2.6.

The high compressibility of carbonate sediments under low confining pressure loading has been noted from triaxial tests conducted by several researchers (i.e. Angemeer et al, 1975; Semple, 1988; Airey et al, 1988; Golightly and Hyde; 1988).

The results of extensive series of triaxial tests on natural calcarenites and sands from the North Rankin site are summarized by Carter et al (1988). They found that the stress–strain relations for cemented and uncemented samples were generally similar, when the samples were consolidated beyond the apparent preconsolidation pressure. These relations are illustrated in Figure 2.7. They also found that the strength of uncemented sands decreased with increasing number of cycles, and that a higher cyclic stress level was required to cause failure for the cemented soil than for the uncemented soil.

The behaviour of different types of calcareous sediments under cyclic loading triaxial tests has been presented by Kaggwa et al (1988). They observed that the resistance of two sands from Bass Strait to cyclic loading is generally higher than the sand from the North Rankin site. These differences may be attributed to the observed differences in the static behaviour of these sands. Fahey (1988) conducted several cyclic loading tests on isotropically consolidated samples and showed that, under cyclic loading, failure depends on the cyclic stress ratio, but does not appear to be influenced by preconsolidation and cementation. The relationship between the rate of generation of pore pressure and the number of cycles that caused failure has been studied by Fahey (1988) and Kaggwa et al (1988).

The behaviour of non–cemented and cemented calcareous sands subjected to large numbers of cycles has been studied by Airey and Fahey (1991). The results of the
tests have been presented as so-called "S−N" (or fatigue) curves which relate the cyclic deviator stress normalized by static strength (S) to the number of cycles to cause failure (N). The normalized cyclic stress reduces with increasing numbers of cycles.

2.5 PREVIOUS TESTING USING DIRECT SHEAR DEVICE

The direct shear box device is probably the oldest type of soil shearing test. It is still considered one of the simplest and most useful tools for determining the shear strength parameters of soil, which are very important for the design of pile foundations. In this thesis, the development of this device has been performed in order to carry out cyclic shearing tests as one part of the experimental program. Thus, it is useful to review previous testing using the direct shear apparatus.

The shear box apparatus was first used more than 200 years ago when Coulomb used a type of shear box test to determine the characteristics of shear strength of the soil. Collin (1846) employed a double shear apparatus to measure the shearing strength of soil. The principal theory of this device is to shear the specimen, which has been placed in the two halves of the shear box, by either pulling or pushing one part horizontally while the other remains fixed. A normal load is applied to the top of the specimen.

The development of the device has involved many modifications to the dimensions of the shear box, the type of shearing load, and the type of normal applied load. However, the basic principles underlying the shear box have not changed throughout its development.

The problem of non-uniformity of shear stress and strain pattern over the shearing surface of a specimen has been observed by several investigators (e.g. Terzaghi and Peck (1948), Hvorslev (1960) and Scarpelli and Wood (1982). Despite the
disadvantages mentioned above, the direct shear test remains popular in the field of geotechnical engineering because it is the simplest shearing device; setting up a specimen is a simple procedure and the results obtained from this apparatus are generally in agreement with those obtained using more sophisticated equipment.

2.5.1 Development of Shear Tests to Investigate Pile Friction

Previously, the direct shear device has usually been used for tests on soil and for interface behaviour tests under monotonic load (Terzaghi and Peck (1948), Roscoe (1953), Hvorslev (1960), and Potyondy (1961)). However, during earthquakes or under the influence of ocean waves the soil is subjected to cyclic loading. Hence, to simulate soil behaviour under these conditions, the shear box has been modified (Desai et al, 1985) to perform cyclic shear tests.

Further modification to the shear box was made by Uesugi (1987), to investigate interface behaviour. With his apparatus it was possible to separate the components of displacement due to the sliding interface from displacement due to the deformation of the sand specimen, which is not possible in a standard direct shear test. The results of Uesugi show that the coefficient of friction is affected by the roughness of the interface surfaces, the particle shape and size and the angle of internal friction. The effects of cyclic loading on the shear resistance of silica sand were observed by Desai et al (1985) and Uesugi (1987), who found that the shear resistance increases with increasing number of cycles.

Other types of shear apparatus (simple shear, ring shear and rod shear) have been used to perform static and repeated loading tests and to investigate pile friction. A number of static and cyclic simple shear tests were conducted by Morrison et al (1988) to investigate the characteristics of the carbonate and calcareous soils from offshore Africa under conditions of constant volume. They adopted the classification system recommended by Clark and Walker (1977) which is dependant on the
carbonate content of the soil.

The soil tested is described as a carbonate, where the carbonate percentage is greater than 50% and less than 90% and it is described as calcareous, if the carbonate percentage is in the range between 12%–50%. The reduction factor $D_r$ for carbonate soil (which is defined as the shear stress for a specified cycle, divided by the maximum static shear stress), was about 80% in the first cycle, and then dropped to the range between 8%–18% for the twentieth cycle. The degradation factor for calcareous sand was more than 80% for the first cycle and dropped to 60% for twenty cycles.

The second type of direct shear apparatus is the Ring shear apparatus. The advantage of this apparatus is that the contact surface area remains constant even for a large shear displacement. This apparatus was employed by Yoshimi and Kishida (1981b) to evaluate the friction between soil and a metal surface under static loading conditions. The tests showed that the interface friction is primarily governed by roughness of the metal surface, irrespective of the density of sand.

Desai et al (1985) conducted several static and cyclic interface tests by using a ring shear apparatus to make comparisons with results obtained from direct shear tests. The results of cyclic tests obtained from the ring shear apparatus were similar to those obtained from the shear box.

A rod shear device was used by Brumund and Leonard (1973) to perform static and dynamic skin friction tests in sands of different grain size using rods of different surface roughness. The static shear tests show that the interface friction angle depends not only on the surface roughness of the material, but also on the angularity and roughness of the sand grains. In the case of crushed quartz sand sliding on a rough pile surface, the interface friction angle equalled the internal friction angle, which implies that failure occurred in the soil. This conclusion is
similar to that stated by Kulhawy and Peterson (1979). However, it is not clear precisely where in the soil the failure occurred. Bea and Doyle (1975) used a similar device in their series of tests to study the effects of various parameters on the skin friction of piles in clay.

Researchers at Sydney University developed a modified rod shear test in which an equal consolidation pressure was applied to both the top and bottom of the soil, and the circumference was restrained. Some early work with this device was carried out by Uesugi. Together with the flexible upper and lower boundaries provided by rubber membranes, the rod adequately simulated a typical pile shaft element (Chan, 1986; Lee, 1988). A further development of this idea was made by Jewell and Randolph (1988), in which the soil surrounding the rod was consolidated in three dimensions.

However, consideration of the shearing adjacent to a pile suggests that the normal stress is indeed affected by the soil compressibility. Randolph (1988a) reported that the lower values of shaft resistance in compressible calcareous sand can be attributed to the crushing and compaction of the sand during driving of the pile which causes shearing of the soil. The effects of both crushing and compaction may result in reduced radial stresses and hence, the skin friction may be reduced.

A direct shear apparatus has been modified to perform static and cyclic shear tests under constant normal stiffness (CNS) conditions. The selection of stiffness is discussed by (Williams, 1980; Lam and Johnston, 1982; Boyce and Kulhawy, 1983; Ooi and Carter, 1987; Boey, 1990; Airey et al, 1990). They have suggested that CNS direct shear tests can simulate the development of interface stress between a pile and the surrounding soil or rock.

2.5.2 Relation of Direct Shear Test to Pile Skin Friction

Model pile tests performed in sand using different materials for the piles (concrete,
steel and wood) have indicated that the pile shaft resistance can be related to the surface roughness (Yoshimi and Kishida, 1981a).

Young (1983) conducted two series of static and cyclic pile tests to investigate the characteristics of skin friction on jacked concrete and aluminium piles in silica and calcareous sands. His results showed that the skin friction of concrete piles embedded in both sands was significantly higher than that of aluminium piles at the same overburden pressure. It was also found that the degradation factor for skin friction of the concrete pile in both sands was significantly less than that of a driven aluminium pile. Young concluded that this difference could be attributed to the high surface roughness of the concrete pile. Several grouted concrete pile tests were also conducted by (Nauroy et al, 1983; Young, 1983; Allman, 1988; Lee, 1988; Randolph, 1988a).

Significant degradation in skin resistance has also been produced by cyclic load applied to piles. Field tests (Angemeer et al, 1973; Semple, 1988) and model pile tests (Poulos and Chan, 1986; Randolph, 1988a) confirm these findings. However, the drop-off in shaft resistance is much greater than observed in cyclic tests under constant normal load (CNL) conditions.

In the case of a pile grouted into a cemented soil or rock formation, the normal stress at the interface during shearing will be altered by dilation or contraction due to the radial movement of the rock or soil surrounding the pile. For this reason other researchers have carried out shear tests under conditions of constant normal stiffness (CNS) (Johnston et al, 1987; Ooi, Carter and Boey; 1988).

Boyce and Kulhawy (1983) conducted several static direct shear tests on dense silica sand under CNS conditions. They found that the increase in normal stress obtained by CNS tests was similar to that which could be expected in pullout tests on shafts in dense sand which were conducted by Stewart and Kulhawy (1981). Airey et al (1990) conducted several cyclic direct shear tests on Bass Strait calcareous sand
under CNS conditions. These tests showed a rapid drop in shear stress during cyclic shearing.

2.6. METHODS OF RAINING SAND IN LABORATORY MODEL TESTING

One of the most important considerations in experiments with both small and large scale model piles is to reproduce homogeneous sand beds and controlled density, since the scale of experimental work is sensitive to variations in density. The technique of raining sand in the laboratory is reviewed here because model pile testing is a very important part of this thesis.

In the past decade, geotechnical engineering studies have shown that the strength and deformation properties of cohesionless soils subjected to static and cyclic loading depend not only on the density of soil but also on the fabric characteristic which is a function of deposition. These findings have been reported by many researchers (Kolbuszewski and Jones, 1961; Oda, 1972; Ladd, 1974; Kozera, Kulhaway and Wittem 1977, Holtz and Kovacs, 1981; and Miura and Toki, 1982). Pioneering experimental work on the maximum and minimum porosities of sands was carried out in 1948 by J.J. Kolbuszewski, who stated that for a specific height of drop, the density decreases with increasing intensity of raining and increases with increasing height of drop.

Two series of tests were conducted on silica sand dropping through water. The first series involved pouring sand into a vessel holding natural water (water containing dissolved air). The second series involved pouring sand through water lacking dissolved air, employing a special device which absorbed the air from water. He found that a higher sand density was obtained in the second type of test. However, the difference between the results obtained by rapid pouring sand through air and through water were not very large. The latter gave somewhat higher porosity and probably resulted in the closest approximation to the loosest packing state.
Lambe and Whitman (1979) reported that tests to determine minimum density and those to determine the maximum density incorporate some kind of vibration. They concluded that the values of density depend on the procedure used to determine them.

A number of methods have been developed to produce a homogeneous sand bed with a specified density. These methods may be divided into the following categories:

a) methods which control the density after placing the sand, such as vibration, tamping or rodding, eg. Hanna (1963), Vesc (1969), Bieganousky and Marcuson (1976), and Miura and Toki (1982).

b) methods which control the density during pouring of sands into water containing dissolved air and water without dissolved air, eg. Kolbuszewski (1948), and Kolbuszewski and Jones (1961).

c) methods which control the density by raining dried sand into a container, e.g. Bagnold (1954), Bieganousky and Marcuson (1976), Augenstein and Hogg (1978), Swane (1983), Hunger and Morgenstern (1984), Sweeney and Clough (1990) and several papers by Kulhawy and his co-workers (e.g. Weller and Kulhawy, 1978; Kulhawy et al, 1979; Trautmann, O'Rourke and Kulhawy, 1985a and 1985b).

The first group of methods is convenient for producing only dense sands. Feda (1961), states that one of the characteristics of a sand bed prepared by these methods is that it is strongly anisotropic. The second group of methods is suitable for achieving both medium and loose densities. The sand bed produced by pouring sands into water without dissolved air is more homogeneous than a sample obtained by pouring into water containing dissolved air. This is because the latter one will be disturbed by the air bubbles (Kolbuszewski, 1948). Lee (1988) conducted several tests by slowly sedimenting calcareous sand into water to obtain 40% relative density of
soil beds in the test vessel.

The third group of techniques is classified into two types. The first involves raining the sand by using an air-activated sediment spreader, which varies the air pressure to obtain different rates of deposition (Butterfield and Andrawes, 1970). The second type involves raining sand by gravity force through the apertures of the rainer. This second type of raining has been adopted in the present study. By this method the placement density of the sediment bed is mainly controlled by two factors, the intensity and the velocity of the sand particles. However, the density increases with velocity and decreasing intensity of raining, where raining intensity is defined as the rate of sand discharge.

Miura and Toki (1982) achieved high densities of sand having a relatively small uniformity coefficient by using a multiple sieving pluviation apparatus (msp method). A sand mass formed by raining presents similar behaviour to that of normally consolidated sand deposited in a marine environment.

It was found that the method used by Bieganousky and Marcuson (1976) provided a highly uniform sand bed. They employed three kinds of raining apparatus (a rotating rainer, a single hose rainer and a circular perforated plate rainer) to obtain a uniform density distribution within a 1.2 m diameter and 1.8 m long container. They found that the last device was more satisfactory than the others and that the density along the container's radius was constant when the height of drop was 560mm. They also observed that the sand density increased with increasing height of drop.

Swane (1983) prepared homogeneous silica sand beds in a steel box container by using a sand raining device. The base of the device consisted of a fixed plate overlying a movable plate. Both base plates were perforated with matching 4.7mm diameter holes. The velocity of sand raining was controlled by varying the height of
fall. Swane conducted two groups of tests to determine the relationship between dry
density and height of fall. The first group of tests was performed without an
attachment. The density of the sand increased from 1.43 to 1.63 t/m³ as the height
of fall increased. The second group of tests was performed with a mesh attachment
to let the sand fall in a more diffused state, therefore resulting in a more uniform
sand bed. In this group of tests the maximum density of the sand bed increased to
1.65 t/m³. This sand rainer was used only for silica sand.

2.7 EXPERIMENTAL DATA ON AXIALLY LOADED PILES AND PILE GROUPS

This section reviews laboratory and field tests on piles and pile groups embedded in
different types of soil, and particularly in carbonate sands. In this section, the
behaviour of piles during installation, the resulting residual loads, and the behaviour
during static and cyclic loading, will be discussed through published experimental
data.

2.7.1 Installation and Residual Load

The process of installation of driven piles leaves a self-equilibrating distribution of
residual stress imposed on the pile-soil system after all external applied load is
removed. The existence of these residual forces is related to the different rates of
unloading of base bearing and shaft friction (Cooke et al, 1979), while the pile
stiffness and the reconsolidation of the soil are also influential in determining the
distribution and magnitude of the residual load.

In many previous studies, the residual stress due to installation of driven piles, was
not taken into account. This residual stress was accounted for by Hunter and
Davisson (1969) when they conducted compression and tension tests on piles in sand.
Tension loading was applied, immediately followed by compressive loading. They
observed significant tensile stress at the pile tip after the tensile load was released.
It was assumed that there was no tensile stress at the pile tip after releasing the load from the pile. Therefore they suggested that the unexpected residual stress due to the compressive loading tests had not been taken into account in their interpretation of subsequent tension tests. Hunter and Davisson concluded that unless the residual load was taken into account, the ultimate compressive and tensile capacity of the pile could be in serious error. Similar conclusions were reached by Holloway et al (1979).

The phenomenon of residual load was considered theoretically and experimentally by several investigators (Hanna and Tan, 1971; Holloway et al, 1978 and 1979; Poulos, 1987; Poulos and Chan, 1988; Houlsby et al, 1988). Hanna and Tan (1971) emphasised that the results of load tests are markedly affected by boundary stress conditions in the tests. Thus the boundary stress conditions which are imposed on the piles have to be examined prior to analysis of the pile test data. Further studies of the influence of residual forces on the axial response of piles were reported by Holloway et al (1979).

The time between 1979 and 1981 was when design of piles for the North Rankin platform was carried out. In this design, the behaviour of calcareous soil was assumed to be similar to silts and silica sands (Dolwin, Khorshid and Goudover, 1988). The lack of design experience was acknowledged and it was decided to carry out a field test to monitor the behaviour of an instrumented pile during driving. The test results indicated firstly that the carbonate sand behaviour was different from that predicted. Analysis of data indicated an average unit skin friction of 15 kPa and a total end resistance of 24 MN at the start of driving. These values indicated residual skin friction after driving in the order of 25% of the peak value, which is consistent with laboratory tests conducted by the same authors.

The behaviour of model piles during installation was reported by Poulos and Chan (1988), who were able to examine the development of stresses in the pile during
jacking and the subsequent residual stresses and forces remaining in the pile after jacking. Figure 2.8 shows typical results for a 25 mm diameter pile during jacking. The maximum total jacking force occurred at a penetration of approximately 75 mm and then decreased slowly until it became more or less constant at about 250 mm penetration. Similar behaviour was seen for the end-bearing force. The average skin friction dropped sharply with increasing penetration, from a value of about 100 kPa at 75mm penetration, to about 10 kPa at the end of penetration.

This severe "strain-softening" behaviour appeared to be related to the reduction in normal stress on the pile-soil interface as pile penetration proceeded, this reduction being in turn related to the tendency for volume contraction of the soil on shearing. The installation force increased non-linearly with increasing overburden pressure. This suggested that the end-bearing capacity also might not increase linearly with overburden pressure. This conclusion is also consistent with the model footing results reported by Chua (1983).

Figure 2.9 shows the residual load distribution along the piles subjected to 200 kPa overburden pressure, immediately after removal of the jacking forces and after a three hour rest period. A significant residual force remained at the pile tip, and this force increased over the three hour period. This increase implied an increase in residual shaft friction with time.

Poulos (1987) presented a simple analysis for estimation of the initial residual stresses in a driven or jacked pile, and then examined the influence of these stresses on the static axial load-settlement of the pile in three idealised soil profiles; soft clay, stiff clay, and medium-dense silica sand. Some solutions were also given to indicate the effect of residual stresses on the cyclic response of a pile in silica sand. The studies indicated that the effects of residual stresses were most significant for piles in sands and least significant for piles in soft clay. Under cyclic loading, the existence of residual stresses tended to reduce the shaft capacity and pile head stiffness.
It is now well accepted that if the residual load in the pile is ignored, the results of the field and laboratory test may be in error.

2.7.2 Static Loading Tests

a) Model Single Pile tests

In silica sand or terrestrial cohesionless material, the process of driving can increase the density of sand around and underneath the piles (Robinsky and Morrison, 1964; Vesic, 1967; Meyerhof, 1975) because of compaction, rearrangement and crushing of the sand particles. The above investigators also concluded that the compaction depends on the properties of the material, grain size, and shape of particles. Kishida (1967) proposed a theory to estimate the increase in ultimate bearing capacity of piles in a group driven into loose sand. The theory included the change of the internal friction angle of sand due to driving. It was found that the increase in skin friction on the shafts of driven piles could be accounted for by an increase in the coefficient of lateral earth pressure, from the at-rest value $K_0$ to a higher value.

In calcareous sand, there are extensive data on model pile tests in uncememted calcareous sand (Nauroy and Le Tirant, 1983 and 1988; Young, 1983; Poulos and Chan, 1988; Poulos and Lee, 1988; Levacher 1988), and on cemented calcareous sands (Allman and Poulos, 1988; Houlsby et al, 1988). Laboratory data on driven pile tests in calcareous sand here show how the skin friction compares with that for piles in silica sands (Nauroy et al, 1983; Poulos et al, 1984; Young, 1983; Chan, 1986; Morrison et al, 1988). Nauroy and Le Tirant (1983) explain that the high compressibility of calcareous sands is the principal parameter influencing the skin friction resistance $f_s$. However, other parameters such as grain shape, grain size distribution, and density play an important role in the determination of $f_s$. Semple (1988) utilised the data by Nauroy and Le Tirant (1983) to show the existence of a linear relationship between the void ratio and skin friction.
The capacity of driven piles in calcareous sand has not been found to increase due to the driving process (Angemeer et al., 1973; McClelland, 1974). They attribute the dramatic reduction in the pile capacity to compressibility of the soil, crushing of the soil particles, and the presence of cementation preventing the development of normal pressure acting on the pile shaft. Datta et al. (1980) reported that the effect of the soft nature of calcareous soil may be to reduce the skin friction to a large degree. The volume reduction caused by particle crushing under pile driving may exceed the volume displaced by the pile, with very little radial displacement of the soil adjacent to the pile (Dutt et al., 1986; Dutt and Ingram, 1990).

Data from model pile tests on calcareous and silica sands conducted by Nauroy and Le Tirant (1983) provided measurements of lateral stress created 50mm from the pile face. They found that the increase in lateral stress was very small or negative in calcareous sand, while in silica sand large lateral stresses were created. They concluded that the reason for the minimal stress increases in calcareous sand was the compressibility and crushing susceptibility of the sand.

Direct shear tests on steel–calcareous sand interfaces were carried out by Noorany (1985), who reported that the low skin friction of steel piles driven into calcareous soil is caused by low normal effective stresses at the pile–soil interface rather than by small pile–soil friction angles. The value of the coefficient of lateral earth pressure K was found to be dependent on the compressibility of the sand and type of pile considered (open or closed-ended pile) (Nauroy and Le Tirant, 1985). The value of lateral stress coefficient K changed markedly during the pile loading process (Boulon and Foray, 1986); they also found that K decreases with increasing density and with increasing depth.

The phenomenon of high compressibility of carbonate soil causes low end bearing capacity \( f_b \) and skin friction \( f_s \) of the pile. In spite of this, the soil exhibits a high critical effective friction angle \( \varphi_{cv}' \). The empirical values of \( f_b \) and \( f_s \) are
correlated with the limiting compression index $C_T$. This is obtained by several isotropic triaxial compression tests at an effective stress of 800 kN/m$^2$ and are represented in Tables 2.1 after Nouray et al (1986). Poulos and Chua (1985) found that the end bearing capacity $f_b$ of piles in calcareous sediments was only about half the value of that of silica sand for a comparable relative density.

The effect of overburden pressure on the static skin friction of a jacked pile in both medium dense and dense calcareous sand has been investigated by Poulos and Lee (1988) and Poulos and Al-Douri (1992). They have shown that skin friction in both types of sands increases with increasing overburden pressure as shown in Figure 2.10. Similar observations have been made by Poulos and Chan (1988).

The effect of the pile tip condition of driven piles on the pile response has been examined by Levacher (1988). He has shown that the friction capacity for the open-ended steel piles is lower than that for the closed-ended piles with a conical-shaped toe. Furthermore, it was shown the open-ended piles had a higher end bearing capacity than the closed-ended piles.

The effect of particle crushing on the behaviour of driven model piles in medium-dense and dense calcareous and silica sands has been studied by Lu (1988). He found that the pullout pile resistance increases with increasing fines content for dense and loose silica sand, and loose calcareous sand. However, this resistance decreases with increasing fines content for dense calcareous sands.

Poulos and Lee (1988) suggested that parameters such as density, over-consolidation ratio and particle characteristics significantly influence the peak skin friction. Their conclusions have been reported by Poulos (1988a). The influence of cementation on the peak skin friction of jacked piles in artificially cemented calcareous sand has been shown by Allman and Poulos (1988).

In previous literature it was reported that low skin friction for piles in calcareous
sands was believed to be due to the susceptibility of particles to crushing and to random cementation within the soil, but according to Semple (1988) the initial void ratio (density) is a very important parameter in determining the behaviour of this type of soil. He also reported that if due account is taken of the initial density, then the differences between the behaviour of calcareous and non-calcaceous sands are less significant.

From the above literature review, it appears that the skin friction or the shear stress along the pile shaft \( f_s \) in dense silica and calcareous sand is higher than in loose silica and calcareous sand respectively. This is because the dilatancy of dense sand is greater than that of loose sand. For a specific soil density, the value of \( f_s \) for piles in silica sand is higher than that for calcareous sand which is attributable to the higher dilatancy for silica sand. \( f_s \) may be expressed as:

\[
f_s = \sigma_n' \tan \delta = K_0 \gamma z \tan \delta
\]  

(2.6)

where \( K_0 \) = coefficient of lateral stress at rest of the soil
\( \gamma \) = effective unit weight of the soil
\( z \) = depth
\( \delta \) = the soil-pile interface friction angle

"\( \delta \)" for silica sand has been found from direct shear tests and triaxial tests to be less than for calcareous sand, while "\( K_0 \)" for silica sand may consequently be higher than for calcareous sand.

Considering the change in the lateral pressure on the pile shaft during installation and testing, the equation for normal effective stress becomes:

\[
\sigma_n' = \sigma_{n0}' + \Delta \sigma_{ni} + \Delta \sigma_{nt}
\]  

(2.7)

where \( \sigma_{n0}' \) = Insitu lateral effective stress
\( \Delta \sigma_{ni} \) = the change stress due to installation
\[ \Delta \sigma_{nt} = \text{the change stress due to testing.} \]

The two parameters \( \Delta \sigma_{ni} \) and \( \Delta \sigma_{nt} \), which depend on the soil dilation characteristics, play an important role in determining the skin friction developed on the pile wall. It appears that both of these quantities are higher for silica sand than for calcareous soil.

b) Model Pile–Group Tests

Various researchers in the past have studied the behaviour of pile groups in sand through experiments, for example Kezdi, 1957; Hanna, 1963; Kishida and Meyerhof, 1965; Beredugo, 1966; Kishida, 1967; Vesic, 1967; Vesic 1969; Tejchman, 1973; Ranganatham and Kaniraj, 1978; Poulos and Davis, 1980; O'Neill, 1983. The investigations have been concerned mainly with the ultimate bearing capacity and the settlement of groups. It has been observed that most of these tests were performed on small scale model piles.

Beredugo (1966) and Kishida (1967) studied the effects of the order of installing piles in a group on the load distribution. They observed that, in the case of a small applied load on the pile group, the earlier-driven piles carried less than those which had been driven later, but as the failure load of the group was approached, the influence of driving order disappeared and the location of the pile in the group became the important factor. Thus, the piles near the centre carried more load than the pile at the corner. Similar observations were presented by Vesic (1969), who measured the axial load in individual piles during driving and load testing of a group. He also observed that the load distribution was almost uniform for four-pile groups, and non-uniform for nine-pile groups. In the tests performed by Vesic (1967), the piles in each group were installed by jacking them simultaneously into the sand (whole group) and not one after the other.

The tests conducted by Ranganatham and Kaniraj (1978) were to investigate the
interaction between the piles in the group and the consequent influence on the settlement of the group. They found that the installation of an adjacent pile had a significant effect on the behaviour of already-driven piles. The effect was more marked for groups in loose sand than for groups in medium–dense sand. In both types of sand, installation of a pile caused additional settlement of the earlier–installed piles.

O'Neill (1983) has presented a useful summary of the vertical efficiencies for model pile groups in both sand and clay. His figures are reproduced in Figure 2.11. O'Neill noted the following trends from the figure: (a) group efficiency \( \eta \) in loose sands always exceeds unity, with the highest values occurring at spacing–to–diameter ratios, \( s/d \), of about 2, (b) generally, higher \( \eta \) occurred with increasing numbers of piles. (c) block failure affected \( \eta \) only below \( s/d = 1.5 \) to 2.0. (d) \( \eta \) in dense sands may be either greater or less than unity, although the trend is towards \( \eta > 1 \) for \( 2 < s/d < 4 \). (e) \( \eta \) for groups in clay is always less than unity and block failure may occur in square groups at \( s/d \) values of less than about 2. No test data appears to be available in the published literature for pile groups in calcareous sand.

c) Field Pile Tests

A field pile test program has been described by Hunter and Davisson (1969) to determine the driving and design requirements for the piles supporting locks and dams on sand for the Arkansas River Navigation Project U.S.A. They found that the friction force during compression loading was 30% higher than that during tensile loading and the friction distribution near the pile tip was markedly lower than that for other parts of the pile. The purpose of this particular test programme was to measure the load transfer of piles and it was found that the load transfer was affected by residual stress as explained earlier in section 2.7.1.

Recently twelve field tests on different types of instrumented piles embedded in the
calcareous sands located in the Wondo and Cooper Rivers in South Carolina in the U.S.A have been reported by Baus and Ray (1988). The purpose of the tests was to determine the installation resistance, load-settlement behaviour, and the skin friction and end-bearing capacity of the piles, and to compare several different types of piles to determine which type performed best. The predicted capacity was calculated using correlations developed from Schmertmann (1978). These correlations are as follows:

\[ Q_b = 0.537 \, q_p \, A_p \]  \hspace{1cm} (2.8)

where

\[ Q_b = \text{end bearing}; \ q_p = \text{average cone resistance at pile tip}; \ A_p = \text{area of pile tip}. \]

\[ f_s = 0.040 \, q_c \, A_s \ (\text{driven pile}) \] \hspace{1cm} (2.9)

where

\[ f_s = \text{skin resistance}; \ q_c = \text{average cone resistance}; \ A_s = \text{soil-pile shaft contact area}. \]

Figure 2.12 (from Baus and Ray, 1988) shows the comparison between the predicted capacity and the measured capacity. Good agreement was found between the pile capacity obtained from theoretical prediction and the measured load capacity of the piles tested. The ratio of predicted to experimental load capacity ranges from 0.8 to 1.7. From this Figure, Baus and Ray suggested that a reasonable factor of safety to be applied to the predicted capacity is 2.0 to 2.5.

2.7.3 Cyclic Loading Tests

Driven piles have been commonly used as foundations supporting offshore structures due to two principal factors; relative economy, and relatively easy installation.
However, in calcareous soils, these two factors have become less important since the recognition that the low load capacity of driven or jacked piles may cause problems for the design of foundations of offshore structures. Dramatic evidence of such problems materialized in 1982 when some driven piles at the site of the North Rankin platform on the North West shelf of Australia dropped as much as 60m (about 50% of design length) under their own weight. This has caused subsequent investigators to perform laboratory work under a range of test boundary conditions. Programmes of model and field testing of piles in calcareous soils have been undertaken in the last decade to better understand the characteristics of behaviour of piles in calcareous soils (e.g. Nauroy and Le Tirant, 1983; Noorany, 1985; Murff 1987; Poulos and Chan, 1986; Poulos and Lee, 1988; and Randolph, 1988a).

Generally the shaft and end bearing resistances of the piles depend on various factors including the stress history of the soil, the soil friction angle, the interface friction angle and the soil compressibility (Tomlinson, 1977; Poulos and Davis 1980; and Carter, Poulos and Randolph, 1985).

a) Model Single Pile Tests

For single piles, laboratory data on driven piles subjected to cyclic axial loading in calcareous sands are more extensively available than field data. Most publications dealing with the static loading tests mentioned in section 2.8.2, also include the effects of cyclic loading on piles.

Model pile tests have been carried out in uncemented calcareous sand by Poulos and Lee (1988) and in cemented calcareous sands (Allman 1988) for different levels of cyclic loading. They found that after severe cyclic loading the residual skin friction could be as low as 5–10% of the peak value.

Small scale laboratory model pile tests in medium dense silica sand have been carried out by Young, 1983; Poulos and Chan, 1986; Golait and Katti, 1988. They
considered that the reduction (or degradation) of skin friction under cyclic loading was related to overburden pressure, over-consolidation ratio, and the cyclic displacement amplitude.

The influence of cyclic displacement on the degradation factor of skin friction $D_f$ for different types of calcareous sands obtained from model pile tests (Young, 1983), for a 25mm model pile in North Rankin sand and from (CNS) tests (Johnston et al, 1987) have been summarized by Poulos and Lee (1988) and the data are shown in Figure 2.13. This Figure shows the following findings: i) a good consistency in the results of the three types of test, ii) very substantial loss in skin friction is caused by a relatively small cyclic slip displacement, of the order of $\pm 1$ to $\pm 2$ mm.

For load-controlled cyclic tests, Lee (1988) reported that the permanent displacement of grouted model piles increases with mean load level and number of cycles as shown in Figure 2.14 (a and b). The behaviour of model footings is similar, as demonstrated by Allman (1988).

b) Model Pile Group Tests

For pile groups, there have been very few tests carried out in clay and silica sands involving cyclic loading, and no tests appear to have been carried out in calcareous sands. A series of tests on model pile groups in remoulded soft clays has been reported by Matlock et al. (1982). After 80 load cycles of two-way cyclic load in which the 6-pile group was cycled to a displacement of 25% of the pile diameter, the cyclic capacity of the group was reduced to about 34% of the ultimate static capacity. The shear transfer variation among the piles in the group is shown in Figure 2.15. This loss in capacity was much greater than that for single piles.

O'Neill, Hawkins and Mahar (1982) described the results of one-way repeated compression loading of full size piles driven into saturated overconsolidated clay. A 9-pile group and two single piles were loaded to failure in compression three times.
Overall side resistance reduction during compression testing was 23% in the group piles compared to 9% for the single piles. The degradation in side resistance was thought to be due to side shear stress reversals. The successive loadings to failure also had the effect of increasing the tip resistance, particularly in the pile groups.

Moey (1983) carried out two series of laboratory tests on 4-pile groups which were subjected to axial cyclic loading conditions jacked into silica sand and clay. He found that degradation caused by cyclic displacement is more severe for pile groups than for single piles and the efficiency of a pile group is significantly less than that for a single pile. Another feature of Moey's research is that the cyclic displacement is mainly dependent on cyclic load levels.

Laboratory tests on 2-pile, 4-pile and 8-pile groups of instrumented piles in clay under cyclic loading conditions have been carried out by Hewitt (1988). The tests indicated that the post-cyclic load capacity is independent of mean load, and that the accumulated deflection tends to increase as both the mean load and the cyclic load increase. The comparison between the cyclic stiffnesses of single piles and pile groups have been made by Hewitt (1988) and is shown in Figure 2.16. It can be seen that there is substantial decrease in cyclic stiffness of the pile groups as the number of cycles increases. This trend was also found in theoretical analysis of pile groups.

c) Field Pile Tests

Van Weele et al. (1979) conducted tests on full scale piles in medium-dense silica sand subjected to one way cyclic loading. He observed the accumulated displacement due to cyclic loading. He also observed that the failure occurred after cyclic loading, even at a mean load as low as 20%–30% of the static failure load. The failure was characterised by a continued accumulation of displacement with number of load cycles. He explained these observations by suggesting that each load cycle generated its own unique load pattern in the sand–grain skeleton in the form of load
trajectories. While certain grains were affected, some were not involved in the load transfer. Therefore under each cyclic loading, certain particles may have been moved aside or crushed until a new system of equilibrium was obtained. Hence deformations continued to increase with increasing number of cycles. Consequently Van Weele concluded that degradation of base resistance was more severe than degradation of skin friction.

Chan and Hanna (1980) conducted cyclic loading tests on a single pile in dry dense silica sand and broadly confirmed Van Weele's results, but the failure here occurred at a lower number of cycles compared with Van Weele's tests. It was also observed in some tests that there was a stable stage followed by a large displacement of the pile at a large number of cycles. The degree of displacement was found to be highly dependent on the magnitude of the cyclic load, the type of cyclic load and number of load cycles.

In calcareous sands, there appears to have been relatively little field data from which to investigate the factors which influence pile skin friction in such soils, and in particular the effects of cyclic loading. The small number of field tests can be attributed to the difficulties encountered in performing such tests.

Small scale field tests reported by King et al (1980) indicated that cyclic loading may cause a major reduction in skin friction. Murff (1987) summarized the available data from field tests on driven piles in calcareous sands and this summary is represented in Table 2.2. The Table shows different types of piles installed in various types of soil at different sites in the world. The description of the soils and the remarks about the tests were also included in Table 2.2.

The ultimate load capacity of piles in calcareous sand is less than that in silica sand, despite the fact that calcareous sands have internal angles of friction and angles of pile–soil friction greater than those for silica sands (Noorany, 1985). The
unexpectedly low skin friction or shaft resistance of piles in calcareous sand \((f_s)\) was reported firstly by Angemeer et al (1973 and 1975) who found that values of \(f_s\) as low as 9 to 18 kPa existed for piles driven in Bass Strait (Australia) and about 38 kPa for piles on the North West Shelf. Angemeer attributed this to the silty nature of calcareous soil. The value of \(f_s\) was also found to be less than 18 kPa in partially cemented calcareous sand (Dutt & Cheng, 1984) and the reason was thought to be that the cementation may prevent the development of lateral pressures at the pile–soil interface. Similar conclusions were reached by Angemeer (1975) and Nauroy et al (1988).

Figure 2.17 shows recommended design values of skin friction resistance and end bearing resistance for piles in carbonate sand to be 20% and 60% respectively of the values for piles in terrestrial silica sand (Datta et al, 1980).

The reduction of lateral pressure on the wall of the pile has been attributed to changes in soil characteristics after crushing particles (Datta et al, 1980; Dutt et al, 1985). It was suggested that the fines created by particle crushing could move into existing voids in the surrounding sands and provide lubrication between the pile shaft and the surrounding soil. This is in contrast to Nauroy's published data from field tests on the piles in calcareous sands which are summarized by Murff (1987). These results indicate that the capacities of driven piles are very low compared with piles in silica sands and the range of \(f_s\) for calcareous sand is between 20 to 50 % of that for silica sands. The summary of Murff indicated that cementation plays a minor role in determining the driven pile capacity. However, Murff mentioned the importance of uniformity of cementation and soil particles and highlighted the critical role of the soil's susceptibility to crushing in controlling the magnitude of the skin friction.
2.8 GROUTED PILES

The increasing use of grouted piles for offshore structures can be attributed to their high load capacity as compared to driven piles. However, greater experience with driven piles, difficulties in installation and quality control of grouted piles, exacerbated by an offshore environment, and the additional expense of grouted piles, have meant that driven piles have been more commonly used as foundations for offshore structures. A recent method for installation of offshore piles, combining both driving and grouting, the pile, was proposed by Nauroy et al (1986). The purpose of this method was to utilize the ease of installation and lower cost of driven piles, with the greater shaft friction capacity of grouted piles.

Driven piles in calcareous sands have been found to encounter very low driving resistance and to have low static load capacity, as has been described in Section 2.8. Laboratory tests on grouted piles in uncemented calcareous sands have been reported by Nauroy et al (1985), Hyden et al (1988), Poulos and Lee (1988) and Poulos and Al-Douri (1992). Their tests showed the shaft resistance of driven model piles in calcareous sand is markedly lower than that of grouted model piles. More evidence has been found from field tests on grouted piles in calcareous sands that have been reported by Angemeer et al (1975) which indicated that the skin friction of grouted piles is more than three times that of driven piles. Nauroy and Le Tirant (1985) suggested that the high values of shaft resistance of grouted piles could be attributed to the bond between the coarse angular grains along an irregular borehole wall.

Allman (1988) summarized the published data on drilled and grouted piles in calcareous sands from both field and model pile tests. These test data are reproduced in Table 2.3. This Table shows the difficulties associated with foundation design in calcareous sands. The soil descriptions are irregular and great variations exist in carbonate content and degree of cementation. These variations make it difficult, and possibly unwise, to infer relationships between particular test sites, as
was indicated by Murff (1987), who presented a similar summary table for driven piles in calcareous sands which is shown in Table 2.2.

Nauroy and Le Tirant (1985) carried out tests on a 7.85 m long, 0.31 m diameter grouted pile in uncedmented calcareous sand. They found that the measured skin friction value was greater than 100 kPa. Withers et al (1986) conducted tension tests on grouted pile in Bass Strait, Australia, and found that the average skin friction values were greater than 100 kPa. Deane et al (1988) conducted static and cyclic loading tests on a grouted pile in cemented calcarenite at the North Rankin "A" site on the North West Shelf of Australia. They found that the peak static skin friction varied between 300 kPa and 700 kPa and there was significant reduction of about 50% after cyclic loading. The results of cyclic tests (Deane et al, 1988) indicated that the pile behaviour depends on the combinations of mean load and cyclic load levels which are similar to those results measured by Nauroy et al (1985). These field tests have confirmed that the values of skin friction for grouted piles are higher than those of driven piles.

Laboratory tests performed on model grouted piles in uncedmented calcareous sand have been conducted by Lee (1988) and have shown the following features: i) the residual loads developed in the pile are small compared with the static ultimate capacity; ii) the average value of static skin friction was about 96 kPa; this value was about 4 times higher than that from a jacked model pile tested by Chan (1986); iii) the cyclic degradation of skin friction is greatly influenced by the cyclic slip displacement; iv) the load distribution was affected by number of cycles and slip displacement for testing under displacement controlled conditions, while for testing under load controlled conditions, the load distribution was affected by mean cyclic load levels and number of cycles.

Values of static skin friction from model grouted piles tested by Young (1983) were similar to those from tests conducted by Nauroy et al (1985). Young concluded the
degradation of skin friction was severe and seemed to be controlled by cyclic displacements.

Tests on grouted piles in cemented soils reported by Allman and Poulos (1988) have shown a high peak skin friction capacity at relatively small displacements. Softening occurred beyond the peak. Similar observations were made in field tests on piles grouted into calcareous sediments (Withers et al, 1986 and Nauroy et al, 1988). The soil compressibility and cementation were found to influence both the peak skin friction and the degree of softening. Allman (1988) showed the amount of softening for piles in cemented sand was higher than that for uncemented sand.

2.9 THEORETICAL ANALYSIS OF AXIALLY LOADED PILES AND PILE GROUPS

In past years, piles have usually been designed using empirical methods which have evolved from previous engineering experience. The empirical methods which provide a solution for the response of single pile and pile-groups (Terzaghi, 1943; Skempton, 1953) have often worked well, but are restricted to cases where the field conditions were similar to those for which the methods were derived.

With the advent of the computer, more sophisticated methods of analysis have been developed to provide more accurate solutions for different pile foundation problems under different site and applied loading conditions. During recent years, there has been an increase in the use of offshore piles which tend to be long and relatively flexible, with the majority of their load capacity being derived from skin friction. These piles are subjected to severe environmental loadings (static, cyclic or dynamic) which differ from those onshore. This difference is more pronounced because the soils encountered at many offshore sites are often different in nature and history to those onshore. (eg. Angemeer, 1975; Poulos, 1982; Nauroy and Le Tirant, 1983; Noorany, 1985; Randolph, 1988b).
These circumstances have made it difficult to employ empirical conventional design approaches and have imposed on geotechnical engineers the need to employ numerical methods which are more capable of modelling the changing stress in the soil during pile installation and loading. The general classification of analysis and design procedures falls into three categories, as was presented by Poulos (1989a) and reproduced in Table 2.4. The same author also presented typical categories of analysis methods for axially loaded piles. The selection of a method in the categories shown in Table 2.4 to design a foundation depends on: economy of project, data available, size of project, site conditions, type of applied load, and finally the design experience for that project.

The author believes that the 3rd category is more appropriate for analysis and design of pile foundations for offshore structure projects. However, still the most important factor for successful design is the appropriate selection of geotechnical parameters.

2.9.1 STATIC LOADING

For both single piles and pile–groups, analyses for static loading will be reviewed first and then the modifications for cyclic loading will be described.

a) Finite Element Approaches

Since its introduction, the finite element (F.E.) method has been applied to an increasingly wide range of problems in geotechnical engineering. Frank (1974) suggested that F.E. methods model the pile shaft as a cylinder, while the surrounding soil is modeled as concentric cylinders. Hence he considered the shear stress mobilized in the soil due to transfer of load from the pile acting on each concentric cylinder. Randolph and Wroth (1978a) have developed effective stress methods using a modified Cam Clay model to simulate the effects on pile behaviour of installation, build–up and dissipation of excess pore pressure during consolidation,
installation and loading.

The F.E. methods are very powerful analytical tools and they are capable of carrying out linear and non-linear analysis of both the soil and the piles. These methods can model nonlinear soil behaviour, and can also simulate the behaviour of piles during installation and subsequent loading (Withiam and Kulhaw, 1979). F.E. methods are presented in three forms, one-dimensional, two-dimensional and three-dimensional methods.

Finite element analysis is suited to the computation of the load and settlement of single piles (Leong and Randolph, 1991). They conducted one-dimensional finite element analysis on a soil plug in open pipe piles under static load. Their study showed that high internal friction is mobilized inside a pipe pile owing to build up of axial effective stresses within the soil plug. This increase in axial effective stresses in turn produces high radial stresses at the wall of the pile and thus high internal friction. Thus, under such situations, pipe piles can be considered as closed-end piles.

The axi-symmetric analysis uses a two-dimensional element to model a pile surrounded by soil. The axi-symmetric element approach has been attractive to many researchers involved with the analysis of the behaviour of axially loaded single piles and pile groups (for example Desai, 1974; Hooper, 1973; Valliapan et al, 1974; Balaam et al, 1975; and Pressley and Poulos, 1986).

For pile groups, each concentric row of piles is simulated by a continuous annulus with an overall stiffness (i.e displacement per unit force) equal to the sum of the stiffnesses of the individual piles. Hooper (1973) used this numerical approach to carry out an analysis of piles supporting the Hyde Park Cavalry Barracks in London. He used the F.E.M. with axi-symmetric elements for modelling the soil, piles and raft foundation and found that he was able to predict the behaviour of the
foundation well. The same method was used by Al-Douri (1986) to analyse a pile group beneath a large storage tank by modelling the piles as a series of annular rings. He found that this method was capable of producing a good prediction of performance of a full scale Ammonia tank which was constructed in India and reported by Mohan et al (1978).

Three-dimensional finite element analyses of 5x3 and 3x3 pile groups were conducted by Ottaviani (1975) to study the maximum shear stress distribution on a horizontal cross-section just above the base of the piles in the group. He showed that the soil stresses inside the perimeter of the pile group were significantly smaller than those outside. His analysis was limited by the assumption of linear-elastic behaviour for the soil, with no allowance for possible slip at the pile-soil interface, or failure within the soil.

Balaam et al (1975) incorporated pile-soil interface behaviour in their nonlinear finite element analysis of axi-symmetric problems. They used this analysis to study the effects on pile settlement of non-homogeneity of the disturbed soil due to pile installation. The above discussions indicate that although the axi-symmetric method is restricted by the geometrical arrangement of piles that it can analyse, it is more popular than other methods because it provides reasonable results for single piles and pile groups and costs less than three dimensional methods.

The comparison between the results of F.E. analyses obtained by Lee (1973) and the corresponding solutions from elastic theory has been described in detail by Poulos and Davis (1980).

b) Load Transfer Approaches

The load transfer approach is a one-dimensional formulation of pile-soil interface response. The analysis in these approaches models the pile shaft as a series of compressible elements while the surrounding soil is modelled as a discrete spring
resistance at each element.

One of the earliest applications of the load transfer function approach was by Coyle and Reese (1966). They developed this approach to compute the settlement of axially loaded piles. Their results relied markedly on the particular \((t-z)\) curves utilised for the computation. Load transfer \((t-z)\) relationships have been evaluated empirically and semi-empirically due to extension of the method by several investigators (Holmquist and Matlock, 1976; and Vijayvergiya, 1977). \(t-z\) curves may be evaluated from careful field tests on instrumented piles, but these tests become increasingly complex and expensive for foundations of offshore piles. Therefore several investigators have addressed the task of constructing these curves theoretically (Randolph and Wroth, 1978a and 1978b; Kraft et al, 1981; Ha and O’Neill, 1983; Ooi et al, 1989).

A comparison between empirical and theoretical load transfer curves has been made by Ha and O’Neill (1983). The results showed a reasonable agreement with the curve proposed by Coyle and Reese (1966) but were markedly different from the empirical curve proposed by Vijayvergiya (1977). The difference was attributed to the higher critical displacement required to generate the maximum shaft resistance in the empirical curves.

The movement of the pile at any element of the pile shaft in these approaches is related to the shear stress at that element and is independent of the interaction shear stress elsewhere on the shaft. Therefore the \(t-z\) approach can not be adopted directly for the load-settlement analysis of pile groups due to the fact that the effects of the pile-soil-pile interaction are ignored. Accordingly further modifications have been derived, based on the theoretical expressions of Randolph (1986); Chow (1986a and 1986b); and Chin and Poulos (1991), to treat this problem.

The analysis of the settlement of a single vertically loaded pile has been described
by Randolph and Wroth (1978a). They described this analysis as an attempt to clarify the load transfer behaviour. Six variables (applied load, length, radius, and Young's modulus of the pile and shear modulus and Poisson's ratio of the soil) were considered. In their analysis the soil was divided into upper and lower layers by a horizontal plane at the level of the pile base. They assumed that the soil in the upper layer was deformed exclusively by the load shed by the pile shaft and that the soil in the lower layer was deformed exclusively by the pile base load.

The deformations of the soil around the shaft were idealized as the shearing of concentric cylinders. In this analysis, the pile compressibility and the soil inhomogeneity were taken into account. This analysis method was an extension of that by Randolph (1977) which was used to study the effect of neighbouring piles. The preceding analysis can be extended readily to consider pile-groups embedded in the soil.

In Randolph and Wroth's analysis for a single pile, the settlement of the pile at a node, \( w_s \) due to an uniform distributed shear stress at the pile shaft \( \tau_0 \) is expressed as:

\[
w_s = \frac{\tau_0 r_0}{G} \ln \left[ \frac{r_m}{r_0} \right] \quad \ldots \ldots (2.10)
\]

where
- \( G \) = soil shear modulus
- \( r_0 \) = pile radius
- \( r_m \) = maximum radius at which the shear stress becomes negligible

The value of \( r_m \) is proportional to the soil Poisson's ratio and the embedded length of the pile (\( \ell \)). Thus the value of \( r_m \) is calibrated against elastic solutions to be:

\[
r_m = 2.5 \ell (1-\nu) \quad \ldots \ldots (2.11)
\]

where
- \( \nu \) = soil Poisson's ratio
The magnitude of $r_m$ has been the subject of much attention by Randolph and Wroth (1979) for the analysis of response of both a single pile and a pile-group embedded in either homogeneous or non-homogeneous soils. For soil in which the shear modulus varies linearly with depth (Gibson soil), $r_m$ is expressed as approximately:

$$r_m = 2.5 \rho \ell (1-\nu) \quad \ldots \ldots \quad (2.12)$$

where

$$\rho = \text{factor for soil inhomogeneity}$$

Randolph and Wroth (1979) described how the analytical method for a single pile may be extended to deal with pile groups embedded in soil in which the modulus increases linearly with depth. This method applies to pile groups with equal length of piles. The preceding analysis has been further refined by Randolph (1986) to be more suitable for application to checking pile groups in both homogeneous and non-homogeneous soils.

It is worth mentioning that not all piles in a group are always of the same diameter and length, and each pile may carry a different load. The analysis of pile groups (Randolph and Wroth 1979) is carried out by assuming all the piles in the group to be rigid and of the same length, and then summing the settlement of each pile.

For the jth pile of a group of n piles, the shaft settlement is:

$$w_s = \frac{1}{G} \sum_{j=1}^{n} (\tau_o(r_o) \ell n \left[ \frac{r_m}{s_{ij}} \right] \ldots \ldots \quad (2.13)$$

where

$$s_{ij} = \text{spacing between piles i and j, and } s_{ij} = r_o \text{ for } i= j$$

The different values of $w_s$ may be related to the n values of shear stress $\tau_o$ in the above equation.

The value of $r_m$ has been found to increase by an amount $r_g$ which is related to
the dimensions of a pile group. Thus \( r_m \) may be expressed in a new form:

\[
    r_m = 2.5 \rho \ell (1-\nu) + r_g \quad \ldots \ldots \quad (2.14)
\]

where

\( r_g \) for example may be taken as the radius of a circle of equivalent area covering the plan area of the pile group.

c) Elastic Approaches

i) Single Pile

These approaches have generally used Mindlin's (1936) elastic equations for an isotropic half space to represent the soil. They include the integral equation approach (Poulos and Davis, 1968; Poulos, 1968; Poulos and Mattes, 1968; Butterfield and Banerjee, 1971a; Poulos, 1979a; Poulos and Davis, 1980) and the discrete element method (O'Neill et al, 1977; Chow, 1986b)

In these methods, the piles are represented as an elastic cylinder and the surrounding soil acts as an elastic continuum. Thus any load applied to the pile will affect the whole half-space. Three forms of loading may be considered to represent the pile-soil interaction stress acting on a pile element:

a) a vertical point load at the centre of the element (Thurman and D'Appolonia, 1965),

b) a uniform vertical load distribution applied over the cross-section area of the element (Salas and Belzunce, 1965; and Lee et al, 1987),

c) a vertical load distribution applied over the outer circumferential area of the element (Poulos and co-investigators; and Butterfield and Banerjee, 1971a).

The elastic approach to settlement computation for end-bearing piles, employing a "mirror-image" technique was firstly developed by Thurman and D'Appolonia (1965).

Poulos and Davis (1968) present a simplified boundary element method to analyse
the behaviour of a single axial incompressible floating pile in an elastic half-space.
Pile-soil slip has been considered in this analysis, and the effect of compressibility of
the pile was subsequently studied by Mattes and Poulos (1969). The analysis was
further extended to the case of non-homogeneous soil (Poulos, 1979a).

Poulos and Davis (1980) describe a simple form of boundary element analysis in
which the piles are divided into a number of equal cylindrical elements and the soil
acts as an elastic continuum as shown in Figure 2.18. For a uniform section in a
homogeneous soil, the incremental soil displacement \( \Delta \rho_T \) at a node may be
expressed as:

\[
(\Delta \rho_T) = \frac{d}{E_S} [I_S] (\Delta p) \quad \ldots \ldots \quad (2.15)
\]

where

\[
[I_S] = \text{matrix of soil displacement influence factors obtained by double integration}
\]

of

Mindlin's (1936) equations, as described by Poulos and Davis (1980).

\[
E_S = \text{Young's modulus of soil},
\]

\[\{\Delta \rho_T\} = \text{the (n+1) vector of soil displacements},\]

\[\{\Delta p\} = \text{the (n+1) vector of shear stresses at the soil-pile interface, and the base}
\]

pressure.

The above formula was extended to cover a non-homogenous soil mass by changing
the modulus of the soil \( E_S \) to be:

\[
E_S = \frac{E_{si} + E_{sj}}{2} \quad \ldots \ldots \quad (2.16)
\]

where

\[
E_{si} = \text{soil modulus at influenced element i},
\]
\[ E_{sj} = \text{soil modulus at influencing element j}. \]

**ii) Pile Groups**

The method used to compute the settlement of a single pile was extended to compute the settlement of pile groups, considering the effects of interaction between two identical piles. The increase in settlement of each pile due to interaction with another pile is expressed in terms of an interaction factor (Poulos, 1968; Poulos and Mattes, 1971). The interaction factor is defined as:

\[
\alpha = \frac{\text{Increase in settlement of pile "i" due to pile "j"}}{\text{Settlement of pile "i" due to its own load}} \quad \ldots \quad (2.17)
\]

The interaction factor \( \alpha \) depends upon the pile length and diameter, spacing, and the stiffness of the piles.

El–Sharnouby and Novak (1985) developed an efficient method of determining the flexibility coefficients for pile group analysis. In this method the displacement of any element of a pile due to all elements of that pile and other piles in the group is considered. The double integration of Mindlin's equation was avoided in this approach and consequently the computation time was reduced substantially.

Hewitt (1988) developed equations which did not rely on the use of interaction factors. Hewitt's analysis considered the case when the lengths of the piles in group are not same. Similar considerations have been taken into account in the t–z analysis approach by Randolph and Wroth (1979).

The analysis for a single pile can be extended to the case of a pile group and the modification of the single pile equation can be written as:

\[
(\delta_z) = [H][\sigma] \quad \ldots \ldots \quad (2.18)
\]

where \( \{\delta_z\} \) and \( \{\sigma\} \) represent the global soil displacement and interaction stress.
incremental vectors, and [H] represents the global soil influence factor matrix.

d) Hybrid Approaches

This type of approach combines the elastic and load-transfer approaches and therefore is often referred to as the Hybrid approach. In this method, the single-pile response is obtained by the load-transfer (t-z) method while the pile-soil-pile interaction is considered by the use of interaction factors. It was first proposed by Focht and Koch (1973) for laterally loaded pile groups and extended by O'Neill et al (1977) to include both axial and lateral loading. They proposed pile-soil-pile interaction correction factors applied to the load-transfer functions for single piles utilising an iterative routine. The deformation of any node on the pile caused by loads at the nodes of other piles was determined by using Mindlin's solution.

The analysis of vertical pile-groups using this approach has been presented by Chow (1986a). The pile-soil-pile interaction is represented by Mindlin's equation and the single pile solution is represented by load transfer curves for the analysis. In this method, the reinforcing effects of the piles in the group was taken into account. This approach originally considered the soil to be a homogeneous, isotropic elastic half-space, but it was extended later to consider layered soil by Chow (1986b). For socketted piles Chin et al (1990) studied the settlement of axially loaded vertical piles and pile groups embedded in homogenous and two-layer soil profiles. Further extension of this analysis to obtain more accurate results was provided by Chin and Poulos (1991). This analysis determined the average value of r_m for two-layer soil profiles.

2.9.2 CYCLIC LOADING

Two points that emerge from the analysis of offshore piles (particularly piles in calcareous soil) need to given consideration: i) the influence of strain softening at the pile-soil interface on axial capacity of the pile under static loading, and ii) the
influence of cyclic loading on the pile capacity and deformation of the soil. The analytical methods employed to analyse a single pile and a pile group under static load have been extended to consider cyclic loading, as described below.

a) Finite Element Approaches

The behaviour of piles under cyclic loading has been studied by Boulon et al (1980) utilising a finite element approach. The study concentrated on the effects of cyclic loading on the soil-pile interaction and soil-fluid coupling. They concluded that the cyclic loading significantly affects the dilatancy of soil and reduces the capacity of the pile.

b) Load Transfer Approaches

The analysis of the non-linear behaviour of a single pile utilising (t-z) approaches has been conducted by several researchers (Matlock and Foo, 1980; Randolph, 1986 and 1988b). A general, versatile numerical procedure was proposed by Matlock and Foo, (1980) to investigate a vertically loaded pile, under both static and cyclic loadings. This analytical approach considered a non-linear, hysteretic, degrading soil model.

Randolph (1986) proposed a form of load transfer curve as shown in Figure 2.19. The curve of shear stress versus displacement linearly progressed from zero to a certain shear stress level. Then the curve was changed to a parabolic shape up to the yield point, followed by strain softening down to the residual shear stress level after certain additional absolute displacement \(\Delta\omega_{\text{res}}\). This curve has been incorporated into Randolph’s computer program "RATZ". Under cyclic loading, the accumulation of plastic displacement (arising from a nonlinear form of the unloading) was treated as being equivalent to the monotonic plastic displacement in calculating the degree of degradation. This load-transfer algorithm was able to separate cyclic shear stress regimes where failure would ultimately occur, from regions in which the
pile did not fail under cyclic loading.

The preceding analysis can take the effects of soil creep into account approximately.

c) Elastic Approaches

Single Pile

Poulos modified his earlier boundary element analysis for static loading to include cyclic loading by employing a number of approaches. This modification is presented in several papers (1979, 1980, 1983, 1984, 1988, 1989 and 1990). The main aspects of the cyclic behaviour of a pile–soil interface which are modelled are:

i) cyclic degradation of skin friction, and

ii) accumulation of permanent displacement.

The most important factor affecting the degradation of skin friction is the cyclic displacement. However, other factors also influence the reduction of skin friction (e.g. number of cycles, load level, loading rate, and plastic settlement of the soil). In order to quantify cyclic degradation of skin friction, Poulos (1979b) introduced the concept of a degradation factor which was defined as the ratio of ultimate skin friction after cyclic loading to the corresponding value for static loading. This definition was generalized (Poulos 1983) to include other soil parameters, the degradation factor "D" being written as:

\[
D = \frac{\text{Property after cyclic loading}}{\text{Property for static loading}} \quad \ldots (2.19)
\]

A summary of the boundary element analysis is described by Poulos (1990) in which the following features are considered:

i) pile–soil slip when the shear stress reaches the current limiting value of shaft
resistance,
ii) accumulation of permanent settlement under cyclic loading,
iii) cyclic degradation of shaft resistance.

The latter is expressed by the simple Matlock and Foo (1980) model:

\[ D_T = (1 - \lambda) (D' - D_{\text{min}}) + D_{\text{min}} \]  \hspace{1cm} (2.20)

where

- \( D_T \) = current value of degradation factor,
- \( D' \) = degradation factor for previous cycle,
- \( D_{\text{min}} \) = minimum value of degradation factor,
- \( \lambda \) = degradation rate parameter.

The second factor is the accumulation of permanent displacement. Reliable theories for this factor have not yet been developed. Poulos (1990) used the following expression to compute the additional permanent soil displacements due to cyclic loading, which is similar to the form applied by Diyaljee and Raymond (1982) to triaxial test results:

\[ \frac{S_p}{d} = A \times e^{(nX)} \times N^m \]  \hspace{1cm} (2.21)

where

- \( S_p \) = permanent displacement,
- \( d \) = pile diameter,
- \( X \) = applied cyclic load level,
- \( N \) = number of cycles,
- \( A,n,m \) = experimentally-determined parameters.
Pile Groups

The above analysis for a single pile has also been used for pile groups by Poulos (1982) and Hewitt (1988), taking account of interaction among the piles in the group. It has been found that group effects tend to increase displacements and cause slightly more severe degradation.

Program for Analysis

The program "SCARP" has been developed by Poulos (1990), and computes the axial static and cyclic displacement and load distribution along a single pile or a pile within groups subjected to specified sequences of static and/or cyclic loading. Either displacement-control or load-control can be specified for static loading. For cyclic loading, load-controlled loading is specified, and both uniform amplitude loads and non-uniform amplitude (storm) loading can be analysed. This analysis is used in Chapter 8 to compare the theoretical behaviour of cyclically loaded piles with that measured in the laboratory model tests.

2.10 SUMMARY

A number of points can be drawn from the foregoing review:

1) The geotechnical investigation of calcareous sediments has only been developed in the last two decades and their engineering behaviour is not yet fully understood.

2) The behaviour of calcareous sand is different to silica sand because of its high compressibility, its high in-situ void ratio, and the presence of cementation and crushable particles. Consequently there is a need to perform more tests to reach a better understanding of the influence of these parameters on engineering behaviour.
3) Initially it was considered that the observed low skin friction for driven piles in calcareous sands was due to the susceptibility of particles to crushing and to random cementation in the soil. However, according to recent research (e.g. Semple, 1988) the initial density also appears to be a very important parameter in determining the behaviour of piles in this type of soil.

4) Driven and grouted piles exhibit a considerable loss of skin friction during cyclic loading. However, for a given displacement amplitude and number of cycles it has been found that the reduction in skin friction for model grouted piles was higher than that of driven piles.

5) Laboratory tests such as triaxial and direct shear tests, used to measure the geotechnical parameters for offshore soils, still need more development to allow better control over test conditions.

6) There is limited data from field and model tests on piles in calcareous soils, most of which is concerned only with head response of grouted and driven piles. Details of pile-soil interaction along the pile shaft are not available in the literature.

7) Grouted piles show higher load capacity than driven piles, but the techniques for installation of grouted piles are expensive and complex.

8) There is an absence of experimental data on the behaviour of single piles and pile groups under storm load conditions. However, some recent experimental and theoretical data on similar types of loading for grouted single piles has been published by Lee and Poulos (1991).

9) There is only limited data from field and laboratory tests on the static and cyclic behaviour of a pile group in clay and silica sands, and apparently no data at all from either field or laboratory tests on the cyclic behaviour of pile
groups in calcareous sand.

10) The existing methods of analysing the interaction of a pile and surrounding soil during axial cyclic loading are an extension of static analysis methods, and need more modification to adequately describe cyclic interaction behaviour.
### Table 2.1a Limiting End-Bearing Pressure for Driven Pile

In Calcareous Sands (Naury et al, 1986)

<table>
<thead>
<tr>
<th>Limiting Compression Index $C_I$</th>
<th>Limiting End-Bearing Pressure MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>$&lt; 0.02$</td>
<td>$\geq 20$</td>
</tr>
<tr>
<td>$0.02 - 0.03$</td>
<td>$15$</td>
</tr>
<tr>
<td>$0.03 - 0.04$</td>
<td>$10$</td>
</tr>
<tr>
<td>$0.04 - 0.05$</td>
<td>$8$</td>
</tr>
<tr>
<td>$0.05 - 0.1$</td>
<td>$4$</td>
</tr>
<tr>
<td>$0.10 - 0.20$</td>
<td>$1.5$</td>
</tr>
<tr>
<td>$0.20 - 0.30$</td>
<td>$1$</td>
</tr>
<tr>
<td>$0.30 - 0.50$</td>
<td>$0.5$</td>
</tr>
<tr>
<td>$&gt; 0.5$</td>
<td>$&lt; 0.5$</td>
</tr>
</tbody>
</table>

### Table 2.1b Limiting Values of Ultimate Skin Friction Driven Pile in Calcareous Sand (Naury et al, 1986)

<table>
<thead>
<tr>
<th>Limiting Compression Index $C_I$ *</th>
<th>Limiting Value of $f_s$ (kN/m$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Open-Ended Pile</td>
</tr>
<tr>
<td>$&lt; 0.02$</td>
<td>100</td>
</tr>
<tr>
<td>$0.02 - 0.03$</td>
<td>50</td>
</tr>
<tr>
<td>$0.03 - 0.04$</td>
<td>20</td>
</tr>
<tr>
<td>$0.04 - 0.05$</td>
<td>10</td>
</tr>
<tr>
<td>$0.05 - 0.1$</td>
<td>5</td>
</tr>
<tr>
<td>$0.10 - 0.20$</td>
<td>0</td>
</tr>
<tr>
<td>$0.20 - 0.30$</td>
<td>0</td>
</tr>
<tr>
<td>$0.30 - 0.50$</td>
<td>0</td>
</tr>
<tr>
<td>$&gt; 0.5$</td>
<td>0</td>
</tr>
</tbody>
</table>

* Determined from isotropic triaxial compression test at effective stress of 800 kN/m$^3$
<table>
<thead>
<tr>
<th>References</th>
<th>Location</th>
<th>Pile description</th>
<th>Pile diameter, D (m)</th>
<th>Pile embedment, E (m)</th>
<th>Width of side for square pile, S (m)</th>
<th>Number of tension tests</th>
<th>Number of compression tests</th>
<th>Soil description</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Angmeer, et al. (1973)</td>
<td>Bass Strait, Australia</td>
<td>Steel pipe, open end</td>
<td>0.407-0.508</td>
<td>—</td>
<td>45.1-102.4</td>
<td>—</td>
<td>6</td>
<td>8</td>
<td>Silty calcareous sand uncremented to highly cemented sand</td>
</tr>
<tr>
<td>Angmeer, et al. (1975)</td>
<td>Northwest Shelf, Australia</td>
<td>Steel casing, open end</td>
<td>0.34</td>
<td>4.9-11.0</td>
<td>—</td>
<td>—</td>
<td>2</td>
<td>—</td>
<td>Calcareous sandy silt with layers of silty sand</td>
</tr>
<tr>
<td>Stevenson and Thompson (1978)</td>
<td>Barbados</td>
<td>Square, prestressed concrete piles</td>
<td>—</td>
<td>18.6-21.9</td>
<td>0.406</td>
<td>—</td>
<td>8</td>
<td>Coral and coral sand of varying density</td>
<td>One test with oversize plate; five proof tests</td>
</tr>
<tr>
<td>Fuller (1977); Engling (1980)</td>
<td>Arabian Gulf near Ja'aymah Saudi Arabia</td>
<td>Circular, prestressed concrete cylinder piles</td>
<td>—</td>
<td>10.6-19.6</td>
<td>1.37</td>
<td>—</td>
<td>6</td>
<td>Thin caprock overlying well cemented carbonate sand</td>
<td>Piles driven in predrilled hole; majority proof tested</td>
</tr>
<tr>
<td>Hagenaar and Vanden Berg (1981)</td>
<td>Red Sea coast of Saudi Arabia</td>
<td>Octagonal prestressed concrete pile with hollow core</td>
<td>0.60</td>
<td>18.0</td>
<td>—</td>
<td>—</td>
<td>1</td>
<td>1</td>
<td>Coral, carbonate sands and gravels, weakly cemented limestone rock, thin alluvial layers</td>
</tr>
<tr>
<td>Hagenaar, et al. (1982; 1985)</td>
<td>Red Sea coast of Saudi Arabia</td>
<td>Steel pipe, open and closed end</td>
<td>0.614-1.42</td>
<td>10.3-45.0</td>
<td>—</td>
<td>—</td>
<td>4</td>
<td>24</td>
<td>Carbonate sands, coral, buried coral, coral detritus, and aluvial sediments intermixed</td>
</tr>
<tr>
<td>Puyuelo, et al. (1983)</td>
<td>Philippines</td>
<td>Steel pipe, partially closed and closed end</td>
<td>0.762</td>
<td>10.0-42.0</td>
<td>—</td>
<td>—</td>
<td>6</td>
<td>6</td>
<td>Coral sands and gravels with cemented coral lenses</td>
</tr>
<tr>
<td>Dutt and Cheng (1984)</td>
<td>Gulf of Suez</td>
<td>Steel pipe, open end</td>
<td>0.609</td>
<td>7.62-30.49</td>
<td>—</td>
<td>—</td>
<td>12</td>
<td>—</td>
<td>Carbonate sand, weakly to moderately cemented</td>
</tr>
<tr>
<td>Dutt, et al. (1985)</td>
<td>Philippines</td>
<td>Steel pipe, open end</td>
<td>1.06</td>
<td>40.5-67.0</td>
<td>—</td>
<td>—</td>
<td>4</td>
<td>—</td>
<td>Coraline limestone and coral gravel; carbonate silt and siltstone</td>
</tr>
<tr>
<td>Gilchrist (1985)</td>
<td>Red Sea coast of Saudi Arabia</td>
<td>Steel pipe, open end</td>
<td>1.422</td>
<td>11.0-44.0</td>
<td>—</td>
<td>—</td>
<td>5</td>
<td>5</td>
<td>Honeycomb coral, coralline silty sand</td>
</tr>
<tr>
<td>Nauroy and Lettrant (1985)</td>
<td>Western France</td>
<td>Steel pipe, open end</td>
<td>0.30</td>
<td>23.0</td>
<td>—</td>
<td>—</td>
<td>1</td>
<td>—</td>
<td>Uncemented carbonate sands with thin cemented seams</td>
</tr>
<tr>
<td>Reference</td>
<td>Location</td>
<td>Pile Diameter, $d$</td>
<td>Pile Embedment, $L$</td>
<td>Number of Tests (T or C)</td>
<td>Soil Description</td>
<td>Static Skin Friction, $f_s$ (kPa)</td>
<td>Remarks</td>
<td></td>
<td></td>
</tr>
<tr>
<td>----------------------</td>
<td>-------------------</td>
<td>--------------------</td>
<td>---------------------</td>
<td>--------------------------</td>
<td>----------------------------------------------------------------------------------</td>
<td>---------------------------------</td>
<td>-------------------</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wees and Chamberlain (1971)</td>
<td>Dubai</td>
<td>0.91 27.4</td>
<td>0.27.4</td>
<td>1T</td>
<td>Layers of well cemented limestone with sands and silts, some weakly cemented</td>
<td>&gt;85</td>
<td>$q_u = 1.5 - 18.5$ MPa</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Angemeer et al. (1973)</td>
<td>Bass Strait, Australia</td>
<td>--</td>
<td>--</td>
<td>2T</td>
<td>Silty calcareous sand uncemented to strongly cemented</td>
<td>&gt;100</td>
<td>CaCO$_3 &gt; 90%$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Angemeer et al. (1975)</td>
<td>Northwest Shelf, Australia</td>
<td>0.48-0.50 12.2 122.0-134.1</td>
<td>91.5-103.7 122.0-134.1</td>
<td>2T</td>
<td>Calcareous sandy silt, slightly cemented</td>
<td>&gt;100</td>
<td>CaCO$_3 &gt; 90%$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Settgest (1980)</td>
<td>Southern Arabian Gulf</td>
<td>0.56 4.3</td>
<td>9.1-13.4</td>
<td>1T</td>
<td>Loose bioclastic sand overlying weakly cemented calcareous siltstone</td>
<td>&gt;540</td>
<td>$q_u = 0.34 - 10.3$ MPa</td>
<td></td>
<td></td>
</tr>
<tr>
<td>King et al (1980)</td>
<td>Northwest Shelf, Australia</td>
<td>0.15 3.0</td>
<td>73.0-76.0</td>
<td>1T</td>
<td>Calcareous sandy silt</td>
<td>?</td>
<td>No actual $f$ value given but less than 100 kPa.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Nauroy et al (1985)</td>
<td>Western France</td>
<td>0.31 7.85</td>
<td>7.15-15.0</td>
<td>1T</td>
<td>Uncemented carbonate sands</td>
<td>&gt;150</td>
<td>CaCO$_3 = 80 - 90%$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ismael and Al-Saned (1986)</td>
<td>Kuwait</td>
<td>0.5 5.35-10.45 3.5-14.5</td>
<td>8.5-14.5</td>
<td>9T</td>
<td>Dense calcareous silty sand, weakly cemented</td>
<td>80 to &gt; 158</td>
<td>CaCO$_3 &lt; 10%$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Withers et al (1986)</td>
<td>Bass Strait, Australia</td>
<td>0.25 1.5-2.0 33.2-143.5</td>
<td>33.2-143.5</td>
<td>16T</td>
<td>Variably cemented carbonate soil</td>
<td>360-955</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Williams and van der Zwagg (1988)</td>
<td>Northwest Shelf, Australia</td>
<td>0.45 2.3</td>
<td>118.9-147.4</td>
<td>4T</td>
<td>Weakly to firmly cemented calcareous sand</td>
<td>438-685</td>
<td>CaCO$_3 &gt; 90%$</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* $T =$ tension test  
  $C =$ compression test
Table 2.3 b  Summary of Drilled and Grouted Pile Field Loading Tests  
(Allman, 1988)

<table>
<thead>
<tr>
<th>Reference</th>
<th>Location</th>
<th>Pile Diameter, $d$</th>
<th>Overburden Pressure $\sigma_m$ (kPa)</th>
<th>Number of Tests (T or C)*</th>
<th>Soil Description</th>
<th>Static Skin Friction $f_s$ (kPa)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young (1983)</td>
<td>Bass Strait, Australia</td>
<td>0.027 0.270</td>
<td>69–276</td>
<td>37T 2C</td>
<td>Siliceous carbonate sand</td>
<td>54–186</td>
<td>CaCO$_3$ = 87 – 89%</td>
</tr>
<tr>
<td>Nauroy and Le Tirant (1985)</td>
<td>Western France</td>
<td>0.03 0.10</td>
<td>50–400</td>
<td>—</td>
<td>Carbonate sand</td>
<td>70–400</td>
<td>CaCO$_3$ = 83 – 90%</td>
</tr>
<tr>
<td>Nauroy and Le Tirant (1985)</td>
<td>Western France</td>
<td>0.07 0.6–0.8</td>
<td>0</td>
<td>—</td>
<td>Carbonate sand, some cemented layers</td>
<td>340–470</td>
<td>tests conducted in situ</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>CaCO$_3$ = 40 – 90%</td>
</tr>
<tr>
<td>Abbs and Needham (1985)</td>
<td>Middle East</td>
<td>—</td>
<td>—</td>
<td>20T</td>
<td>Weak carbonate rock</td>
<td>1000 – 4000</td>
<td>$q_u = 1.7 – 7.0$ MPa</td>
</tr>
<tr>
<td>Nutt and Watt (1987)</td>
<td>Northwest Shelf, Australia</td>
<td>0.025–0.075 0.58</td>
<td>200</td>
<td>6T</td>
<td>Carbonate sand</td>
<td>61–137</td>
<td>CaCO$_3$ &gt; 90%</td>
</tr>
<tr>
<td>Poulos and Lee (1987)</td>
<td>Northwest Shelf, Australia</td>
<td>0.024 0.256</td>
<td>100–400</td>
<td>24C</td>
<td>Carbonate sand</td>
<td>78–255</td>
<td>CaCO$_3$ &gt; 90%</td>
</tr>
<tr>
<td>Jewell and Randolph (1988)</td>
<td>Northwest Shelf, Australia</td>
<td>0.016 0.125</td>
<td>0–350</td>
<td>4C</td>
<td>Cemented and uncemented carbonate sand</td>
<td>110–285</td>
<td>Tests on 83 mm diameter cores</td>
</tr>
<tr>
<td>Fahey and Jewell (1988)</td>
<td>Northwest Shelf, Australia</td>
<td>0.025 0.125</td>
<td>0–500</td>
<td>8C</td>
<td>Cemented carbonate sand</td>
<td>580–741</td>
<td>Tests on 83 mm diameter cores</td>
</tr>
</tbody>
</table>

* T = tension test  
  C = compression test
Table 2.4  Categories of Analysis/Design Procedures (Poulos, 1989)

<table>
<thead>
<tr>
<th>Category</th>
<th>Sub-Division</th>
<th>Characteristics</th>
<th>Method of Parameter Determination</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td></td>
<td>Empirical - not based on soil mechanics principles</td>
<td>Simple in-situ or laboratory tests, with correlations</td>
</tr>
<tr>
<td>2</td>
<td>2A</td>
<td>Based on simplified theory or charts - uses soil mechanics principles - amenable to hand calculation. Theory is linear elastic (deformation) or rigid plastic (stability)</td>
<td>Routine relevant in-situ tests - may require some correlations</td>
</tr>
<tr>
<td>2</td>
<td>2B</td>
<td>As for 2A, but theory is non-linear (deformation) or elasto-plastic (stability)</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>3A</td>
<td>Based on theory using site-specific analysis, uses soil mechanics principles. Theory is linear elastic (deformation) or rigid plastic (stability)</td>
<td>Careful laboratory and/or in-situ tests which follow the appropriate stress paths</td>
</tr>
<tr>
<td>3</td>
<td>3B</td>
<td>As for 3A, but non-linearity is allowed for in a relatively simple manner</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>3C</td>
<td>As for 3A, but non-linearity is allowed for via proper constitutive models of soil behaviour</td>
<td></td>
</tr>
</tbody>
</table>
Figure 2.1 Locations of Bass strait and North Rankin Sites on the North-West Shelf (Modified After King et al, 1980)
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(After Allman, 1988)
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(Poulos and Chan, 1988)

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Figure 2.13  Dependence of Skin Friction Degradation on Cyclic Slip Displacement for Grouted Pile in Calcareous Soils (Poulos, 1988)
Figure 2.14a Development of Permanent Settlement with Number of Cycles (Lee, 1988)

Figure 2.14b Permanent Settlement Versus Cyclic Load Level (Lee, 1988)
Figure 2.15  Shear Transfer Variation Among Piles in a Group
(Matlock et al, 1982)
Figure 2.16  Cyclic Stiffness for $P_c/P_u = 0.70$
(Hewitt, 1988)
Figure 2.17  Variation of Unit Skin Friction and Unit End Bearing Capacity With Depth For Calcareous and Non-Calcereous Sands (Datta et al, 1980)
Incremental Interaction Stresses Acting on Pile

Incremental Interaction Stresses Acting on Soil

Notes: 1. Interaction stresses shown only at a few elements
2. Each dot represents an element collocation point

Figure 2.18 Division of Pile Model into Elements
Figure 2.19  Load Transfer Curve (Randolph, 1986)
Chapter 3  STATIC AND CYCLIC DIRECT SHEAR TESTS ON SANDS

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   3.3.1 Soil Sediments
   3.3.2 Interface Materials

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   3.4.2 Loading Machine
   3.4.3 Data Acquisition System

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   3.8.3 Effect of Void Ratio
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Chapter 3

STATIC AND CYCLIC DIRECT SHEAR TESTS ON SANDS

3.1 INTRODUCTION

An understanding of the shear behaviour of soil and solid-soil interfaces under static and cyclic loading is important for the design and analysis of long piles, such as offshore piles, which depend largely on interface shear resistance for their load-carrying capacity.

In calcareous soils, the skin friction has been found to be lower than in terrestrial silica sands, the principal reason being the relatively high compressibility of these soils (Poulos et al, 1982; Murff, 1987; Randolph, 1988a). The high compressibility of carbonate soils has been attributed in part to crushing of soil particles (Datta et al, 1982; Hull et al, 1988). Nauroy and Le Tirant (1985) state that the value of skin friction of the sand may be very low for highly compressible sand. The skin friction developed by piles is also affected by the roughness of the pile surface. It has been shown (Potyondy, 1961; Suklje and Brodnik, 1963 and Kishida and Uesugi, 1987) that the mobilised friction can be correlated to the surface roughness. For rough
concrete, Kulhawy and Peterson (1979) have found that the interface friction angle $\delta$ can equal the internal friction angle $\varphi$ of soil, and failure may then occur in the soil adjacent to the interface.

In order to understand more clearly the factors affecting the response of pile–soil interfaces, a simple modification to a conventional direct shear box has been made to simulate the cyclic behaviour of pile–soil interfaces, and several series of direct shear tests have been carried out on calcareous and silica sands.

The main objectives were as follows:

1) investigate cyclic shear behaviour via modifications which were made to a conventional direct shear box.

2) to study the effects of initial density, normal stress, grain size and shape, moisture content, and surface roughness on the shear resistance and volume change characteristics of soil–soil and metal–soil interfaces subjected to both static and cyclic loading,

3) to examine the effect of cyclic displacement amplitude and number of cycles on shear resistance and volume change,

4) to examine the amount of particle crushing caused by cyclic loading and how this crushing is influenced by characteristics of the soil.

5) to determine the effect of the gap thickness between the two halves of the shear box apparatus.
The results of both standard and interface static tests can supply useful information about the strength parameters (\( \phi \) and \( \delta \)) which are important in assessing skin friction for the design of pile foundations.

This chapter presents results of the static and cyclic tests under the constant normal load "CNL" condition, while those tests under the constant normal stiffness "CNS" condition will be presented in Chapter 4. The tests to examine the crushing of calcareous soils are described in this Chapter. The implications of these results in assessing pile skin friction are also discussed.

Reasonable agreement has been found between the results of the tests performed on carbonate and non-carbonate sediments and those obtained by other investigators (Poulos et al, 1982; Potyondy, 1961; Suklje and Brodnik, 1963; Valent, 1979 and Desai et al, 1985).

3.2 SCOPE OF "CNL" TESTS

Table 3.1 summarizes the four series of soil shear tests which were performed on seven types of sediment. The variable factors considered in these tests were the following:

1. The effective normal stress \( \sigma_n = 50, 110, 160 \text{kPa} \).
2. The horizontal static shear displacement "\( \rho_s \)" = 5, 6, 7mm.
3. The horizontal cyclic shear displacement "\( \rho_c \)" = 0.5, 1, 2, 5mm.
4. The gap thickness between the two parts of the shear box "\( t \)" = 0, 1, 1.6 and 2mm.
5. The effect of the particle size.
All the tests were performed on dry sands except those in the third series which were carried out on saturated New North Rankin (N.N.R) Calcareous sand.

3.3 MATERIAL PROPERTIES

This section describes the interfaces used in the experimental work, the soil sediments, and the steel and aluminium used for the interfaces.

3.3.1 Soil Sediments

Six different calcareous sands and one silica sand were tested: New North Rankin (N.N.R) and Old North Rankin (O.N.R) from the North West Shelf, Bass Strait (BS) and Barry's Beach (BB) from Victoria, two artificially processed sands supplied by the Commonwealth Scientific and Industrial Research Organization of Australia (CSIRO) Division of Geomechanics with, nominal sizes of 0.25 and 0.5 mm and silica sand from a Sydney Beach.

The grading curves of these sands are shown in Figure 3.1 and some information on the general characteristics of these soils has been given by Hull et al (1988). The results of triaxial tests, presented in several papers in the Proceedings of the International Conference on Calcareous Sediments (Airey et al, 1988; Allman and Poulos, 1988; Kaggwa et al, 1988; and Semple, 1988), give a good description of the general behaviour of calcareous sediments.

The calcareous sands had carbonate contents between 89% and 94%, and were composed predominantly of the remains of marine organisms (Poulos, 1988). The specific gravities \( G_s \) of these sands ranged from 2.72 to 2.77. For comparison, a
quartz Sydney sand was also tested having a grading curve similar to CSIRO 0.25mm sand, and GS of 2.65. Table 3.2 summarizes some physical properties of the soils tested. The maximum and minimum dry densities $\gamma_{d_{\max}}$ and $\gamma_{d_{\min}}$ were obtained using the procedures set out in the Australian Standard for Testing Soils, AS1289 (1979). All sands were tested dry, except for tests in the second series, which were carried out on saturated N.N.R calcareous sand (see Table 3.1).

3.3.2 Interface Materials

Japanese investigators Yoshimi and Kishida (1981a,b) and Uesugi (1987) quantify the interface surface roughness by $R_{\max}$ which is the relative height between the highest and the lowest point of trough along the profile over a specified gauge length. The surface roughness of concrete depends on the surface roughness of the casting form.

Kulhawy and Peterson (1979) found that the roughness of soil and the face of the structural material could be determined from:

$$R = \frac{(D_{60} - D_{10})}{D_{50}}$$

where: $R = \text{roughness}$ and $D_{60}$, $D_{50}$ and $D_{10} = \text{respective particle sizes at 60\%, 50\%, and 10\% finer}$. The grading curve of the soil and the grading curve of the aggregate exposed on the surface of the construction material (concrete) were used to obtained $D_{60}$, $D_{50}$ and $D_{10}$. They described the relative roughness coefficient for a construction material "$R_r$" as the ratio of roughness coefficient for the structure material "$R_{st}$" to that for soil $R_{Soil}$. They found that for rough interfaces, the friction angle of the interface is equal to or greater than the soil internal friction angle.

Valent (1979) found the coefficient of friction between calcareous sands and
construction materials depended on the surface roughness. Yoshimi and Kishida (1981b) reported that the skin friction between a sand and various types of material depended on the quantified surface roughness and did not depend on the type of material.

Three aluminium and three steel blocks, were used in the direct shear box tests to examine the behaviour of interfaces between the above sediments and construction materials. These blocks were 15mm high and were placed in the bottom portion of the 60mm x 60mm shear box.

The surfaces of the metal blocks were finished to a specified roughness. The method used to evaluate the surface roughness was similar to that used by Uesugi (1987) and was related to the particle diameter of the soil. The surface roughness \( R_{\text{max}} \) is defined as the relative height between the highest peak and lowest trough along the profile over a specified gauge length \( L \) as shown in Figure 3.2. The gauge length \( L \) depends on the \( D_{50} \) particle size of the sand which can be derived from a grading curve of the soil. It was found that \( D_{50} \) for the silica sand was about 0.4mm and for calcareous sands varied from 0.15 to 0.6mm. The corresponding \( R_{\text{max}} \) values for both silica and N.N.R calcareous sands were determined using the same procedure by gluing the respective sand to a block.

The surface roughness of the metal–soil interfaces was investigated using microscope photographs taken at different points across the surface of the metal. One photograph was chosen of each metal to demonstrate the surface roughness. These photographs are presented in figures 3.3 (a, b, and c). Another two microscope photographs were taken to show the interface between steel and silica sand, and the interface between steel and calcareous sand (Figures 3.4 a and b). Figures 3.3 and 3.4 show that the surface roughness of the metal blocks is very small compared to
the particle diameter of both N.N.R calcareous and silica soils. It can also be noted from figure 3.4 that the particle size of N.N.R calcareous soil is much smaller than that of silica sand. The silica sand has more rounded grains whereas the calcareous sand has more angularity.

The length of the metal surface shown in the micrograph was about 0.31mm. This length is adequate to represent one gauge length \( L \) for silica sand and up to two gauge lengths for calcareous sand according to the diameter of the particles \( (D_{50}) \) for both sands. An average \( R_{\text{max}} \) value of 5\( \mu \text{m} \) for steel was determined. The value of \( R_{\text{max}} \) for the rougher surface of the aluminium was about 13\( \mu \text{m} \).

A comparison was also made between the surface roughness of these two metals and a stainless steel sheet which was used as a lining of the vessel for model pile tests. It was found that \( R_{\text{max}} \) for stainless steel was about 2\( \mu \text{m} \). Thus, the highest value of \( R_{\text{max}} \) was found for the rough aluminium surface, but this was very much less than the particle diameter \( D_{50} \) of both silica and calcareous sands. It can therefore be concluded that surfaces of these metals were effectively smooth.

### 3.4 TESTING APPARATUS

#### 3.4.1 Testing in Direct Shear

Figures 3.5 and 3.6 show photographic views of the modified equipment used to carry out static and cyclic direct shear tests. It consisted of three shear boxes placed on the structural loading frame. Electrical displacement transducers, monitored by a computer, were used to measure displacement and the shear load in calibrated proving rings.
Three pairs of steel plates used in the tests had thicknesses of 1.0, 1.6, and 2.0mm which were the desired thicknesses of the gaps between the two halves of the shear boxes. One pair of these steel plates is shown in Figure 3.6.

Two methods were developed to prevent the fine particles of the soil escaping from the gap between the leading and trailing edges of the top and bottom halves of the shear box. Escaping particles had been observed during the high cyclic numbers of some non standard tests.

The first arrangement, shown in Figure 3.7a, included a 0.5mm thick strip of rubber membrane which was fixed to the bottom of the shear box. One end of the strip was turned over and clamped on the front wall of the top half, and the other end on the back wall of the top half. This is referred to as a two sided membrane. Soil was prevented from escaping from the sides of the box by two steel plates the same thickness as the gap.

The second arrangement, shown in Figure 3.7b, included a 0.5mm thick rubber tube which was placed around four sides of the soil specimen, instead of two sides of the specimen as in the first type. This type of arrangement was more secure than the first type for preventing the leakage of dust and fine particles. However, the disadvantage of this arrangement was that it could cause some non-uniformity in the stress distribution near the corners, and it also influenced the horizontal force measurement of the test. Consequently, it was decided to use the first arrangement in the subsequent work.

In the initial trial cyclic tests the top half of the shear box was observed to tilt, a problem which has been of concern to several investigators (e.g. Jewell, 1980; Wernik, 1977 and Desai et al, 1985). In the present tests, to prevent tilting and
keep the back and front horizontal forces in the same line, two pairs of steel ribs were placed on each side of each shear box as shown in Figures 3.6 and 3.7. One end of the rib was screwed onto the side of the bottom part of shear box and at the other end a steel roller resting on the top part of the shear box prevented that part from tilting during testing.

The apparatus was calibrated by operating the empty shear boxes, and it was found that there were no measurable friction force or membrane resistance due to these modifications.

3.4.2 Loading Machine

A converted Wykeham Farrance loading machine was used to drive the three shear boxes at the same time from a belt connected between the loading machine and the engaging clutches of the boxes. The Wykeham Farrance machine allowed the selection of rates of loading between 0.000165 mm/min and 2.057 mm/min. If the horizontal shear displacement exceeded that of the design range, due to either a fluctuation of the electrical source or any other reason, an automatic cut-out would stop the machine.

3.4.3 Data Acquisition System and The Computer Accessories

Seven LVDT (linear variable differential transformers) transducers were used to measure the loads and displacements. One LVDT was used to measure horizontal shear movement (the same for all three boxes), three LVDT's were mounted on the top of the specimens in the shear boxes, to read the vertical displacements, and three replaced the original dial gauges in the proving rings to measure the shear forces.
The complete automatic data acquisition system consisted of a Tandy Model 4 computer with two disk drives, a line filter, and the data acquisition and control unit built in the School of Civil and Mining Engineering of Sydney University. It also included an eight-channel digital voltmeter controlled by eight differential amplifiers with a 2 or 6 volt DC power supply.

The BASIC computer program could convert the readings of the transducers to deflection and load values. The tests were controlled by deflection, so that when the shear boxes had deflected a specified horizontal distance the shearing machine stopped and reversed the direction of the movement. Static and cyclic shear stress versus displacement curves for any of the cycles could be stored, and then plotted later by other programs.

3.5 Sample Preparation and Procedure

The specimens were prepared by raining a predetermined quantity of sand into the shear box through a square tube 450mm high with a sieve mesh diffuser. The details of the raining technique are presented in Chapter 5. The height of fall was chosen to produce "medium-dense" sand with a density generally in the range of 9.5–10 kN/m³ for calcareous sands, and 14.5–16 kN/m³ for the silica sand.

To produce denser specimens, the same procedure was used as for medium dense sands, but the wall of the shear box was tapped with a steel rod until no further increase in density was evident. Looser specimens were produced by raining sand into the equipment from a height of not more than 50mm. The ranges of unit weight for "loose", "medium-dense" and "dense" samples are shown in Table 3.3.
Saturated sand samples were prepared in a vacuum desiccator device to remove as much air as possible from the sample, and then by placing the saturated sand into the shear box which was full with deaired water. For all tests, the initial void ratio and density were determined by measuring the mass and the volume of each specimen in the shear box.

In the tests concerned with particle crushing (4th test group, as shown in Table 3.1), the New North Rankin soil was sieved to obtain soils with a particular range of grain sizes. The resulting soil was classified as "fine", "medium", or "coarse" depending on the grain size. The "fine" samples contained particles finer than 300 \( \mu m \), the "medium" had particles between 300 \( \mu m \) and 425\( \mu m \), and the "coarse" sample contained particles between 600\( \mu m \) and 2mm.

A rate of loading of 1 mm/min was used for the series of tests which were conducted with cyclic horizontal displacements of \( \pm 0.5 \) and \( \pm 1 \) mm. A loading rate of 2 mm/min was used for larger horizontal cyclic displacements (\( \pm 2, \pm 3, \) and \( \pm 5 \) mm). Some tests were also performed for the larger displacements using a lower loading rate of 1 mm/min, but the loading rate was found to have no discernible effect in these tests.

### 3.6 RESULTS OF STATIC SOIL–SOIL SHEAR TESTS

Standard monotonic shear tests were performed on 34 dry samples of the various sands, and an additional 7 tests were performed on saturated New North Rankin (N.N.R) sand.

Table 3.4 shows the values of the peak and ultimate shear stresses, friction angles
and the void ratios which were measured after applying normal load to the dry samples.

3.6.1 Effect of Normal Stress and Void Ratio on Shear Behaviour

Results for six sediments are displayed in Figure 3.8. The development of shear stress with horizontal displacement (for "medium dense" sands) is shown in Figure 3.8a, while the associated normal displacement of the specimens is shown in Figure 3.8b. For most sediments, the stress increases towards a peak value and then decreases to an ultimate, but there is relatively little difference between the peak and ultimate shear strengths.

Figure 3.8a shows that the strengths of B.B sand, B.S sand and 0.5 CSIRO sand are slightly higher than for the other sands.

The influences of density and normal stress are shown in Figures 3.9 and 3.10 for the New North Rankin (N.N.R) and Bass Strait (B.S) sands respectively. With "dense" samples, a peak–ultimate behaviour is observed and the samples dilate at failure. For the "loose" samples, no peak is observed and the volume changes are contractile. The ultimate shear stresses at failure are however independent of the initial density.

For all sands, the value of the peak friction angle decreases slightly with increasing normal stress. The peak friction angle of silica sand has a lower value than that of calcareous sediments within the range of normal stress used in the tests, and this may be attributed to the fact that the angularity of silica sand particles is less than that of the calcareous particles.
For N.N.R sand, Figure 3.11 shows the influence of the initial void ratio prior to shearing on the normal displacement of the sample at a horizontal displacement of 7mm. Dilation occurs in this case if the initial void ratio is less than about 1.3, independent of the initial normal effective stress.

Table 3.4 shows the values of friction angle at peak ($\varphi_p$) and ultimate ($\varphi_u$) for all dry sediments which were tested. The values of $\varphi_p$ were approximately $3^\circ$ greater than $\varphi_u$, but there is significant scatter in the results. Similar values have been obtained by other researchers (e.g. Young (1983) and Wong (1989)).

The highest value of peak friction angle for the dense carbonate sediments tested under 160 kPa normal stress was $48.4^\circ$ for Barry's beach (B.B) sand and $51.4^\circ$ for Bass Strait (B.S) sand. These results are similar to those reported by Airey and Randolph and Hyden (1988), who conducted a series of triaxial tests and observed maximum friction angles for two Bass Strait sands of $49.5^\circ$ and $46.2^\circ$. Hull et al (1988) carried out several triaxial tests on Barry's beach (B.B) sand and found the peak friction angle of the sand was $48.7^\circ$.

3.6.2 Effect of Water

One series of tests was performed on saturated N.N.R. sand under different normal stresses. It was found that the initial void ratio of saturated N.N.R specimens was less than for dry specimens. The strength of the saturated specimens was found to be slightly higher than that of the dry specimens, but this was consistent with the lower initial void ratio. Consequently, it is concluded that saturation does not have any significant effect on the direct shear test results for the soil tested.
3.6.3 Effect of Particle Size

The N.N.R sand was sieved to produce specimens with fine and coarse particles, and samples composed of these particles were tested under two normal stresses (55kPa and 160kPa). The results of these tests were compared with those for the natural sand.

Figure 3.12a shows that the specimens of different particle size exhibited nearly the same shear stress versus displacement behaviour under the same normal stress condition. However, Figure 3.12b shows that the normal displacement for each specimen differed considerably, with the coarse sand showing more contraction at all normal stresses.

Figure 3.12c shows that the volumetric change in this test, especially for the specimen with coarse particles, is dependent on the normal stress. In all the tests, crushing of the soil particles occurred during shearing, with a greater amount of crushing occurring for the coarse particles.

3.7 RESULTS OF STATIC INTERFACE SHEAR TESTS

3.7.1 Effect of Normal Stress on the Interface Behaviour

Table 3.5 summarizes the results of the interface tests. The tests were performed using three different normal stresses 55, 110, and 160kPa. Some tests, also shown in Table 3.5, were carried out on three different calcareous sands (O.N.R., 0.25CSIRO and 0.5CSIRO) under a normal stress of 160kPa. The shear stress increases with increasing normal stress.
Typical relationships between shear stress and horizontal displacement for the aluminium interface are shown in Figure 3.13. The interface shear behaviour was found to be similar to the soil–soil shear behaviour.

The ultimate stresses for dense N.N.R sand on aluminium were found to be slightly higher than those obtained from the same soil sheared on steel. This was considered to be due to the fact that the surface roughness of aluminium (R_{\text{max}}) was a little higher than for the steel. However, for N.N.R sand, the peak values of the shear interface stress did not appear to depend on the type of metal. More details about the effect of roughness on the interface behaviour are described in the next section.

Figure 3.14 shows, the peak and ultimate friction angles for soil–soil, soil–aluminium, and soil–steel interfaces. As would be expected from previous experiences, the internal friction angle is higher than the sand–metal angle of friction of interface (\(\delta\)).

3.7.2 Effect of Surface Roughness on Interface Behaviour

The relationship between R_{\text{max}} and the peak and ultimate shear stresses for both calcareous and silica sands are illustrated in Figure 3.15. The peak and ultimate shear stress for both sands increases with increasing R_{\text{max}}.

3.8 RESULTS OF CYCLIC SOIL–SOIL SHEAR TESTS

For the cyclic loading tests, the loading pattern was divided into three stages. The first stage involved applying a shear displacement of 5 or 6mm, and then unloading to zero shear stress. The second stage consisted of a number of two-way
displacement-controlled cycles of various amplitudes. The third stage consisted of final static shearing to a total of 5 or 6mm displacement.

3.8.1 Effect of Normal Stress

Two types of dense sand (N.N.R and Silica) were used to investigate the effects of normal stress on the cyclic behaviour. The behaviour of these two types of sand during cyclic loading are illustrated in Figures 3.16 and 3.17.

Figure 3.16 shows the shear stress and vertical movement variations for samples of dense N.N.R calcareous sand tested at three different normal stresses. The peak shear stress decreased significantly in the first few cycles and then more slowly until reaching a limit after about 40 cycles. The final post-cyclic static shear tests to 6mm displacement showed no increase in shear stress beyond the limit reached in cyclic loading. Figures 3.16 (d, e, and f) show that the vertical movements of the samples increase as the applied normal stress increases, with rate of change in normal displacement for all these normal stresses being greatest during the first few cycles. The corresponding results for the dense silica samples are presented in Figure 3.17 and the general behaviour is similar to that of the calcareous sand in Figure 3.16.

The increase in normal displacement for both sands with increasing numbers of cycles has been observed previously by several researchers (i.e. Silver and Seed, 1971a and 1971b; Desai et al, 1985; Ooi and Carter, 1987; Uesugi, 1987; Morrison, 1988; Airey et al, 1990). However, for a given normal stress, the normal displacement of silica sand is less than that of calcareous sand.

For a cyclic displacement of ± 5mm, Figure 3.18a shows the reduction in shear
stress on the calcareous sand with the number of cycles, while Figure 3.18b shows a plot of shear stress reduction factor versus the number of cycles. The reduction factor \( D_f \) is defined here as the ratio of shear stress, after a certain number of cycles, to the maximum shear stress for the first cycle. Figure 3.18b shows that \( D_f \) decreases as the normal stress increases, that is, the shear stress reduction is greater for higher normal stresses.

Further results of cyclic tests on dense sand samples are discussed in Section 4.6.3.

3.8.2 Effect of Displacement Amplitude

The effects of the displacement amplitude of two way cyclic loading \( (\rho) \) on cyclic shear stress and vertical displacement are shown in Figure 3.19. In this figure the solid line represents the 2nd cycle in the second stage and the dotted line represents the 70th cycle. The results of shear behaviour for N.N.R. calcareous sediment were affected by both the cyclic displacement amplitude \( (\rho) \) and normal load. The shear stress does not significantly decrease with small values of \( (\rho) \), but it decreases significantly with large values of \( \rho \), especially with \( \rho \) equal to 5mm.

Figure 3.20 shows the effect of \( \rho \) on the normal displacement of calcareous and silica sediments, for a normal stress of 160kPa. This figure shows that the normal displacement increases with increasing numbers of cycles. This increase becomes more pronounced as the cyclic displacement amplitude increases. However, the normal displacement of calcareous sand is greater than that for silica sand for a given value of displacement amplitude \( (\rho) \).
3.8.3 Effect of Void Ratio

A series of tests was carried out to investigate the differences in the cyclic shear behaviour of dense and loose calcareous sands. The void ratios of these two samples were 1.09 and 1.9, respectively, after normal stresses of 55kPa were applied. The horizontal cyclic displacement of the two samples was ±5 mm.

Figure 3.21 shows that the dense sample reached a peak in the 1st cycle and then the peak shear stress reduced with further cycling, whereas for the loose sample the shear stress reached a peak in the 2nd cycle, and then reduced slightly.

Figure 3.21c shows the vertical displacement versus horizontal displacement for various numbers of cycles. The dense sample exhibited dilation in the initial shearing, but during subsequent cycles it contracted. The loose sample exhibited contraction in the initial shearing, and the subsequent cyclic volumetric contraction was larger than for the dense sample. As expected the normal displacement of the loose sample was more than that of the dense sample.

Figure 3.21d plots the initial void ratio versus vertical displacement at the end of a cycle. For each cycle, the vertical displacement increases with increasing void ratio.

3.8.4 Effect of Particle Size

To investigate the effects of particle size, tests were performed using natural N.N.R sand, and samples containing sieved fine and coarse-sized particles. For "medium dense" samples, the shear behaviour of the natural and fine particle samples was found to be similar. Initially the curves of shear stress versus displacement for both the natural and fine sized samples rose to a peak and then declined following
further shearing. However, for the sample composed of coarse particles, the shear stress did not attain a peak value during static shearing.

Figure 3.22 plots the normal displacement versus number of cycles for the three samples. After five cycles, the vertical displacement of the coarse sand is significantly higher than that for natural sand or the fine sand, indicating that the particle size (and possibly particle shape) may influence both the shear stress and volume change developed during cyclic loading.

3.8.5 Effect of width of Gap Between Two Halves of Shear Box

An attempt was made to examine the effects of the width of the gap between the top and the bottom halves of the shear box on the soil behaviour. Several cyclic tests were carried out on medium–dense calcareous sand subjected to a normal stress of 55kPa. The thickness of the steel plate, which is equal to the width of the gaps shown in Figure 3.7 (a and b), was 1, 1.6, and 2mm. In order to let the shearing force axis correspond to the central plane between the two halves of the shear box, the diameter of the bearings underneath the bottom half (see Figure 3.7) were changed when the gap width was changed.

Figure 3.23 shows a typical pattern of cyclic shear stress and vertical displacement for three samples of N.N.R calcareous sand tested at various gap widths. Figure 3.23 (a, b, and c) shows that for a given gap width, the shear stress reduces as the number of cycles increases and the greatest stress degradation occurs with the greater thickness of shear plane. Figure 3.23 (d, e, and f) illustrates that the difference in normal strain for the three different gap widths is not significant.

These results suggest that the shear stress may decrease with increasing thickness of
the shear plane. But, the stress state in the shear box will have been affected by
the presence of the gap, possibly becoming more non-uniform, and thus making it
difficult to come to this conclusion.

3.9 RESULTS OF CYCLIC INTERFACE BEHAVIOUR

3.9.1 Effect of Normal Stress and Displacement Amplitude

Figures 3.24a and 3.24b show the maximum shear stress developed in two tests on a
N.N.R sand–aluminium interface versus the number of cycles under two different
normal stresses. As with the soil–soil tests, which are also shown in Figure 3.24, the
shear resistance decreases as the number of cycles increases, with the specimen
under the higher normal stress showing the greater decrease in shear stress.

Figure 3.25 summarizes the relationship between the vertical displacement and the
number of cycles for six tests on N.N.R. calcareous sand. Three specimens were
sheared to ±1 mm under different normal stresses (55, 110 and 160kPa) and the
other three specimens were sheared to ±5 mm under the same set of normal stresses
after an initial shear displacement of 5 mm. This figure shows that the vertical
displacement increases with increasing number of cycles, increasing normal stress, and
increasing cyclic displacement.

Comparison with Figure 3.19 shows that the vertical displacement for the soil–metal
interface is less than that of the soil–soil interface. The variation of vertical
displacement with horizontal displacement during successive cycles is also similar in
nature to that for the soil–soil tests (Figure 3.20)
3.9.3 Effect of Cycling on Particle Crushing

To obtain samples with identical grading curves, the soil was sieved, and the material retained on different sieves was mixed with constant proportions. The grading curves for fine and coarse samples before testing are shown in Figure 3.26.

Tests were carried out on soils containing fine and coarse particles. The degree of crushing can be expressed in terms of a crushing coefficient $C_c'$ which has been defined (Datta et al.; 1982) as the ratio of the percentage of particles of sand (after being subjected to stress) finer than $D_{10}$ of the original sand, to the percentage of the particles of original sand finer than $D_{10}$.

$C_c'$ for the two types of different particle size are presented in Table 3.6. The coarse sand shows the greatest tendency to suffer damage. This may be due to the lower number of contact points, and hence higher contact stress, between adjacent particles for the coarse sand. The lower number of contact points in the coarse sand is a consequence of the voids between the large particles and the individual particle characteristics (thin walled, rough, angular and containing intra-particle voids).

For two soils, one containing mainly fine particles, and the other mainly coarse particles, Figure 3.27 shows the influence of the number of cycles on the crushing coefficient. The greater damage occurred in the coarse sands where the inter-particle contact stresses are likely to be greater. Other researchers have observed similar behaviour for similar soils (i.e. Valent et al., 1982; Joustra and De Gilt, 1982; Morrison et al., 1988).

The rate of damage (i.e the rate of increase in $C_c'$) is greatest for the first few cycles and then decreases with further cycles. The reduction in the rate of particle
damage with increasing number of cycles may be due to the creation of finer particles which move to fill the space between the large particles, so that the sand is packed more densely (Price 1988). This may increase the contact area for individual particles and lessen the interparticle contact stresses and hence the amount of subsequent damage.

Tests were performed on samples subjected to two different normal stresses (55 kPa and 160 kPa), sheared to a constant displacement amplitude of 5 mm. The grading curves for both fine and coarse sands before and after 50 cycles are displayed in Figure 3.28. The crushing coefficient was found to increase only slightly for the larger normal stresses.

3.9.4. Effect of Displacement Amplitude on Particle Damage

Six types of calcareous sand were tested to investigate the effect of displacement amplitude on particle damage under a normal stress of 160. The results of the tests are summarized in Table 3.7. The table shows the crushing coefficient ($C_c'$) and the change in the crushing coefficient ($\Delta C_c'$) for these sands after 50 cycles at two different cyclic displacements (1 and 5 mm).

Figure 3.29 shows a plot of $\Delta C_c'$ against the displacement amplitude, where $\Delta C_c'$ is defined as the increase in crushing coefficient due to cyclic shearing. The larger the value of $\Delta C_c'$, the greater is the degree of particle damage.

The N.N.R and O.N.R calcareous sands exhibit a lower rate of particle damage than the B.S and B.B calcareous sands. In addition, the rate of particle damage for 0.5 CSIRO is higher than that for 0.25 CSIRO. Thus the rate of particle damage may depend both on the type of sand and the fineness of the sand. For a given type of
sand, the damage may be attributed to the effects of particle-size distribution, density, stress state, particle size and shape. Similar observations have been made by Hardin (1985).

Similar tests were also carried out on a sand–aluminium interface. It was found that, for all types of natural calcareous sand, there was no significant difference between the $\Delta C_c'$ values after 50 cycles for the soil–soil and soil–interface tests. However, for the 0.5 CSIRO sand, the particle damage obtained from soil–soil tests was significantly higher than that obtained from interface tests. It is therefore concluded that, at the interface between a calcareous soil and aluminium, the particle damage is less than or equal to that when the soil itself is sheared.

3.10 IMPLICATIONS FOR PILE DESIGN

The foregoing direct shear test results clearly have some relevance to the skin friction which may be developed along piles in sand. In so far as they affect the interface friction angle between a pile surface and a sand, such factors as initial density or void ratio, particle size, soil grading and normal stress level may influence the pile skin friction developed under static loading conditions. However, perhaps of more importance is the volume change behaviour of the soil, which may substantially influence skin friction under both static and cyclic loading conditions.

For soils such as dense silica sands which tend to dilate during shear under constant normal stress conditions, it would be expected that, under constant normal stiffness conditions (which more closely represent the conditions of a pile–soil interface), there would be an increase in normal stress and consequently an increase in frictional resistance.
Conversely, for soils which tend to contract (such as loose to medium–dense calcareous soils), there would be a tendency for a reduction in normal stress and a consequent reduction in frictional resistance. Thus, it would be expected that the skin friction developed along a pile in calcareous sand would be less than that along the same pile in silica sand, and this indeed is the common experience (Young, 1983 and Poulos, 1988).

Under cyclic loading conditions, even initially dense soils contract, thus implying that cyclic loading may lead to a gradual reduction of skin friction. This reduction will generally be greater for calcareous sand than for silica sand, since the former exhibits greater volume reductions during shear. Moreover, the greater the cyclic displacement, the greater is the tendency for volume reduction, and the greater the potential for reduction of skin friction. Again, these characteristics have been observed from several model pile tests (e.g. Poulos 1988). In Section 4.7, the relationship between constant normal stress and constant normal stiffness test results is developed, and it is demonstrated that conventional constant normal stress direct shear tests may be able to provide useful design data for assessing the cyclic degradation of skin friction of piles in sand.

3.11 SUMMARY AND CONCLUSIONS

Direct shear tests have been carried out to investigate the shear behaviour of various sands and sand–metal interfaces under static and cyclic loading.

Nearly one hundred tests were performed on different calcareous sands and a silica (Sydney Beach) sand. For the interface tests, both steel and aluminium plates were used, and the surface roughness $R_{\text{max}}$ was measured. The highest value of $R_{\text{max}}$,
for the aluminium surface, was significantly less than the particle diameter $D_{50}$ of the sand tested, and it was therefore concluded that the surface of the metals were effectively smooth.

The results of static shear tests for both soil–soil and soil–metal interfaces indicate that:

i) the peak and ultimate shear strength depend on the normal stress, void ratio, and angularity of the soil particles, as is well-known from previous experience.

ii) the peak shear strength of dense dry calcareous sand was developed at relatively large shear displacements, whereas that of dense silica sand was developed at a smaller shear displacement. This is consistent with the fact that the friction angle was fully mobilised at a smaller shear displacement in a sand of low initial void ratio than in a sand of high initial void ratio, as observed by Semple (1988).

iii) for a particular void ratio and normal stress, the effect of particle size on shear behaviour is relatively small.

iv) the peak and ultimate shear stresses of calcareous and silica sands increase with increasing $R_{\text{max}}$.

v) both the peak internal friction angle and the soil–metal interface friction angle decrease as the initial void ratio increases.

vi) for the same relative density, the internal friction angle ($\varphi$) and the interface friction ($\delta$) for calcareous sand are greater than those for silica sand. The magnitude of $\varphi$ is greater than that of $\delta$ for each type of sand.
The results of cyclic shear tests on sand indicate the following features:

i) for both silica and calcareous sands, the reduction in shear stress during cyclic loading depends on the normal stress, cyclic horizontal displacement and void ratio, but the size and shape of the particles also have an effect for calcareous sand.

ii) for calcareous sand, the shear stress decreases as the number of cycles increases, but the rate of change decreases with increasing numbers of cycles.

iii) for a particular displacement amplitude, the rate of shear stress reduction for calcareous sand decreases with increasing void ratio.

vi) for similar relative densities of specimens, the results of sand–sand and sand–interface tests show that increasing normal stress, void ratio, displacement amplitude, and angularity of sand particles induce increasing compressibility of the sand. All the calcareous sands show greater compressibility than the silica sand.

The results of the tests to investigate particle damage due to grain crushing indicate that the coarse particles of calcareous soil are more prone to particle crushing than finer particles.

It is believed that the characteristics of behaviour revealed in these tests have important implications for the skin friction developed along piles in both silica and calcareous sands.
### Table 3.1 Static and Cyclic CNL Test Programme Summary

#### STATIC & CYCLIC TESTS

**Soil Shear Behaviour**
- Under These Conditions:
  - \( \rho_S = 6 \text{ & } 7 \text{mm} \)
  - \( \sigma_{OV} = 55, 110, 160 \text{kPa} \)

**Alluminium or Steel-Soil Interface**
- (3rd Group Tests)
  - \( \rho_S = 6 \text{ & } 7 \text{mm} \)
  - \( N_C = 50 \text{ Cycles} \)

**Crushing Particles**
- (4th Group Tests)
  - \( N_C = 50 \text{ Cycles} \)

#### Standard Tests (1st Group Tests)
- 1st Series
  - Each Sand was Tested
  - Dry Using:
    - \( \rho_C = 0.5, 1, 2, 5 \text{mm} \)
    - \( N_C = 5 \text{ to } 100 \text{ Cycles} \)

#### Non-Standard Tests (2nd Group Tests)
- 2nd Series
  - Saturated N.N.R Calcareous Sand Using:
    - \( \rho_C = 5 \text{mm} \)
    - \( N_C = 5 \text{ to } 30 \)

#### 3rd Series
- N.N.R Calcareous Sand Using:
  - G.size = Fine & Coarse
  - \( \rho_C = 5 \text{mm} \)
  - \( N_C = 5 \text{ to } 50 \text{ Cycles} \)

- 4th Series
  - Dry N.N.R & Silica Sands were Tested Using:
    - \( \rho_C = 5 \text{mm} \)
    - \( N_C = 50 \text{ Cycles} \)
    - \( t = 0.5, 1, 1.6, 2 \text{mm} \)

- 5th Series
  - N.N.R & Silica Sands were Tested Using:
    - \( \sigma_{OV} = 55, 110, 160 \text{kPa} \)

- 6th Series
  - O.N.R, 0.25 CSIRO & 0.5 CSIRO Sand were Tested Using:
    - \( \sigma_{OV} = 160 \text{kPa} \)

- 7th Series
  - N.N.R & Silica Sands were Tested Using:
    - \( \sigma_{OV} = 55, 160 \text{kPa} \)

**Notes:**
- \( \rho_S \) = Horizontal Displacement of Static Shear
- \( \rho_C \) = Horizontal Displacement of Cyclic Shear
- \( N_C \) = Number of Cycles
- \( t \) = Thickness of Shear Surface (i.e. \( t = 0 \text{ for Standard Tests \& } t > 0 \text{ for Non-Standard Tests} \))
- G.size = Grain Size of Soil
Table 3.2 Some Physical Properties of Carbonate and Non Carbonate Sediments
(Modified After Hull et al, 1988)

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Denoted By</th>
<th>$d_{10}$ $d_{60}$</th>
<th>$Cu - d_{60}/d_{10}$</th>
<th>$G_s$</th>
<th>Carbonate Content %</th>
<th>$\gamma_{d_{max}}$</th>
<th>$\gamma_{d_{min}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Silica (Sydney)</td>
<td>Silica</td>
<td>0.23</td>
<td>0.34</td>
<td>1.47</td>
<td>2.65</td>
<td>0</td>
<td>16.2</td>
</tr>
<tr>
<td>North Rankin (New)</td>
<td>N.N.R.</td>
<td>0.07</td>
<td>0.17</td>
<td>2.43</td>
<td>2.77</td>
<td>94</td>
<td>13.0</td>
</tr>
<tr>
<td>North Rankin (Old)</td>
<td>O.N.R.</td>
<td>0.08</td>
<td>0.36</td>
<td>4.5</td>
<td>2.72</td>
<td>91</td>
<td>13.4</td>
</tr>
<tr>
<td>Bass Strait</td>
<td>B.S.</td>
<td>0.13</td>
<td>0.60</td>
<td>4.6</td>
<td>2.73</td>
<td>88</td>
<td>17.7</td>
</tr>
<tr>
<td>Barry's Beach</td>
<td>B.B.</td>
<td>0.27</td>
<td>1.32</td>
<td>4.88</td>
<td>2.73</td>
<td>89</td>
<td>14.5</td>
</tr>
<tr>
<td>CSIRO 500μm</td>
<td>CSIRO</td>
<td>0.3</td>
<td>0.58</td>
<td>1.93</td>
<td>2.72</td>
<td>94</td>
<td>9.8</td>
</tr>
<tr>
<td>CSIRO 250μm</td>
<td>CSIRO</td>
<td>0.17</td>
<td>0.28</td>
<td>1.65</td>
<td>2.72</td>
<td>93</td>
<td>9.95</td>
</tr>
</tbody>
</table>

$Cu$ - Coefficient of crushing  
$G_s$ - Specific gravity  

**Note:** The maximum and minimum densities were from using the sand raining device, except for the N.N.R. and B.S. maximum densities which were obtained by vibration.
Table 3.3 Ranges of Sand Dry Density

<table>
<thead>
<tr>
<th>Sand Type</th>
<th>Density Definition</th>
<th>$\gamma_d$ kN/m$^3$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Calcareous</td>
<td></td>
<td></td>
</tr>
<tr>
<td>(N.N.R)</td>
<td>Loose</td>
<td>$&lt; 9.6$</td>
</tr>
<tr>
<td></td>
<td>Medium-Dense</td>
<td>9.6-10.5</td>
</tr>
<tr>
<td></td>
<td>Dense</td>
<td>$&gt; 10.5$</td>
</tr>
<tr>
<td>Silica</td>
<td></td>
<td></td>
</tr>
<tr>
<td>(Sydney)</td>
<td>Loose</td>
<td>$&lt; 14.5$</td>
</tr>
<tr>
<td></td>
<td>Medium-Dense</td>
<td>14.5-16.2</td>
</tr>
<tr>
<td></td>
<td>Dense</td>
<td>$&gt; 16.2$</td>
</tr>
<tr>
<td>Sand Type</td>
<td>Test No.</td>
<td>Density (kN/m³)</td>
</tr>
<tr>
<td>-----------</td>
<td>---------</td>
<td>----------------</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Silica</td>
<td>1</td>
<td>16.2</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>15.6</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>15.8</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>15.1</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>14.9</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>15.2</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>14.2</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>14.6</td>
</tr>
<tr>
<td></td>
<td>9</td>
<td>14.3</td>
</tr>
<tr>
<td>New North Rankin (N.N.R)</td>
<td>1</td>
<td>12.3</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>12.2</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>13.0</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>10.4</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>10.2</td>
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<tr>
<td></td>
<td>6</td>
<td>10.5</td>
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<tr>
<td></td>
<td>7</td>
<td>9.4</td>
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<tr>
<td></td>
<td>8</td>
<td>9.5</td>
</tr>
<tr>
<td></td>
<td>9</td>
<td>9.4</td>
</tr>
<tr>
<td>Old North Rankin (O.N.R)</td>
<td>1</td>
<td>13.3</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>12.8</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>11.9</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>12.1</td>
</tr>
<tr>
<td>Bass Strait (B.S)</td>
<td>1</td>
<td>15.2</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>17.2</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>16.7</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>14.0</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>14.2</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>13.2</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>13.4</td>
</tr>
</tbody>
</table>

D = Dense, M = Medium Dense, L = Loose

$\sigma_{ov}$ = Normal Stress

$\varphi_p$ & $\varphi_u$ = Friction Angle at Peak & Ultimate Stress (Degrees)

$\epsilon_o$ = Initial Void Ratio.
<table>
<thead>
<tr>
<th>Sand Type</th>
<th>Test No.</th>
<th>Density ( \text{kN/m}^3 )</th>
<th>( \sigma_{ov} ) (kPa)</th>
<th>Shear Stress (kPa)</th>
<th>( \phi_p )</th>
<th>( \phi_u )</th>
<th>( e_o )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Barry's Beach (B.B)</td>
<td>1</td>
<td>D 13.9</td>
<td>160 55</td>
<td>175 62</td>
<td>47.6</td>
<td>45.4</td>
<td>0.92</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>14.2</td>
<td></td>
<td>162.3 56.5</td>
<td>48.4</td>
<td>45.7</td>
<td>0.88</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>M 12.6</td>
<td>160 55</td>
<td>164.1 56.2</td>
<td>45.9</td>
<td>45.9</td>
<td>1.12</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>12.7</td>
<td></td>
<td>164.1 56.2</td>
<td>45.6</td>
<td>45.6</td>
<td>1.1</td>
</tr>
<tr>
<td>(0.5-CSIRO)</td>
<td>1</td>
<td>D 10.0</td>
<td>160 55</td>
<td>159.2 55.1</td>
<td>44.9</td>
<td>44.35</td>
<td>1.65</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>9.9</td>
<td></td>
<td>156.4 53.5</td>
<td>45.1</td>
<td>44.2</td>
<td>1.7</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>M 8.9</td>
<td>160 55</td>
<td>155.6 44.2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(0.25-CSIRO)</td>
<td>1</td>
<td>D 10.7</td>
<td>160 55</td>
<td>163 54.9</td>
<td>45.4</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>9.9</td>
<td></td>
<td>153.2 44.1</td>
<td>43.8</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

D = Dense, M = Medium Dense, L = Loose

\( \sigma_{ov} \) = Normal Stress

\( \phi_p \) & \( \phi_u \) = Friction Angle at Peak & Ultimate Stress (Degrees)

\( e_o \) = Initial Void Ratio.
<table>
<thead>
<tr>
<th>Sand Type</th>
<th>Test No.</th>
<th>R.D</th>
<th>$\sigma_{ov}$ (kPa)</th>
<th>Shear Stress (kPa)</th>
<th>$\phi_p$</th>
<th>$\phi_u$</th>
<th>$e_o$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$\tau_p$</td>
<td>$\tau_u$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Saturated New</td>
<td>1</td>
<td>D</td>
<td>160</td>
<td>158.4</td>
<td>44.7</td>
<td>43.7</td>
<td>0.99</td>
</tr>
<tr>
<td>North Rankin</td>
<td>2</td>
<td></td>
<td>110</td>
<td>115.5</td>
<td>46.4</td>
<td>43.9</td>
<td>0.98</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td></td>
<td>55</td>
<td>59.7</td>
<td>47.3</td>
<td>42.3</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>M</td>
<td>110</td>
<td>115.2</td>
<td>46.3</td>
<td>46.3</td>
<td>1.21</td>
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<td>55</td>
<td>47.2</td>
<td>40.6</td>
<td>40.6</td>
<td>1.15</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>L</td>
<td>110</td>
<td>103.1</td>
<td>43.1</td>
<td>43.1</td>
<td>1.7</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td></td>
<td>55</td>
<td>49.2</td>
<td>41.8</td>
<td>41.8</td>
<td>1.6</td>
</tr>
</tbody>
</table>

R.D = Relative Density (D = Dense, M = Medium Dense & L = Loose)

$\sigma_{ov}$ = Normal Stress

$\phi_p$ & $\phi_u$ = Friction Angle at Peak & Ultimate Stress (Degrees)

e_o = Initial Void Ratio.
<table>
<thead>
<tr>
<th>Sand Type</th>
<th>Test No.</th>
<th>R.D</th>
<th>$\sigma_{ov}$ kPa</th>
<th>Sand-Metal Interface</th>
<th>Aluminium</th>
<th>Steel</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$\tau_p$ (kPa)</td>
<td>$\tau_u$ (kPa)</td>
<td>$\delta_p$ deg.</td>
</tr>
<tr>
<td>Silica</td>
<td>1</td>
<td>D</td>
<td>160</td>
<td>97.2</td>
<td>94.3</td>
<td>31.27</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>110</td>
<td>55</td>
<td>67</td>
<td>64</td>
<td>31.24</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>55</td>
<td></td>
<td>31.27</td>
<td>30.9</td>
<td>29.6</td>
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<tr>
<td></td>
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<td>160</td>
<td>97</td>
<td>97</td>
<td>31.2</td>
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<td>64.6</td>
<td>64.6</td>
<td>30.4</td>
</tr>
<tr>
<td></td>
<td>9</td>
<td>55</td>
<td></td>
<td>32</td>
<td>32</td>
<td>30.2</td>
</tr>
<tr>
<td>New North Rankin N.N.R</td>
<td>1</td>
<td>D</td>
<td>160</td>
<td>111.2</td>
<td>109.4</td>
<td>34.8</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>110</td>
<td>55</td>
<td>69</td>
<td>68</td>
<td>32.1</td>
</tr>
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<td>3</td>
<td>55</td>
<td></td>
<td>36</td>
<td>35.6</td>
<td>33.2</td>
</tr>
<tr>
<td>O.N.R</td>
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<td>D</td>
<td>160</td>
<td>110</td>
<td>109.6</td>
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<tr>
<td></td>
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<td>L</td>
<td>160</td>
<td>106.5</td>
<td>105</td>
<td>33.6</td>
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<tr>
<td>0.25-CSIRO</td>
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<td>M</td>
<td>160</td>
<td>108.3</td>
<td>108.3</td>
<td>34.1</td>
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<td>L</td>
<td>160</td>
<td>91.7</td>
<td>91.7</td>
<td>29.8</td>
</tr>
<tr>
<td>0.50-CSIRO</td>
<td>1</td>
<td>M</td>
<td>160</td>
<td>102.8</td>
<td>102.8</td>
<td>32.7</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>L</td>
<td>160</td>
<td>100.5</td>
<td>100.5</td>
<td>32.1</td>
</tr>
</tbody>
</table>

**R.D** = Relative Density (D = Dense, M = Medium Dense & L = Loose Sand)

**$\sigma_{ov}$** = Normal Stress

**$\tau_p$ & $\tau_u$** = Shear Stress at Peak and Ultimate Stress

**$\delta_p$ & $\delta_u$** = Friction Angle at Peak & Ultimate Stress. (in Degrees)
Table 3.6 Crushing Coefficients for the Calcareous Sand after Cycling under a Normal Stress of 55 kPa

<table>
<thead>
<tr>
<th>Type of Particle</th>
<th>Size of Particle</th>
<th>$C_c$ after N Cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>1</td>
</tr>
<tr>
<td>Coarse</td>
<td>600µm - 2mm</td>
<td>3.1</td>
</tr>
<tr>
<td>Fine</td>
<td>&lt; 300mm</td>
<td>1.2</td>
</tr>
</tbody>
</table>

Table 3.7 Effect of Displacement Amplitude on Particle Damage under a Normal Stress of 55 kPa

<table>
<thead>
<tr>
<th>Type of Sand</th>
<th>Symbol</th>
<th>Coefficient of Crushing $C_c$ Before* Cycling</th>
<th>Overall Change of Crushing Coefficient ($\delta C_c$)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$\rho_C$ - 1mm</td>
<td>$\rho_C$ - 5mm</td>
</tr>
<tr>
<td>N.N.R.</td>
<td>2.43</td>
<td>2.54</td>
<td>2.82</td>
</tr>
<tr>
<td>O.N.R.</td>
<td>4.5</td>
<td>4.66</td>
<td>5.0</td>
</tr>
<tr>
<td>B.S.</td>
<td>4.6</td>
<td>5.36</td>
<td>6.0</td>
</tr>
<tr>
<td>B.B.</td>
<td>4.88</td>
<td>5.45</td>
<td>6.66</td>
</tr>
<tr>
<td>0.5CSIRO</td>
<td>1.93</td>
<td>3.17</td>
<td>5.86</td>
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<td>0.25CSIRO</td>
<td>1.65</td>
<td>2.2</td>
<td>2.80</td>
</tr>
</tbody>
</table>

* After Shearing to 6 mm.
Figure 3.1. Grading Curve for the Sands Before Testing
a) Rougher Zone of Steel Specimen

b) Rougher Zone of Aluminium Specimen

c) Rougher Zone of Stainless Steel Specimen

**Figure 3.3.** Micrographs Showing the Surface Roughness of Different Types of Metal
a) Interface Between Steel Plate and Silica Sand

b) Interface Between Steel Plate and Calcareous Sand

Figure 3.4. Interface Between Steel Plate and Silica, and Calcareous Sands
Figure 3.5. The Modified Direct Shear Box
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Direct Shear Box</td>
</tr>
<tr>
<td>B</td>
<td>Two Thin Steel Plates With Screws to Clamp The Strip of Rubber membrane</td>
</tr>
<tr>
<td>C</td>
<td>Steel Block</td>
</tr>
<tr>
<td>D</td>
<td>Aluminium Block</td>
</tr>
<tr>
<td>E</td>
<td>Upper Grooved Plate</td>
</tr>
<tr>
<td>F</td>
<td>One Pair of Steel Plates</td>
</tr>
<tr>
<td>G</td>
<td>Scoop</td>
</tr>
<tr>
<td>H</td>
<td>Aluminium Spatulas Used to level and control the specimen</td>
</tr>
<tr>
<td>I</td>
<td>450 mm High Square Cardboard Tube</td>
</tr>
</tbody>
</table>

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B) Lower grooved plate Placed underneath the specimen
C) Strip of rubber membrane
D) Thin steel plate and screws to clamp the rubber membrane on the top surface of the shear box
E) A pair of steel plates: Their thicknesses represent the gap width
F) Horizontal line where the front and the back forces located
G) Steel balls: Their diameters changed with changing gap width
H) Steel rollers connected by steel strips

Figure 3.7a. Schematic Diagram of Shear Box
A) Wall of shear box
B) Lower grooved plate placed underneath the specimen
C) Strip of rubber membrane
D) Thin steel plate and screws to clamp the rubber membrane on the top surface of the shear box
E) A pair of steel plates: Their thicknesses represent the gap width
F) Horizontal line where the front and the back forces located
G) Steel balls: Their diameters change with changing gap width
H) Steel rollers connected by steel strips

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- Numerals denote cycle No.
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- Numerals denote cycle No.
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Chapter 4

ESTIMATION OF PILE FRICTION DEGRADATION FROM SHEAR BOX TESTS

4.1 INTRODUCTION

The extensive experimental results of static and cyclic direct shear box constant normal load (CNL) tests for different types of carbonate and non-carbonate sands have been presented in the previous chapter.

This chapter presents the results obtained from a series of two-way cyclic shear box tests on very dense calcareous sand samples under both constant normal load (CNL) and constant normal stiffness (CNS) conditions. It is demonstrated that the reduction in shear stress in the CNS tests can be predicted from the results of the standard CNL tests. Thus it is suggested that standard shear box tests may be used to estimate the degradation of pile skin friction during cyclic pile loading.

In this chapter the modification of the direct shear box to perform shearing under constant normal stiffness (CNS) conditions is described, and the operation of the
apparatus is also discussed.

The relationship between the CNL test and CNS test requires consideration of the change in volume occurring due to changing normal stress. Thus a number of oedometer tests have been carried out to measure the volume compressibility of the soil tested, and the results of the tests are presented herein.

Table 4.1 summarizes the results of the tests carried out.

4.2 CONCEPT OF CONSTANT NORMAL STIFFNESS TESTS

The conventional method of testing interface behaviour has been the direct shear test with one of the important aims being to determine the peak shear strength of the interface. It is usual for constant normal load (CNL) to be applied across the interface while the behaviour under steadily increasing shear load or relative shear displacement is observed. However, there exists a class of practical problems where normal stress acting on the interface may not remain constant as sliding occurs. The dilation or contraction of the soil or rock during shearing results in a change in the normal stress.

The relationship between dilation of a granular material and its shear stress have been studied through direct shear testing under constant volume conditions, by Boyce and Kulhawy 1983. The authors found that during shearing of the dense samples, the normal stress on the failure surface increased to a critical value.

The piles used to support offshore structures tend to be long and relatively flexible, with the majority of their load capacity generated from frictional forces between the
pile and the surrounding soil. When subjected to cyclic loading, from the action of wind and waves, the skin friction mobilised on piles in sandy soils has been found (e.g. Poulos 1988a) to decrease. This behaviour has been observed in model pile and full scale cyclic loading tests, but has proved difficult to predict from the results of conventional laboratory tests.

In calcareous soil, the degradation in the skin friction is associated with normal stress reductions resulting from both crushing of particles and compressive strains in the soil adjacent to the pile.

Analytical approaches to load–settlement prediction, particularly the load–transfer approach, are based on the results of laboratory model pile and field pile tests. Several attempts have been made to find an alternative to pile model tests. One such alternative is the direct shear test using the concept of constant normal stiffness conditions. In this type of test, the effects of expansion or contraction at the pile–soil interface are considered by using a spring to apply the normal load (Johnston et al 1987). The appropriate spring stiffness \( K \) is given by:

\[
K = 4G/d \\
\text{( 4.1 )}
\]

Where

\[G = \text{shear modulus of soil and} \]
\[d = \text{pile diameter.}\]

In the case of a pile grouted into a cemented soil or rock formation, the normal stress at the interface during shearing may be altered by dilation and contraction due to the radial movement of the rock or soil surrounding the pile (Johnston, Lam and Williams (1987) and Ooi, Carter and Boey (1988)). A constant normal stiffness device, described by Ooi and Carter (1987), is capable of applying both monotonic
and cyclic loading to examine the interface behaviour of piles in soft rocks, such as sandstone and calcarenite. However, in its present form, it can not be used for uncemented soil samples.

The results from Ooi and Carter (1987) show that the normal stiffness can have a large influence on the shear strength of the interface. There are also indications that the CNS direct shear test has some practical application in the design of grouted pile foundations (Jewell and Andrews, 1988; Jewell and Khorshid, 1988).

CNS tests have also been used for uncemented sand by Boulon and Foray (1986) to investigate pile response. Boulon and Foray suggested that CNS tests are relevant to the behaviour of piles in uncemented sands, as in this case most of the soil deformation occurs in a narrow band close to the pile and outside this band the soil can be considered to be elastic with a fixed shear modulus.

Boulon et al, 1988 have used the results of monotonic CNS tests to predict pile response during monotonic loading. Airey et al (1990) have reported several CNS direct shear tests on a Bass Strait calcareous sand.

4.3 SCOPE OF TESTS

The laboratory test programme in this chapter includes several oedometer tests, one series of constant normal load (CNL) tests, and finally one series of constant normal stiffness (CNS) tests.

The spring used for all the CNS tests gave a normal stiffness of 1600 kPa/mm. This stiffness was estimated to be appropriate for the model pile tests based on a
representative value of soil modulus and pile diameter as described in Chapter 7. The following variable factors for the direct shear tests were considered:

(1) The horizontal cyclic shear displacement $\rho_c$ = ± 0.5, ± 1, ± 2, ± 5 mm.

(2) The effective normal stress $\sigma_n' = 50, 100, 150, 200, 250$ and 400 kPa.

(3) Number of shear cycles $N_c$ between 30 and 80 cycles.

The differences between the cyclic constant normal load (CNL) tests performed in this chapter and those in the previous chapter are as follows:

a) the CNL tests which have been reported in Chapter 3 had lower normal stresses (55, 110, 160kPa);

b) the tests described herein were carried out on denser samples of sand;

c) For the cyclic CNL tests in Chapter 3 the samples were sheared monotonically to a shear displacement of about 6mm and then cycled between a given shear displacement, whereas in the tests described in this Chapter the cyclic loading was applied directly without initial monotonic sheering.

4.4 MODIFICATION OF CNL TEST DEVICE

The device used in these tests was a conventional shear box with sample dimensions
60 x 60 x 20 mm, that had been modified to carry out static and cyclic loading tests under CNL conditions as described in Chapter 3.

Further modification was carried out on this device to perform constant normal stiffness (CNS) tests, as described by Airey et al (1990). Figure 4.1 shows a photograph of the CNS shear box device on the steel frame. The load cell, the base frame supporting the spring, the steel hanger, and the screw above the sample are shown in this photo. In CNL tests the normal load is applied via a steel hanger by dead weights. The modification made to the steel hanger was the addition of a spring beam. The new hanger enables the CNS condition to be applied to the shear box sample as shown in Figure 4.2. The steel hanger was manufactured with rods that were chosen to be sufficiently long enough to limit the possibility of normal stress change accompanying the slight movement of the top of the shear box.

The constant normal stiffness was provided by the hanger rods and a strain gauged spring beam, as shown in Figure 4.3, attached to the support frame, which could be pre-loaded by turning the screw above the sample. This Figure shows the four strain gauges positioned above and below the plate to form a full Wheatstone Bridge circuit, this arrangement measures the bending of the spring beam. A specially cut groove on the top of the spring beam ensured that the contact beam was positioned mid-way between the strain gauges. With this arrangement dead-weights could still be applied to the hanger. The loading or unloading of the hanger and spring beam contributed to the actual stiffness of the top plate of the shear box. The stiffness of the system could be conveniently varied by changing the spring beam dimensions. The spring stiffness $K$ is related to the dimensions of the spring by:

$$4Ebt^3$$

$$K = \frac{\ldots}{t^3} \quad (4.2)$$
Where

\[ E = \text{elastic modulus of the steel spring beam (200000 MPa)} \]

\[ b = \text{width of spring beam} \]

\[ t = \text{thickness of spring beam} \]

\[ L = \text{the distance between the two support points on the spring beam}. \]

The stiffness of the system was best obtained by calibration rather than summing the effects of the individual component. The spring beam was calibrated against a load cell to read the load on the sample. By using different thicknesses of the spring beam a range of constant normal stiffnesses can be applied.

The shear stress was measured by an instrumented proving ring (LVDT transducer) and both vertical and horizontal displacements were measured by displacement transducers (LVDT), as described in Chapter 3. The readings of these three transducers and the strain-gauged spring were all recorded by a computer system. Otherwise, the operation of the apparatus was similar to that used for the CNL tests described previously.

### 4.5 PREPARATION OF SAMPLES AND TEST PROCEDURE

The preparation of dense samples of sands which were tested under constant normal stiffness conditions was slightly different than that used for the CNL tests described in Chapter 3. Here, the samples were prepared as dense as possible by compacting with a tamping rod and lightly hammering the metal block which was placed on the sand sample. The method used for measurement of initial void ratio for each sample was similar to that for the CNL tests, reported in Chapter 3. The final void ratio could also be determined by measuring the volume of sample after testing.
Before the hanger was placed, the computer and the data acquisition software was initialised. All the transducers and strain gauges which were attached to the spring beam were zeroed. The vertical displacement of each sample was measured by the vertical displacement transducer (LVDT) during placing the required dead weights.

The spring beam was calibrated before testing the samples, and a linear calibration was found. It was therefore assumed that the behaviour of the load cell was linear during the static and cyclic shear tests.

4.6 RESULTS

In this section the results of oedometer tests are presented first followed by the results for CNS tests under monotonic loading. Finally, the results from cyclic shear stresses developed during (CNL) and (CNS) direct shear tests are presented finally.

4.6.1 Oedometer Results

One series of conventional oedometer tests were carried out on calcareous sand samples of 12.5 mm initial thickness. The tests were carried out for two different sample diameters 25 mm and 50 mm. The measurements of vertical displacement were made by using a displacement transducer (LVDT) which was fixed above the samples and connected to the Hewlett Packard computer–controlled data acquisition system.

The results of the oedometer tests are summarised in Table 4.2. The values of both compression index \(C_C\) and swelling index \(C_S\) for a stress range from 150 to 500
kPa are shown in this table. The values of initial density are also shown in this table. Figure 4.4 shows the resulting void ratio versus normal stress relationships for a typical test.

4.6.2 Monotonic Shear Test Results

It was felt important to study the monotonic as well as the cyclic behaviour of the soil under constant normal stiffness (CNS) conditions in order to:

a) examine the difference between the value of ultimate shear stress from this type of test and that obtained from the more conventional constant normal load (CNL) tests,

b) determine the value of degradation factor "$D_\tau$" for cyclic shear tests,

c) assess the influence of the sand density on shear behaviour as obtained from CNS tests.

Table 4.3 summarizes the results of static tests obtained from "dense" and "loose" calcareous sands tested under CNS conditions with a spring stiffness of approximately 1600 kPa/mm.

The influence of initial density on the shear stress developed during shear for two types of calcareous sands tested under CNS conditions is displayed in Figure 4.5. The development of shear stress with horizontal displacement for dense and loose samples is shown in Figure 4.5a, while the associated vertical displacement of the samples are shown in Figure 4.5b.
The shear stress for both types of sands increases to a peak value and then decreases to approach an ultimate state. The strength of Bass Strait (B.S) sand is slightly higher than that of N.N.R sand consistent with the higher dilation of dense B.S sand as shown in Figure 4.5b. For loose samples no peak is observed and the value of ultimate shear stress of both sands is about the same. The volume strain for B.S sand is larger than N.N.R sand as shown in Figure 4.5b. The values of the ultimate shear stress for denser samples are significantly higher than the maximum values of loose samples. These observations are similar to those reported by Airey et al (1990).

Figure 4.6 shows the influence of initial normal stress on the shear behaviour of dense samples under CNS conditions. For both B.S and N.N.R calcareous sands, the shear stresses rise to a peak and then approach an ultimate value, and the sands dilate during shear. The values for both peak and the observed ultimate shear stress increase with increasing initial normal stress.

The relationship between normal stress and shear stress for dense and loose samples is displayed in Figure 4.7. The static shear stresses for dense samples of both types of calcareous sand are greatly affected by constant normal stiffness condition, while those sand samples which were loose can be seen to be little affected. For dense samples, the shear stress increases proportionally to the normal stress from about 200 to 600 kPa as the normal stress increases from about 220 to 550kPa, owing to the dilation of the dense sand during shearing. The slope of the line represents the ratio of the ultimate shear stresses to normal stresses, which was obtained from CNL tests. The slope of the line is about 1.09. Figure 4.7 shows that the ultimate stresses from the CNS tests agree with those obtained from CNL tests.
A comparison between typical results from CNS tests and those from the CNL tests both with an initial normal stress of 250 kPa are presented in Figure 4.8. For dense samples, the peak shear stress from the CNS tests is higher than that from the CNL test, while for the loose samples the value of ultimate shear stresses from the CNS test is about half of that from the CNL tests. Figure 4.8a shows that the strain softening for the dense samples obtained from CNL tests is significantly greater than those from CNS tests. Similar behaviour has been observed by Airey et al (1990).

4.6.3 Cyclic Test Results

Table 4.4 summarizes the results of cyclic tests obtained from "dense" calcareous sands under the CNL condition. The shear stresses developed in these tests are generally slightly higher than for the tests described in Chapter 3 because the densities are higher.

The results of corresponding CNS cyclic tests for "dense" calcareous sands using a spring stiffness of 1600 kPa/mm are shown in Table 4.5. Density, number of cycles, horizontal cyclic displacement, vertical displacement, and shear stress degradation factor are shown in this Table. The degradation factor \( D_\gamma \) is defined as post cyclic shear stress (measured beyond 6mm displacement), divided by the average maximum static shear stress obtained from the monotonic tests.

Figures 4.9 and 4.10 show the variations of shear stress and vertical displacement for a dense sample (having an initial unit weight of 17.47 kN/m\(^3\)), with horizontal displacement from a standard cyclic shear box test and with a constant normal stress of 250 kPa. The maximum shear stress attained in each cycle remains practically constant, and the stress versus displacement curves rapidly reach a repeatable response. In the first cycle the soil expanded, as expected for tests on dense sand,
but all subsequent cycles saw a net contraction, with the rate of contraction decreasing with increasing numbers of cycles.

Figure 4.11 shows the variations of the vertical displacements measured at the maximum shear displacement with cycle number for "CNL" tests with a range of normal stresses all using a cyclic displacement of +/- 1 mm. As the normal stress increases there is less expansion in the first cycle and there are greater net contractions in the subsequent cycles.

Figure 4.12 shows the vertical displacement at the maximum shear displacement plotted against cycle number for "CNL" tests with the same normal stress of 150 kPa but different cyclic displacements. Increasing cyclic displacement is associated with greater expansion in the first cycle, and greater contractions in the subsequent cycles. The same patterns of behaviour with varying cyclic displacements and different normal stresses are repeated irrespective of the type and relative density of the sand as reported in Chapter 3, and Poulos (1988a). However, as the relative density decreases the expansion in the first cycle decreases and the net contraction in the subsequent cycles becomes greater.

Figures 4.13 and 4.14 show the variations of shear stress and normal stress with horizontal displacement from a CNS cyclic shear box test using a constant normal stiffness of 1600 kPa/mm. The maximum shear stress decreased rapidly in the first few cycles and then more slowly until reaching a limit after about 45 cycles. It can be seen from Figure 4.14 that the reduction in shear stress is associated with a rapid drop in the normal stress. Owing to the method used to apply the CNS condition (Airey et al 1990) the normal stress could not drop below 30 kPa, the limiting value in Figure 4.14, but further vertical contractions could and did occur.
Figure 4.15 shows the relation between the shear and normal stresses during this CNS test. It is noticeable that the normal stress drops rapidly after stress reversal, but otherwise the response is very similar to that obtained in undrained cyclic triaxial tests. The variations of the normal stress at the maximum shear displacement with cycle number in CNS tests performed with different cyclic displacements are shown in Figure 4.16. These plots show the normal stress increasing in the first cycle, and then decreasing in subsequent cycles, similar to the trend of vertical displacement in the CNL tests.

4.7 RELATIONSHIP BETWEEN CNL AND CNS TESTS

The similarity of Figures 4.10 and 4.14 suggests that it should be possible to predict the decrease in normal stress in a CNS test from the vertical displacements measured in the CNL tests. However, the CNL results can not be used directly because allowance must be made for the oedometric change in volume (height) occurring due to the normal stress change, and for the effects of changes in normal stress on the change in vertical displacement during any given cycle (see Figure 4.11).

As described above, one-dimensional load-unload tests on the sand were performed to determine the swelling and compression indices, giving average values of $C_S = 0.0076$ and $C_C \approx 0.02$ for an initial density of about 17.4 kN/m$^3$ and the stress range of interest. The following analysis is only concerned with the normal stress and shear stress changes at the maximum shear displacement in each cycle, as these are of most interest in predicting pile performance. The same procedure could nevertheless be applied to every point during a load cycle.
The following procedure has been used:

1. From Figure 4.11 determine the change in vertical displacement $dy$ for the next cycle in the CNL test given the current normal stress $\sigma_n$.

2. Calculate the change in normal stress $\Delta \sigma_n$ in the CNS test that would result from the change in vertical displacement $dy$ less the oedometric change in height associated with the change $\Delta \sigma_n$.

\[
\frac{\Delta \sigma_n}{K} = dy - m_v \ h \ \Delta \sigma_n \quad \ldots \ldots \quad (4.3)
\]

where

- $K$ = spring stiffness
- $dy$ = incremental vertical displacement from the CNL test data
- $h$ = sample height
- $m_v = C / (2.303 \times (1+e) \ \sigma_n)$
- $e$ = void ratio
- $C = C_c$ or $C_s$ depending on whether the stress is increasing or decreasing respectively,
- $C_c$ = compression index
- $C_s$ = swelling index

This can be re-arranged to give:
\[
\Delta \sigma_n = \frac{K dy}{1 + m_v K h} \quad \ldots \ldots \quad (4.4)
\]

and hence the value of \( e \) for the next cycle can be determined.

3. Assume that the cumulative vertical displacement from the start of cycling \( y = \Sigma \Delta \sigma_n / K \) is the controlling parameter. The starting position on Figure 4.11 for the next cycle is then defined by \( y \) and the new normal stress \( \sigma_n \).

4. Return to step 1 and repeat.

Figure 4.17 shows the predicted variation in normal stress during a CNS test with a cyclic displacement of +/- 1 mm using the CNL data shown in Figure 4.11. This compares favourably with the measured response in the CNS test. Alternative assumptions made at step 3, making no allowance for the effect of changing normal stress on the incremental vertical displacements and taking the number of cycles of loading to control the response, caused the normal stress to drop off much more quickly than was observed, and demonstrated the importance of allowing for the changes in normal stress on the soil displacement.

To predict the reduction in the maximum shear stress during cyclic loading requires a relationship between the shear and normal stresses in addition to the prediction of the normal stress reduction. The simplest relationship is:

\[
\tau = \sigma_n' \tan \varphi' \quad \ldots \ldots \quad (4.5)
\]

where the critical state angle of friction is used for \( \varphi' \). From monotonic CNL tests
a value of 44° has been obtained for the critical state angle $\varphi'$. The variation of shear and normal stresses during a CNS test with +/- 1 mm displacement has been shown in Figure 4.15. Lines showing the critical state ratio have been drawn on Figure 4.15, from which it can be seen that the maximum shear stress in every cycle exceeds the critical state value (as expected as the sample was dilating at the maximum displacement).

Nevertheless using Equation 4.5 should give a conservative estimate of the shear stress that can be mobilised in any cycle, and this is demonstrated in Figure 4.18 which shows the predicted variation of the maximum shear stress using the normal stress prediction from Figure 4.17 and equation 4.5. From Figures 4.17 and 4.18, it can be seen that quite reasonable predictions of the CNS test results can be obtained from the CNL tests when using the assumptions discussed above.

4.8 APPLICATION TO PILES

It has been argued by Boulon and Foray (1986), Boulon et al (1988), and Boulon and Nova (1990) that the results of CNS tests are directly applicable to the monotonic behaviour of piles. Tests on model piles in cemented (Allman, 1988) and uncemented calcareous sands (Lee, 1988 and Chapter 6) have shown that the shear stress mobilised during displacement controlled cyclic loading can reduce significantly, an observation consistent with the CNS test results.

Figure 4.19 shows the decrease in skin friction normalised by the value in the first cycle ("$D_f$" which was defined previously in Chapter 3) for displacement controlled ( +/- 1 mm) axial cyclic loading. $D_f$ decreases significantly in the first 10 cycles and then more slowly until reaching a limit after about 40 cycles.
Before attention is focussed on the correlation between the degradation factors $D_\tau$ obtained from CNS tests and model pile tests, the prediction using CNL tests results of the effect of cyclic loading on the degradation factors "$D_\tau$" for the skin friction of piles is discussed here.

The degradation factor ($D_\tau$) defined as:

$$D_\tau = \frac{f_C}{f_S} \quad \ldots \ldots \quad (2.6)$$

Where

- $f_C$ is skin friction after cyclic loading, and
- $f_S$ is maximum skin friction under static loading (first cycle).

The value of $D_\tau$ depends on the influence of cyclic loading on the skin friction. $D_\tau = 1$ when cyclic loading has no influence on the skin friction; $D_\tau < 1$ when reduction in skin friction due to cyclic loading occurs, and $D_\tau > 1$ when cyclic loading increases skin friction (this may occur in the first few cycles for loose sand under small amplitude cyclic loading).

Figure 4.20 shows the degradation factor ($D_\tau$) plotted against cyclic displacement ($\rho_C$) for the tests (CNS1 to CNS7). The highest rate of degradation occurs for small cyclic displacement and then the rate reduces for large $\rho_C$ particularly in the $\pm \rho_C = 5 \text{ mm}$. This figure shows a good agreement between the results of the CNS test and the results of model pile tests. Further details of the model pile tests are given in Chapter 6.

For the dense sand used in these tests the soil formation stiffness $K$ was estimated to be approximately 1600 kPa/mm, the same as used in the CNS tests. The average reduction of skin friction along the pile is similar to that observed in the CNS tests, also shown in Figure 4.19. In the pile test the normal stress acting on
the pile will depend significantly on the stress changes occurring during installation of the pile, an aspect of behaviour which is still poorly understood for piles in sandy soils.

As the normal stress has been shown to significantly affect the soil contractions during cycling it is clear that some care is required in applying the results of CNS tests to piles. Nevertheless, the CNS and CNL tests do allow the effects of different pile diameters and soil moduli, both of which affect the formation stiffness \( K \), to be investigated.

Using the CNL data and the procedure described above the effects of different spring stiffnesses can be investigated. As the stiffness, \( K \), increases the change of the normal stress in the first cycle will increase, and the subsequent rate of decrease of the normal and shear stresses will also increase. However, because the curves shown in Figure 4.11 appear to level off at large numbers of cycles the normal stress may be expected to approach the same ultimate normal stress, irrespective of the stiffness.

The effect of increasing the pile diameter will be to reduce the effective stiffness, and hence to reduce the magnitude and rate of decrease of the shear and normal stresses during cycling. However, it can be expected that the ultimate shear stress (skin friction) will approach a unique value, irrespective of the diameter, because the normal stress is expected to approach a unique value.

Soil modulus depends on the mean effective stress so that even in a soil with uniform density a more rapid loss of skin friction may be expected as the depth, and hence stress level, increases. As the density of the soil reduces then much greater contractions can be expected in the soil adjacent to the pile leading to a
more rapid drop in skin friction, and lower ultimate shear and normal stresses.

Tests on silica sands (e.g. Chapter 3) have shown similar trends to those discussed above for calcareous soil and thus piles in silica sands may be expected to experience similar decreases in normal stress and skin friction during cycling.

It is not expected that the results from the cyclic tests can give accurate predictions of the pile behaviour because the pile installation process, which affects the initial stress and strain state adjacent to the pile, cannot be modelled in the small shear box. Further, sand has been used in both halves of the shear box, thus not modelling the interface correctly. However, the trends evident in these tests have been observed when the soil is first sheared to its ultimate state before cycling, as shown in Chapter 3. It is therefore believed that CNL cyclic, tests can be used to give an indication of the relative magnitudes of the loss of skin friction, for instance between tests at different relative densities, and between different sands.

4.9 SUMMARY AND CONCLUSIONS

The first objective of this chapter was to estimate the pile friction degradation from direct shear box tests, while the second objective was to make a correlation between the CNL tests and CNS tests. Consequently two series of CNL and CNS tests were conducted using the direct shear apparatus. Oedometer tests were performed to assess the relationship between CNL tests and CNS tests.

The main features from the study in this chapter were:

1) A simple modification was performed on the existing shear box apparatus to
enable CNS tests to be carried out on uncemented soil samples.

2) The CNS tests allowed a good understanding of the effects of cyclic loading on shear stress degradation for calcareous sand, and were consistent with the results from CNL tests performed in this and the previous Chapter. These results are similar to those reported for CNS cyclic simple shear tests by Morrison et al (1988).

3) The constant normal stiffness tests have indicated that most of the degradation of skin friction occurred in the first few cycles. This pattern of behaviour is similar to that shown for model and full scale pile tests.

4) A simple procedure has been demonstrated for estimating the response of constant normal stiffness shear tests from the results of conventional constant load shear box and oedometer tests.

5) The CNS test can give reasonable estimates of the behaviour of piles in sand, taking into account both the stiffness of the surrounding soil and the diameter of the pile. The degradation of pile skin friction can be demonstrated from simple cyclic CNS tests.

6) It is not expected that this method will necessarily give good predictions of the skin friction adjacent to real piles which will depend on the normal stress resulting from the method of installation. However, it will allow comparison between different soils, and give an indication of the relative loss of skin friction under different conditions.
Table 4.1. Summary of Test Programme for Dense N.N.R and B.S Sands

Tests

- Oedometer Tests (4 Tests)
- Static CNS Tests (13 Tests)
- CNL Tests (8 Tests)
- CNS Tests (6 Tests)

* \( \sigma_n = 50, 150, \text{ and } 250 \text{ kPa} \)
* \( K = 1600 \text{ kPa/mm} \)

The Sand Was Tested as Dry Using:
* \( \rho_C = 0.5, 1, 2, 5 \text{ mm} \)
* \( N_c = 30 \text{ to } 75 \text{ Cycles} \)
* \( \sigma_n = 50, 150, \text{ and } 250 \text{ kPa} \)

Notes:

- \( \rho_C \) = Horizontal Displacement of Cyclic Shear
- \( N_c \) = Number of Cycles
- \( \sigma_n \) = Normal Stress
- \( K \) = Spring Stiffness
Table 4.2  Summary of Oedometer Test Results

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Initial Density kN/m³</th>
<th>Initial Void Ratio</th>
<th>Compr. Index (Cc)</th>
<th>Swel. Index (Cs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>17.47</td>
<td>0.53</td>
<td>0.02</td>
<td>0.0076</td>
</tr>
<tr>
<td>2</td>
<td>17.54</td>
<td>0.52</td>
<td>0.019</td>
<td>0.0074</td>
</tr>
<tr>
<td>3</td>
<td>17.1</td>
<td>0.56</td>
<td>0.0202</td>
<td>0.0079</td>
</tr>
<tr>
<td>4</td>
<td>16.63</td>
<td>0.605</td>
<td>0.023</td>
<td>0.0095</td>
</tr>
</tbody>
</table>
Table 4.3  Static CNS Test Results (Spring Stiffness *K* = 1600 kPa/mm)

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Soil Type</th>
<th>Type of Density</th>
<th>Density (kN/m²)</th>
<th>Initial Normal Stress (kPa)</th>
<th>Peak Shear Stress (kPa)</th>
<th>Ult. Shear Stress (kPa)</th>
<th>Vertical Disp. (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CNS1</td>
<td>N.N.R</td>
<td>D</td>
<td>11.8</td>
<td>55</td>
<td>318.5</td>
<td>292.6</td>
<td>-</td>
</tr>
<tr>
<td>CNS2</td>
<td>N.N.R</td>
<td>D</td>
<td>12.5</td>
<td>150</td>
<td>465.6</td>
<td>431.5</td>
<td>+0.22</td>
</tr>
<tr>
<td>CNS3</td>
<td>N.N.R</td>
<td>D</td>
<td>12.2</td>
<td>250</td>
<td>585</td>
<td>526</td>
<td>+0.27</td>
</tr>
<tr>
<td>CNS4</td>
<td>N.N.R</td>
<td>L</td>
<td>9.5</td>
<td>55</td>
<td>43.8</td>
<td></td>
<td>-0.02</td>
</tr>
<tr>
<td>CNS5</td>
<td>N.N.R</td>
<td>L</td>
<td>9.4</td>
<td>150</td>
<td>118</td>
<td></td>
<td>-0.03</td>
</tr>
<tr>
<td>CNS6</td>
<td>N.N.R</td>
<td>L</td>
<td>9.5</td>
<td>250</td>
<td>163</td>
<td></td>
<td>-0.03</td>
</tr>
<tr>
<td>CNS7</td>
<td>B.S</td>
<td>D</td>
<td>17.4</td>
<td>55</td>
<td>315.4</td>
<td>302.3</td>
<td>+.23</td>
</tr>
<tr>
<td>CNS8</td>
<td>B.S</td>
<td>D</td>
<td>16.9</td>
<td>150</td>
<td>478</td>
<td>457.4</td>
<td>0.3</td>
</tr>
<tr>
<td>CNS9</td>
<td>B.S</td>
<td>D</td>
<td>16.5</td>
<td>250</td>
<td>605</td>
<td>562</td>
<td>+0.26</td>
</tr>
<tr>
<td>CNS10</td>
<td>B.S</td>
<td>L</td>
<td>13.1</td>
<td>55</td>
<td>46</td>
<td></td>
<td>-0.07</td>
</tr>
<tr>
<td>CNS11</td>
<td>B.S</td>
<td>L</td>
<td>13.25</td>
<td>150</td>
<td>110</td>
<td></td>
<td>-0.1</td>
</tr>
<tr>
<td>CNS12</td>
<td>B.S</td>
<td>L</td>
<td>13.4</td>
<td>250</td>
<td>172</td>
<td></td>
<td>-0.08</td>
</tr>
<tr>
<td>CNS13</td>
<td>B.S</td>
<td>D</td>
<td>17.3</td>
<td>250</td>
<td>663</td>
<td>637</td>
<td>+0.28</td>
</tr>
<tr>
<td>Test No.</td>
<td>Density (kN/m³)</td>
<td>Normal Stress (kPa)</td>
<td>No. of Cycles (Nc)</td>
<td>Horizontal Displac. (mm)</td>
<td>Vertical Displac. after Cycling (mm)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>----------</td>
<td>-----------------</td>
<td>---------------------</td>
<td>-------------------</td>
<td>------------------------</td>
<td>-------------------------------------</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CNL1</td>
<td>17.36</td>
<td>250</td>
<td>45</td>
<td>1</td>
<td>-0.17</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CNL2</td>
<td>17.45</td>
<td>50</td>
<td>55</td>
<td>1</td>
<td>+0.28</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CNL3</td>
<td>17.39</td>
<td>100</td>
<td>45</td>
<td>1</td>
<td>+0.16</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CNL4</td>
<td>17.1</td>
<td>400</td>
<td>30</td>
<td>1</td>
<td>-0.42</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CNL5</td>
<td>17.33</td>
<td>150</td>
<td>80</td>
<td>1</td>
<td>-0.18</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CNL6</td>
<td>17.46</td>
<td>150</td>
<td>75</td>
<td>2</td>
<td>+0.09</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CNL7</td>
<td>17.28</td>
<td>150</td>
<td>60</td>
<td>0.5</td>
<td>-0.02</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CNL8</td>
<td>17.32</td>
<td>150</td>
<td>50</td>
<td>5</td>
<td>-0.41</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Table 4.5 Cyclic CNS Test Results (Initial Normal Stress = 250kPa and Spring Stiffness $K^* = 1600$ kPa/mm)

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Density $\text{kN/m}^3$</th>
<th>No. of Cycles (Nc)</th>
<th>Horizontal Displac. (mm)</th>
<th>Vertical Displac. after Cycles (mm)</th>
<th>Shear Stress at Pre-strain $^{<em>}F_s^</em>$ (kPa)</th>
<th>Post Cyclic Shear Stress $^{<em>}F_c^</em>$ (kPa)</th>
<th>$D_r^*$ = $F_c/625$</th>
</tr>
</thead>
<tbody>
<tr>
<td>CNS1</td>
<td>17.3</td>
<td>45</td>
<td>1</td>
<td>-0.36</td>
<td>386</td>
<td>312</td>
<td>0.5</td>
</tr>
<tr>
<td>CNS2</td>
<td>17.32</td>
<td>30</td>
<td>2</td>
<td>-0.44</td>
<td>490</td>
<td>125</td>
<td>0.2</td>
</tr>
<tr>
<td>CNS3</td>
<td>17</td>
<td>58</td>
<td>0.5</td>
<td>-0.03</td>
<td>260</td>
<td>587</td>
<td>0.94</td>
</tr>
<tr>
<td>CNS4</td>
<td>16.9</td>
<td>50</td>
<td>1</td>
<td>-0.39</td>
<td>361</td>
<td>238</td>
<td>0.38</td>
</tr>
<tr>
<td>CNS5</td>
<td>17.15</td>
<td>50</td>
<td>0.5</td>
<td>-0.01</td>
<td>245</td>
<td>490</td>
<td>0.79</td>
</tr>
<tr>
<td>CNS6</td>
<td>17.4</td>
<td>41</td>
<td>5</td>
<td>-0.43</td>
<td>625</td>
<td>87.5</td>
<td>0.14</td>
</tr>
<tr>
<td>CNS7</td>
<td>17.2</td>
<td>30</td>
<td>5</td>
<td>-0.46</td>
<td>620</td>
<td>107</td>
<td>0.17</td>
</tr>
</tbody>
</table>

* Degradation factor $D_r^*$ is Post Cyclic Shear Stress Measured beyond 6 mm ($F_c$) Divided by Average Maximum Static Shear Stress (about 625kPa).
A = Load Cell
B = Base Frame Support for Spring Beam
C = Rigid Hanger Frame
D = Screw Above the Sample

Figure 4.1 Photograph of the CNS Apparatus
Figure 4.2 Detail Drawing of the CNS Shear Box Device

A = Shear box
B = Loading Frame
C = Spring Beam
D = 4 Steel rods of 12mm diam. used to clamp a spring beam and make the beam work as simply supported
E = Top plate of steel hanger
F = 2 Steel rods of 12mm diam. work as hanger frame
G = Bottom plate of steel hanger
H = Weight hanger
I = Turn screw

All Dimensions in mm
(a) Top View of the Spring Beam

(b) Side View of the Spring Beam Showing the Layout of the Strain Gauges

(c) Circuit Diagram Showing the 4 Active Gauge Configuration of the Wheatstone Bridge

Figure 4.3 Strain Gauge Configuration For the Spring Beam
Figure 4.4  Typical Curve for Relationship Between Void Ratio and Normal Stress from Oedometer Test
Figure 4.5 Typical CNS Test For Dense Calcareous Sands With 250kPa Initial Normal Stress
Figure 4.6 Effects of Initial Normal Stress on the Shear Behaviour of Dense Calcareous Sands Under CNS Test Conditions
Figure 4.7 Effects of Initial Density on the Relationship Between Shear Stress and Normal Stress from CNS Tests
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(Dense sand, Horizontal displacement = +/- 1mm)

Figure 4.12 Effect of displacement amplitude on vertical displacements
(Dense sand, Vertical stress = 150 kPa)
Figure 4.13 Typical CNS shearbox response ($K = 1600$ kPa/mm)

Figure 4.14 Typical CNS shearbox response ($K = 1600$ kPa/mm)
Figure 4.15 Typical CNS shearbox response (K = 1600 kPa/mm)

Figure 4.16 Effect of displacement amplitude on variation of normal stress during CNS tests (K = 1600 kPa/mm)
Figure 4.17 Comparison of measured and predicted normal stress variation in a CNS test ($K = 1600$ kPa/mm)

Figure 4.18 Comparison of measured and predicted variation of shear stress in a CNS test ($K = 1600$ kPa/mm)
Figure 4.19  Comparison of Shear Stress Reduction in CNS and Model Pile Tests. (+/- 1 mm, K = 1600 kPa/mm)
**Figure 4.20** Effects of Cyclic Displacement on the Degradation Factor of Shear Stress for CNS Tests (50 Cycles)
Chapter 5 EXPERIMENTAL APPARATUS AND TEST PROCEDURES

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Chapter 5

EXPERIMENTAL APPARATUS AND TEST PROCEDURES

5.1 INTRODUCTION

Two series of carefully controlled tests on model single piles and pile groups were performed in reconstituted beds of N.N.R calcareous sand. The tests were conducted to investigate the behaviour of offshore single piles and pile groups and to study the effects of the laboratory boundaries on the results of these tests. The first series of tests was carried out using a single instrumented pile jacked in both "medium dense" and "dense" sands which were consolidated into both large and small vessels. The second series of tests, on model pile groups, was carried out in "medium dense" calcareous sediment using only the large test vessel. The preparation of reconstituted beds of sand is described in Section 5.2. In each test, jacking, static loading and cyclic or storm loading responses were measured.

The tests were conducted on single piles and pile groups in both vessels with different conditions at the base boundary. Details concerning these boundaries are described in Section 5.4. Tests were carried out to investigate the transfer of the
overburden pressure to the bottom of the soil, with attention given to the influence of shear resistance between the soil and internal wall of the test vessel.

This chapter describes each piece of apparatus and the procedure used in the experimental work carried out in this thesis, including details of a data acquisition system which monitored the testing. The test results are described and discussed in Chapters 6 and 7.

5.2 PREPARATION AND MEASUREMENT OF UNIFORM SAND BEDS.

Two types of soil sample were employed for the single pile tests; dry and wet calcareous sand. For the pile group tests only dry calcareous soil was used.

Before the test programme commenced, the sand sample was oven dried and then sieved so that only particles smaller than 6mm were used in this series of tests. Grading curves of the soils at different stages of the experimental programme were prepared and are shown in Figure 3.1 in Chapter 3. It has been found that the amount of finer particles of calcareous sand at the end of all model pile tests increased slightly. This is indicative of the breakdown of the sand particles due to the repeated use of the same sand sample in these tests.

Density is one of the most important measurable parameters of a granular material, because it affects both the bearing capacity and settlement response of foundations. This section describes an attempt to produce sand beds of controlled and reproducible density in laboratory tests. Two soil types were employed here, calcareous and silica sand.
In order to ensure accurate measurement of dry and bulk densities of the sand bed, it was necessary to carry out in-situ sampling. This was achieved by using small tins with different end conditions to assess the most effective means of accurately determining the density, as well as to examine the various problems associated with sampling.

In the past decade, geotechnical engineering studies have shown that the strength and deformation properties of cohesionless soils subjected to static and cyclic loading depend not only on the density of the soil but also on the fabric characteristics which are a function of deposition. These findings have been reported by many researchers (Kolbuszewski and Jones, 1961; Oda, 1972; Ladd, 1974; Kulhawy et al, 1979; Holtz and Kovacs, 1981; and Miura and Toki, 1982).

In the current investigation, two simple devices were designed and used to prepare beds of the soil by uniformly depositing the soil into three test containers of different sizes. The first device was a single sieve rainer used for raining silica sand into a 300 mm diameter steel vessel. The second device was a diffusing rainer with a diameter of 580 mm, utilized to rain calcareous and silica soils into a steel vessel. These carefully prepared specimens of homogeneous soil were subsequently used for testing piles under static and cyclic loading.

5.2.1 Procedure and Equipment for Sand Raining

Figures 5.1 and 5.2 show the two devices employed in this research (single sieve rainer and diffusing rainer). Both were designed to rain sand into a container to produce uniform beds with specified density. Each rainer could be moved vertically on rods screwed onto the top flange of the vessels.
a) Single Sieve Rainer

This apparatus was used to rain sand into the 300mm diameter test vessel. It consisted of a 6mm mesh sieve extended by a 450mm deep cylinder of plastic sheet to prevent the dust and small particles escaping out of the vessel. The sieve was able to slide on two rods so as to drop the sand from different heights. A plastic sheet was on the grid of the sieve before the sand was placed. The single sieve was suitable to rain only silica sand (see Figure 5.1). The rainer could not rain calcareous sediment because the coarse particles of this soil were not as regular in shape and some were larger than the mesh size in one dimension. For these reasons, the holes of the mesh could be easily blocked.

The procedure of raining silica sand by this device consisted of gently pulling up the piece of plastic sheet from the sieve centre to allow the sand to drop into the container.

b) Diffusing Rainer

The sand rainer device employed in this research is shown in Figures 5.2 and 5.3 and is referred to as a "Diffusing Rainer". A sketch of the separate components is given in Figure 5.2.

This device was designed for raining both calcareous and silica sands into a 590 mm diameter pile-testing vessel. The rainer consisted of a thin-wall drum of 575mm diameter and 230mm height, again extended downwards by a 450mm deep plastic sheet cylinder to prevent the dust and fine particles from escaping. The perforated base of the device contained different apertures, the sizes of which increased radially. However, the size of the apertures around each annulus remained the same.
This base was composed of two parts, the top part rotating around the centre, while the bottom one remained stationary. This mechanism allowed sand to pour when the holes of both parts of the rainer bases corresponded. Two sieves of 6mm steel mesh were attached underneath the perforated base to diffuse the sand and distribute it into the vessel uniformly. The intensity of deposition could be governed by changing the size of the sieve mesh or by changing the spacings between the sieves and the base of the sand rainer.

The sand rainer moved vertically on four rods screwed to the top face of the container. It was connected to a hand winch by means of a pulley system. The hand winch was used to control the height of sand drop, (see Figure 5.3). The soil was dropped into the vessel from approximately 450mm to produce the bottom layer, and then the rainer was raised by the winch so as to drop the soil from the same height for each layer (of about 120mm thickness).

5.2.2 Measurement of Sand Density

Two types of method for determining the density of sand exist: the indirect method representing laboratory tests, and the direct method involving field tests in which a hole is excavated in the earth at a desired elevation and the volume and mass of excavated soil is measured by one of three common techniques (Holtz and Kovacs, 1981).

The measurement of density in the laboratory has been carried out by taking a soil sample from the soil mass using a tin. Kolbuszewski (1948) and Tschebotariof (1944) used the same method to measure the porosity of dry and wet granular materials.
Four types of tin were used here to measure the sand density, as shown in Figure 5.4:

1) Hollow tins: These were made of aluminium and had two open ends. The volume of the two tins used was 87 cm\(^3\) and 87.5 cm\(^3\) respectively.

2) Permeable base tin: This was made of tin plate and had one end open and the other closed by fine wire mesh. Its volume was 96 cm\(^3\).

3) Flexible base tins: These were made of aluminium and had one end open and the other closed by a flexible rubber membrane. The volume of each was 87.5 cm\(^3\).

4) Rigid base tins: These were made of tin plate and had one closed end. The volume of each was 96 cm\(^3\).

The walls of the tins prevented radial movement of particles, while the vertical movement of particles depended on the type of tin used.

The levelling of the top surface of the sand in any type of tin was performed by using a thin flat scraper. This procedure was designed to reduce the error in the density of a sample due to disturbance of the arrangement of the particles.

The arrangement of the particles controls the void ratio and the movements of the particles inside the rigid-based tin are different from those inside the flexible and hollow tins, which again may be different to those outside the tin. This is because of the different interaction movements between the particles.

The existence of the base in the rigid base tin completely prevents the movement of some particles in the vertical direction, while the flexible base tin will allow vertical movement to occur, so that the arrangement of particles inside the flexible tin is different from that inside the rigid one. The value of density obtained with a rigid
base tin is more likely to be influenced by the tin base altering the particle distribution than in a flexible base tin.

The sample taken with the hollow tin may theoretically provide a more realistic value of density than that provided by the other types of tins, but the taking of a sample by this tin is not as easy to control as for the other tins. It was found that some particles can escape from, or enter into, the tin throughout the procedure of closing the bottom end of the tin.

It is obvious that the dry density of the sand bed depends on the initial arrangement after dropping or subsequent rearrangement of particles, which is controlled by the movement of particles during the raining procedure. The size and the shape of particles also affects the density. Considering the arrangement of particles and the condition of taking samples, the density provided by the flexible base tin should be more accurate than that obtained by the rigid base tin and less accurate than by the hollow tin.

The reasons for the use of a permeable base tin for measuring wet sand density are as follows:

1) To allow the water to move inside the tin in the vertical direction. The water will be able to move up or down according to the stages of the experiment. The first stage is when the sand is dry and the water flows up until it reaches the surface of the sand in the vessel. During the second stage the water is allowed to drain down by gravity.

2) The water content of the sand inside of the permeable tin is essentially the same as that outside the tin; the other tins do not lead to this state.

3) The movement of the particles inside the tin resulting from the water
movement is expected to be almost the same as on the outside of the tin.

4) Good control can be achieved on taking the sample from the vessel. From the above, it would seem that a permeable based tin will provide better results for density than a rigid base tin in bulk (wet) density measurements.

5.2.3 Procedure for Measurement of Dry Density

i) Large Container

For both calcareous and silica sands, the 590mm diameter large steel container was used. The sand was dropped from the diffusing rainer to obtain a homogeneous soil layer and to simulate the deposition process in the offshore environment. The soil bed was then used for model pile tests. The calcareous sand bed was prepared in four layers, each 120mm thick. Rigid base tins were placed on the base of the sand container so that they were located at the bottom of the first layer.

After placing each layer of sand, five tins (three with rigid bases and two with flexible bases) were placed, and the distribution of the tins along the diameter of the container was as follows:

1) One rigid base tin was placed on the centre line.

2) Two rigid base tins were placed along one side of a diameter at radial distances of ±0.44r and ±0.88r from the centre line, where "r" is the radius of the container.

3) Two flexible base tins were placed on the other side of the same diameter at the same radial distances as in (2) above. Figure 5.2 shows
the locations of these tins.

After the raining of the soil had been completed and the rainer device was removed, the soil was carefully excavated. Any movement of the tin disturbed the sand in the tin, so that the excavation had to be started from a point far from the tin. The excavation continued until all the sand around the tin was removed. A thin aluminium plate was then used to scrape off any excess from the sand at the top of the tin.

ii) Small Container

The soil used in the small container was silica sand and the profile of the sand was divided into three layers, each being about 120 mm thick (see Figure 5.1). In each layer the following procedure was adopted:

1) A rigid base tin was placed so as to be at the bottom of the first layer.

2) During the raining of the sand, one hollow tin and one flexible base tin were placed side by side on the top of both the bottom and middle layers.

3) The excavation procedure and the removal of the tins was the same as described before for the large container.

4) A flat piece of aluminium was used to adjust the surface level of the sand in the rigid and flexible base tins, as before. For the hollow tin, a second piece of aluminium was inserted underneath the tin to close the bottom end. The hollow tin, the enclosed sand, and the two pieces of aluminium were then weighed.
5.2.4 Procedure for Measurement of Wet Density

In this section, the measurement of wet density for calcareous sand are described. However, those for silica sand are described else where by Al-Douri et al (1990).

Wet calcareous sand was produced when the sand was rained into the large vessel containing water. The water used in this method was eventually drained out from side drains near to the top and bottom of the container.

There were four round filter papers, two on the base of the vessel and the other two on the surface of the sand. Fine wire mesh was placed beneath the sand under the bottom filter papers. Four strips of filter paper were used to allow transfer of the water between the round filters and the side drains, and to assist the water to flow uniformly over the cross sectional area of the sand.

Three tests were conducted to measure the bulk density of calcareous sand in a large vessel. Each test involved one hollow tin and one rigid base tin placed in the middle and bottom layers of the sand profile. To produce wet samples, the sand was rained through about a 100mm depth of water and the tins were buried before the raining procedure began. Subsequently the sand was subjected to 200 kPa overburden pressure for about 40 hours.

5.2.5 Results for Dry Density

a) Large Container

The results of tests using the two types of sand are as follows:
i) Calcareous Sand

The results of 21 tests are shown in Table 5.1. The drop heights for the sand that were used were 10mm, 100mm, 200m, 300mm, 450mm, 600mm, and 850mm. Three tests were carried out for each height and the densities obtained were within the range \( \pm 0.01 \ t/m^3 \) of the average value. This represents acceptable accuracy for measurement of the density of calcareous sand.

For each height, the density of the soil at distances of \( \pm 0.44r \) and \( \pm 0.88r \) from the centre line (C.L) was taken as the average value of the density obtained by the rigid base tin and the flexible base tin. The density at the centre line was obtained only from a rigid base tin.

Figure 5.5 shows the relationship between the dry density and the radial distance across the vessel. For the first four heights (10, 100, 200 and 300 mm), the values of sand density at three locations (C.L, \( \pm 0.44r \) and \( \pm 0.88r \)) are not significantly different. The density was found to be slightly different for the other heights.

Figure 5.6 shows the height of drop versus the dry density. The average density at the C.L varied from 0.97 to 1.05 \( t/m^3 \) when changing the drop height from 10 to 850 mm. The density increased uniformly between 10mm and 300mm drop height, whereas, beyond 400mm height, the density remained more or less constant. Figure 5.6 also shows that a small reduction in the values of density is connected with an increasing distance from the centre line of the vessel for all the heights of drop.

The average values of density obtained from the flexible-base tins are less than the values obtained from the rigid base tins for 10mm, 100mm, 200mm, and 450mm heights, but the density values from flexible base tins are almost the same as those
values of rigid base tins for all other heights.

ii) Silica Sand

Table 5.2 shows the results of 15 tests from five heights (100mm, 300mm, 450mm, 600mm and 850mm). Three tests were carried out for each height and the resulting densities are within ±0.08 t/m$^3$ of the average value.

Figure 5.7 plots the radial distance of measurement of the sand density versus the dry density. The density obtained at the C.L is higher than that obtained at other locations for all heights. The density at the ±0.44r location is almost equal to that at the ±0.88r location for a height range between 100mm to 450mm. Beyond these heights the location closer to the centre gives a higher density. This is attributed to the reduction in the flow rate of sand due to the hole size reducing radially towards the centre.

Figure 5.8 shows three curves for the densities at C.L, at ±0.44r and at ±0.88r. The densities at locations C.L and 0.88r increase uniformly when the sand drop height lies between 100mm and 600mm. Beyond these heights, the densities decrease slightly, while the density at ±0.44r increases uniformly for all heights.

b) Small Containers

The results for the density of the rained dry silica sand in the small container are shown in Table 5.3. Figure 5.9 shows the results for the density measured by the different tins for each layer of the sand profile. Table 5.3 shows that the range of values as well as the standard deviation of the density obtained by hollow tins are larger than those obtained by the flexible base tins.
5.2.6 Results for Wet Density

Table 5.4 presents the results of bulk (wet) and dry densities from the middle and bottom layers of calcareous sand in the large container. The moisture content obtained directly from the sand in the vessel by pushing a sample tube into the sand ($\omega_v$) and those obtained by tins ($\omega_T$) are shown in this table. Table 5.4 also shows the overburden pressure and the time of consolidation adopted in the tests.

Generally, the bulk density ranged between 1.59 to 1.63 t/m$^3$ for the middle layer and between 1.59 to 1.64 t/m$^3$ for the bottom layer, while the measured dry density ranged between 1.09 to 1.12 t/m$^3$ for the middle layer and between 1.07 to 1.14 t/m$^3$ for bottom layer. These values were higher than for the dry soil because of the application of an overburden pressure of 200 kPa.

The wet densities obtained by hollow tins were consistent and their values were slightly higher than those obtained by rigid base tins.

Figures 5.10 (a and b) show the relationships between the bulk densities obtained by rigid base tins and hollow tins and the height above the base of vessel.

For each test the value of moisture content ($\omega_v$) for the middle layer is less than that for the bottom layer. All values of the moisture content obtained by hollow tins ($\omega_T$) for the middle layer were less than those for the bottom layer, whereas the values of ($\omega_T$) obtained from rigid tins for the middle layer were not always less than those for the bottom layer. However, values of moisture content obtained by hollow tins show a good agreement with the sampled values ($\omega_v$).

The relationship between the moisture content and the dry density of the sand is shown in Figure 5.11. Dry density increases as moisture content decreases.
5.2.7 Discussion of Sand Density Results

The density can be changed from test to test by choosing different fall heights. Thus, the effect of sand density on the static and cyclic behaviour of loaded piles can be investigated.

The highest value of dry density for the New North Rankin calcareous sand obtained by the technique of raining the sand in the empty vessel, as presented in Table 5.1, was 1.05 t/m$^3$. This was the lowest value of the three methods. The maximum dry density obtained by raining the sand in the vessel containing water was 1.14 t/m$^3$ as in Table 5.4.

The maximum dry density ($\gamma_{d\text{max}}$) of silica sand was obtained by raining in the large container. This value (1.67 t/m$^3$) is higher than the maximum dry density (1.63 t/m$^3$) determined by Chua (1983) and approximately equal to the maximum (1.65 t/m$^3$) reported by Swane (1983).

Figure 5.12 summarizes the influence of drop height on dry density. Despite the fact that the calcareous sediment density increased from 0.96 t/m$^3$ to 1.05 t/m$^3$ as the height of fall increased from 100mm to 450mm, beyond this height the density was not height-dependent. However, for the silica sand, the density increased from 1.42 t/m$^3$ to 1.67 t/m$^3$ as the height of fall increased from 100mm to 600mm. The raining produced a maximum density of silica sand substantially higher than that of the calcareous sand. This may be because the size of mesh attached beneath the base of the rainer was only small enough to break up the flow of calcareous soil, while allowing higher velocities of flow for silica sand.

In general the density increases with increasing drop height up to a certain height.
Beyond that height, the density does not increase and it will not reach the maximum value that can be obtained by tamping or vibrating processes.

The accuracy of the measurements of density obtained by using the four types of tins is influenced by the following factors:

1) The density results may be influenced by the diameter of the tins used. Kolbuszewski (1948) used the same techniques to measure the density of dry and wet granular materials, and found that the density increases as the diameter of the tin increases. This phenomenon was explained by the influence of the wall friction in the tin.

2) Inaccuracies in measuring the volume of the tin.
Because of the small size of tin used, any small error in the measurement of the dimensions will adversely affect the calculation of the volume. Also, the calculation of volume for a small tin is less accurate than for a large tin. It was found that the best way to find the volume of the small tins was by measuring the quantity of the water occupying the tins, allowing for surface tension effects. The accuracy of measuring the volume is influenced by the surface shape of the water which has a convex shape due to the phenomenon of surface tension. The cross-sectional area of the tin also influences the surface shape of the water.

3) Disturbance of the sand surface.
A flat scraper was used to smooth the sand surface. It was noted that the friction between the tin end and the scraper increased as the roughness of the end of the tin increases. Therefore, for the hollow tin and flexible base tins, the friction is more significant than when the end is sharp. The effect of this friction is to disturb the sand surface, which then affects the experimental values of sand density obtained.
4) Smoothness of the tin walls.

The internal surface of the walls of the Hollow and Flexible base tins are smoother than the Rigid and Permeable base tins, because the first two tins are made of aluminium and the others are of tin plate which is subject to oxidisation. The roughness of the surface affects the arrangement of the particles near the wall of the tin.

5) The drop height of the sand.

It was also found in the experiments that the sand dropped from one side of the sieve directly into one tin and then after a short period of time into the other tin. The value of density for the sand in the first tin was found to be greater than that for the second one. This may be due to the displaced air flowing upwards over the second tin, delaying the filling of the second tin.

5.3 PREPARATION OF DENSE SAND

The dense samples for the large vessel were prepared by the same method as for the medium dense sample, except that after raining, the soil was vibrated by two machines which were placed opposite each other on the top of the vessel as shown in Figure 5.13. This approach was repeated to produce a dense sample in the small vessel, but only one vibrator machine was used. The vibrator was centrally attached on a temporary steel bridge across the diameter and fixed on top of the pot by two Allen screws. The range of density for dense samples was from about 1.07 t/m$^3$ (10.5 kN/m$^3$) to 1.3 t/m$^3$ (13.0 kN/m$^3$).

The difference between the preparation of a dry and wet sample was that the former was prepared when the sand was rained into an empty vessel, while the
latter one was prepared when the sand was rained into a vessel which was partially filled with water. For the wet samples, a filter paper was placed over a fine wire mesh on the base plate for the bottom drainage of water, and similar arrangements were repeated on the top. The arrangement for dry samples was that four pieces of fine wire mesh were placed on the bottom and side drainage holes.

Two ways have been adopted to measure the sand density used for model pile tests. The first way measured the density for different layers of the soil using small tins which were placed in these layers of soil. The second way was to measure the density of the total soil mass by dividing the weight of soil in the test container by the volume of the test container. The minimum and maximum dry densities of the sand beds prepared for the model testing have been found to be about 0.96 t/m$^3$ (9.4 kN/m$^3$) and 1.3 t/m$^3$ (13 kN/m$^3$) respectively for calcareous soil. It was also found that the minimum and maximum densities of silica sand were about 1.42 t/m$^3$ (14 kN/m$^3$) and 1.65 t/m$^3$ (16.2 kN/m$^3$) respectively using the same raining technique.

5.4 TEST VESSELS

Two different sizes of test vessel were employed in this study, in order to gain a better understanding of the influence on the behaviour of the pile of the boundary condition of the test vessel (e.g. the friction between the soil and the vessel wall). The two vessels and associated apparatus are pictured in Figure 5.14 and the large vessel is shown diagrammatically in Figure 5.15. Both vessels were similar, and comprised of a top, middle and bottom part which are described later. It was possible to employ three alternative base conditions which are shown in Figure 5.16 and are as follows:
1. Rigid base: the base of the test vessel was a thick steel plate. This is commonly used in model pile tests. The soil was consolidated by an applied pressure only at the top of the soil.

2. Semi-Rigid base: in this case the soil base was supported by a rubber membrane lying over water which has been deaired and confined in the bottom chamber. The soil was consolidated by an applied pressure at the top, as in the first case.

3. Pressure Controlled base: This base is almost the same as in the second case except that the pressure is also applied at the bottom. In this case the soil is subjected to equal pressure at both top and bottom boundaries.

The details of these test vessels are given below.

5.4.1 Large Test Vessel

The major series of single pile tests, (discussed in Chapter 6), and all pile group tests, (discussed in Chapter 7), have been performed using this test vessel. The central body of the vessel was a thick steel cylinder with an internal diameter of 590mm and depth of 480mm as shown in Figure 5.15. These dimensions were deemed sufficient for the installation of a 4-pile group in calcareous sand, containing piles less than 300mm long, and ensured that the vessel boundary was at a reasonable distance from the outside of the pile group.

The top part was designed as a chamber to apply pressure on the top of the soil which was to be consolidated in the body of the vessel. This top part consisted of a
1.5mm rubber membrane, aluminium spacing ring and a strengthened flat lid. Five holes were made on the lid to fit the sealing collars. One collar was at the centre for single pile tests and four collars surrounded the centre one for pile group tests. The top assembly was secured to the top flange of the vessel's body with twenty Allen screws. Rubber O-rings placed at the top of the spacing ring and in the flange of the main body below the spacing ring provided sealing.

The design of the base of the vessel was dependent on the lower boundary condition of the test. For a rigid base boundary, the base was a 20 mm thick steel plate. However in the case of both a semi-rigid base "S.R.B" and pressure controlled base "P.C.B" boundary, the bottom part of the vessel was modified to be a chamber containing water at the bottom pressure as shown in Figure 5.17. This part of the vessel consisted of two rubber O-rings placed between the bottom flange of the middle part and the rubber membrane and between the spacing ring and steel plate. The bottom assembly was secured to the base of the pot with Allen screws.

The body of the vessel was lined with stainless steel to reduce the friction between the internal wall of the vessel and the sand. Four drains were employed, two of them on either side near to the top and the other two on either side near to the bottom. The drains were protected with double layers of gauze to permit air to flow out of the dry sand samples or drain water from saturated sand samples under pressure and allow two-way drainage to be effected during consolidation and testing.

A hole in the test vessel lid connected the chamber to the air pressure which was supplied using a Norgren pressure regulator coupled to a compressed air supply. The supplied pressure was measured by a Budenberg standard test gauge. The confining air pressure in the top chamber was separated from the soil by a rubber membrane. Another hole at the centre of the steel plate at the base was connected to the
air/water exchange steel pressure bottle by brass fittings as shown in Figure 5.15. The means of pressure supply (when required) was similar to those used for the top chamber. A pore water pressure transducer was connected using a brass tube and fittings to measure the pressure generated at the bottom of the soil due to applied overburden on the top of the soil. A rubber membrane separated the soil from the water confined in the bottom chamber.

5.4.2 Small Test Vessel

The small vessel which was used for two different types of tests is pictured in Figure 5.14. The first type of test was for single piles. The second type was for the effects of the shear resistance acting along the internal wall of the vessel upon the transfer of the overburden pressure to the base. This is described in Section 5.11.

The body of the small vessel, the top and the base chambers were of the same basic design as those of the large vessel. The vessel had an internal diameter of 300 mm, about half of that of the large vessel. The purpose in using this dimension was to investigate the effect of the vessel diameter on the behaviour of the pile tests under different base boundary conditions. The body of the vessel consisted of one or more thick steel cylinders to provide different depths, depending on the test requirements. The combined depth of two of the cylinders was 470mm for the single pile tests. In this type of test, the vessel was always lined with stainless steel sheet, the same as the large vessel, to reduce the friction between the soil and the walls of the vessel. For the second type of test, the depth was varied from 120mm for one cylinder to 590mm for the combined depth of three cylinders. The stainless steel lining the internal wall was removed in some tests and was replaced by teflon in other tests.

The lid and the spacing ring connected to the top chamber was one piece and one
pile collar was attached to the centre by four screws.

5.5 COLLAR DESIGN

To achieve the jacking of piles without releasing the confining pressure in the top chamber, a special sealing collar was designed. The sealing collar consisted of a part which is fixed on the vessel lid by four bolts, and a sliding collar which is fitted through the fixed collar and was attached to the rubber membrane, as shown diagrammatically in Figure 5.18. The sliding collar was able to move down as the soil consolidated. The friction between the pile shaft and the O-ring in the sliding collar was negligible.

In order to reduce the area underneath the sliding collar which might affect the response of the pile or the interaction between the piles in the pile group tests, the base of the sliding collar was modified to be smaller than that used by previous investigators (i.e Chan, 1986 and Allman, 1988). As shown in Figure 5.19, a thin small plate was attached by three screws to the edge of the sliding collar (attached to the membrane), through which the instrumented pile was jacked. This plate allowed the core of the deflection transducer to rest on it and measure the settlement of the collar (soil surface) during testing.

5.6 INSTRUMENTED PILE

The instrumented model pile which was used for both single pile tests and pile group tests was fabricated from 25mm external diameter aluminium tube, with a 3mm wall thickness. The pile as pictured in Figure 5.20, was instrumented at 5
sections inside the shaft of the pile and is shown diagrammatically in Figure 5.21. A 45° solid aluminium cone was fixed to the tip of the pile and a steel cap was attached to the top of the aluminium shaft of the pile. The steel cap was threaded at the top to fit into a steel connector which consisted of three pieces. This connection was used to fix the pile to the plunger of the loading machine as shown in Figure 5.22 (a and b). The connector enabled the pile to be disconnected from the plunger without turning the pile. A 10mm diameter hole was provided in the steel cap for egress of instrumentation wiring.

At each of 5 instrumented sections in the pile, four electrical resistance strain gauges were mounted inside the wall of the tube at a distance of 10mm below the joint. The four strain gauges were positioned 90° apart to form a full Wheatstone Bridge circuit. Two active gauges were mounted vertically on opposite sides while the other two gauges were dummy gauges. This arrangement eliminated the possibility of errors through bending or torsion of the pile.

The strain gauge load cells were positioned in such a way that the top cell was above the soil surface when the pile was embedded into the soil to about 300mm depth. The distance between the centres of the load cells was 80mm. The sections of the pile were glued together with high strength Araldite adhesive.

**5.7 DUMMY PILES**

The dummy aluminium piles were used only for pile group tests. The design and the dimensions of these model piles were the same as the instrumented piles, except that these piles were without strain gauges. The aluminium shaft part of the pile was one tube instead of five connected tubes. In the pile group tests, three dummy piles, and
tube instead of five connected tubes. In the pile group tests, three dummy piles, and one instrumented pile were attached to the pile cap, for the 4-pile group tests. However, for 2-pile group tests, one dummy pile and one instrumented pile were used.

5.8 PILE CAP

Three types of 22mm thick steel pile connecting plates (caps) were used for pile group tests. The dimensions of these caps were dependent on the number and configuration of the piles as shown in Figure 5.23. The largest cap was 400x400 mm and contained two different types of hole with centre-to-centre spacing 70mm between adjacent piles. The first type of hole was tapered to use when grouting the piles into the cap, while the second type was threaded to be used when the piles were screwed into the cap. The first (grouted) connection was used after the piles were jacked individually, and the threaded holes were used for the jacking of entire pile groups as shown in Figure 5.24. This type of pile cap was used for 4-pile group tests and for 2-pile group tests which had a 125 mm centre-to-centre pile spacing. The holes in the other pile caps were threaded holes.

The dimensions of the medium size and the small pile caps which are shown in Figure 5.23 were 400x200 mm and 300x200 mm respectively. The space between the two centres of the piles for the medium pile cap was 100.0 mm and for the small cap 70.0 mm. The loading plunger was screwed into the central threaded hole of each type of cap for connection to the testing machine.
5.9 DATA ACQUISITION SYSTEM

Two different types of data acquisition systems were used during the experimental work in this thesis. The first type was manufactured at the University of Sydney for use with a Tandy computer, and was employed for direct shear tests. The second type was a Hewlett Packard Data Acquisition system and was employed for single pile and pile-group tests. The pile testing system is described below.

The data acquisition system for the pile tests was different from that used for the shear box tests described in Chapter 3. Figure 5.25 shows the diagrammatic lay out of the Hewlett Packard Data Acquisition System, which consists of five components; a plotter, a digital voltmeter, a printer, a controller and a scanner. The voltage readings from both the load and deflection transducers were calibrated and stored as calibration data.

This system used a computer program in the controller to monitor test data and store it on floppy disc. The plotter enabled results to be plotted as the test was running or replotted whenever needed after completion of the test. A simple modification was made to the computer program that was written by Dr T.S. Hull in order to use it in these tests. The program could convert displacement transducer and strain gauge voltage readings to deflection and load. The tests conducted were displacement or load controlled, i.e. at the specified displacement or load, the loading machine changed direction under control of the HP9825A controller.

5.10 INVESTIGATION OF VERTICAL STRESS TRANSFER TO VESSEL BASE (V.S.T.V.B)

In this series of tests, the main objective was to investigate the effects of arching on the behaviour of the soil rained inside the pile testing vessel.
The main factors affecting the vertical stress along the soil profile mobilized by the applied overburden pressure were:

a) the value of the applied overburden pressure,
b) the dimensions of the vessel,
c) the roughness of the internal vessel wall,
d) the sand void ratio,
e) the soil type, and
f) the over consolidation ratio.

It was desired to measure the vertical stress at the base of the confining sand inside the testing vessel when different overburden pressures were applied to the soil surface. To find the effects of the vessel's size and the roughness of its walls, the tests were performed on three different sizes of vessel and each vessel was tested for both the smooth wall case, produced by lining the wall with a stainless steel sheet, and the rough wall case without a stainless steel sheet.

The New North Rankin calcareous sand and silica sand were employed in these tests. A comparison between the vertical stresses measured at the bottom provides a good understanding of the behaviour of the soil in the cylindrical vessel.

The void ratio or the density of the sand can be controlled by raining sand into the vessel from a constant height as described in Section 5.2.

5.10.1 Background of (V.S.T.V.B)

Figure 5.26 shows the forces acting upon a horizontal section of soil in a cylindrical container. Since, to a first approximation the friction $p_f$ of the wall is proportional
to the horizontal pressure \( p_h \) then:

\[
p_f = \mu \ p_h \quad \ldots \quad (5.1)
\]

where

\[ \mu = \text{coefficient of friction between the soil and the wall.} \]

The equilibrium of the vertical forces at a horizontal section can be expressed by Janssen's formula:

\[
q.A + \gamma.A.dz - p_f. c \ dz - (q + dq).A = 0 \quad \ldots \quad (5.2)
\]

where

\[ q = \text{vertical pressure in the vessel} \]
\[ c = 2\pi R \text{ circumference of the vessel,} \]
\[ \gamma = \text{bulk density of the soil,} \]
\[ A = \pi R^2 \text{ cross-sectional area of the vessel and} \]
\[ z = \text{depth of the soil above the horizontal section.} \]

The pressure coefficient \( k \) is defined as the ratio of the horizontal pressure to the vertical pressure in the soil:

\[
p_h = k.q \quad \ldots \quad (5.3)
\]

Substituting equation (5.3) into (5.2) yields:

\[
dq/dz + 2\mu kq/R = \gamma \quad \ldots \quad (5.4)
\]

Defining \( z_0 = R/2\mu k \), this differential equation can be integrated to give:
\[ q = q_0 \ e^{-z/z_0} + \gamma R/2\mu k \ (1-e^{-z/z_0}) \] ........................ (5.5)

where \( q_0 = q (z = 0) \) is the overburden pressure.

This theoretical relationship was used to predict the vertical stress at the base of vessel.

5.10.2 Procedure for (V.S.T.V.B)

At the beginning of this series of tests, the top part of the vessel was removed, and the middle and the bottom parts were positioned. Water was poured into the bottom space through an open central valve until the space was full, then the water was poured over the membrane to about 100 mm below the vessel surface. The lid was assembled on the vessel and a negative pressure used to de-air the water.

To confirm that the water was de-aired as far as possible, the de-airing was continued overnight under a gauge pressure of \(-60kPa\). After the de-airing, the lid was removed and the immersed central valve was closed tightly to separate the water above and below the rubber membrane completely.

The lid was assembled again and the overburden pressure was applied on the water surface directly. The pressure was gradually increased from 0 to 200kPa and was then decreased. During these stages, the data acquisition system was recording the reading of the pore water pressure transducer. This procedure was repeated several times to produce an accurate calibration.

The overburden pressure was released and the lid removed after the calibration. All
water over the bottom membrane was drained, and the middle and top parts of the vessel were dried. Either calcareous or silica sand was rained into the vessel to a height of 10mm above the surface of the vessel. The top part of the vessel was placed in position and the overburden pressure was applied to the top surface of the membrane. Readings of the pore water pressure transducer were taken for four pressures (50, 100, 150, 200kPa). The readings were taken 5 minutes after applying the overburden pressure to allow stabilization of the transducer reading. The readings were taken again 6 hours after applying the overburden pressure.

The tests for the "smooth" vessel walls were performed when a stainless steel sheet lined the vessel, and the tests for rough vessel walls were performed without stainless steel sheet.

5.10.3 Results and Conclusions for (V.S.T.V.B)

Tables 5.5 and 5.6 summarize the results of the tests performed on the calcareous and silica sands. The overburden pressures "q₀" used were 50, 100, 150, and 200kPa. Two tests were carried out for each "q₀" and the vertical stresses at the bottom of the soil obtained were within ±5 kPa of the average value. This is considered to be within acceptable accuracy of measurement for the stress at the bottom of the soil.

For a large vessel of length (L) 480 mm and diameter (D) 590 mm (D/L = 1.23), with a smooth wall surface, the value of vertical stress at the bottom of the soil "q" was about 70% of q₀ after 6 hours, whereas, the value of q was about 50% of q₀ after 6 hours where the vessel wall surface was rough. In the case of the medium vessel of length 620 mm and diameter 420 mm where (D/L = 0.65) the tests were performed only for a smooth wall surface. For a low overburden pressure
\(q_0 = 50\text{kPa}\), the value of \(q\) was about 44% of \(q_0\) after 6 hours.

Similarly for the small vessel of length 470 mm and diameter 290 mm (\(D/L = 0.62\)), the vertical stress at the bottom of the soil was approximately 55% of each overburden pressure \(q_0\) for a smooth wall surface and 45% of \(q_0\) for a rough wall surface.

From the above results it can be concluded that the reduction in the vertical stress at the bottom of the soil was most likely attributable to developed skin friction on the wall surface (even when this surface was smooth). The consequent arching causes a reduction in vertical stress appearing at sections down each vessel, with more reduction being associated with lower values of \(D/L\).

Table 5.5 compares measured and predicted values of vertical stress at the base of the vessel. The values predicted from the theoretical relationship (eq. 5.5) are slightly less than those measured in the tests for the large and medium vessels and slightly larger than those measured from the small vessel. It is possible that the reason for this difference is that the frictional resistance between the soil and the wall of the vessel has not been fully mobilized for the large and medium vessels. It should also be mentioned that, in the simple theory developed in Section 5.10.1, \(D/L\) has no influence on the vertical stress.

5.11 MEASUREMENT OF SAND SURFACE MOVEMENT (M.S.S.M)

It is known that the surface movement of calcareous sand during consolidation and its rate of creep are higher than that of silica sand. This has been shown through various types of one dimensional (oedometer) and three dimensional (triaxial) tests.
which have been reported by various investigators, among them Datta et al (1979), Poulos et al (1984, 1985), Hull et al (1988), Airey et al (1988). It was also observed from the shear box tests described in Chapter 3, that the reduction in the vertical settlement of calcareous sand is more severe than that of silica sand when both sands are subjected to same level of cyclic loading (also Al-Douri and Poulos, 1992). Data from cyclic triaxial tests conducted by Kaggwa et al (1988) showed that the vertical displacement of calcareous sand from the North Rankin Platform site was higher than that from Bass Strait.

An investigation was undertaken to study the surface deflection of calcareous sand in the large vessel during consolidation and also during the process of pile loading. The objectives of this study can be summarized as follows:

(1) to determine the total time required for the complete consolidation of the sand sample.

(2) to evaluate the settlement of the soil surface. This provides a basis upon which to contour the surface of the sand during filling of the vessel so as to maintain it at a horizontal level, and at the height of the vessel, after consolidation has taken place.

(3) to evaluate the surface deflection during testing and its effects on the settlement of the piles particularly on the interaction between the two piles.

(4) to investigate the effect of applied pressure at both the top and bottom ends of sands on the movement of the sand surface.

5.11.1 Procedure and Equipment for (M.S.S.M)

Two Thompson plungers passing through sealing collars, and a thin steel plate fixed on the sliding collar through which the pile was jacked, were designed to determine
the deflection of the sand surface near the pile during the consolidation and testing, as shown in Figure 5.19. The plungers were located in collar guides fitted with linear bearings, to minimise friction, and sized "O" rings to effect a seal. The location of the two plungers and the collar on the lid of the large test vessel are pictured in Figure 5.14. Each base of the collars was fixed on the lid of the test vessel by four screws and one end of the plunger (Thompson rod) was connected to a circular plate attached to the rubber membrane, as shown in Figure 5.27 (a and b). Three displacement transducers were used to measure the movement of the soil underneath the pile collar and at the soil surface at two locations on the sand surface. Each transducer was held by a magnetic clamp. Readings were recorded and printed using the Hewlett Packard data acquisition system.

5.11.2 Test Results of (M.S.S.M)

The configuration of these three deflection measurement points are shown schematically in Figure 5.27b.

In the previous phase of this investigation, a rapid increases of deflection in the downward direction during the early stage of the application of overburden pressure, particularly on loose sand was found to occur. However, the rate of surface settlement decreases and becomes essentially zero after less than 20 hours. The results of surface settlement after consolidation are summarized in Table 5.7. This table shows the distance from the vessel centre for the three points at which the settlements were measured and also indicates the various overburden pressures used in the tests.

Figure 5.28a shows that the surface settlement decreases with increasing sand density as would be expected. The settlement distribution after consolidation is one which
increases towards the centre of the vessel. However the smaller surface deflections for dense samples do not exhibit such a noticeable differential settlement.

Table 5.8 summarizes the results of the surface deflection after jacking and after cyclic loading. Figure 5.28b shows that the settlement of calcareous sand increases as the density decreases. The surface deflections were oriented downward for jacking of the pile in "loose" and in "medium-dense" sands, while they were oriented upward for jacking of the pile in "dense" sands.

A similar trend was found after cyclic loading was applied on the model piles jacked in different sand densities under various overburden pressure conditions, as shown in Figure 5.28c. However, the magnitudes of surface deflections due to jacking were higher than those due to cyclic loading.

The general trend in the settlement of the sand surface found from jacking piles in calcareous sand appear to be similar to that from loading driven piles and buried piles in silica sand (Vesic, 1967).

Both consolidation deformation and deflection of the surface during jacking and testing the piles in loose and medium-dense sand was reduced by applying a pressure at both the top and bottom of the sand as shown in Tables 5.7 and 5.8.

5.11.3 Conclusions Regarding (M.S.S.M)

The general trends of surface deflections of calcareous sand, obtained either from consolidation of the sand in the tests vessel or from jacking the pile, were similar. It was found that all surface deflections were increasing in the direction from the edge to the vessel centre for loose and medium-dense sands, while they were more
or less constant for dense sands. This behaviour is similar to that observed on the driven and buried piles in silica sand by Vesic (1967).

The findings from this series of tests are useful for pile tests, particularly for tests which measured interaction between the piles, and can be summarized as follows:

1) The time for consolidation of the sand used in the model pile tests was about 24 hours.

2) Applied overburden pressure at both the top and bottom ends of sands reduces the surface deflection during jacking and loading of piles.

3) The influence of initial sand density on the trend of surface deflection of calcareous sand is similar to that found for silica sand.

5.12 PILE INSTALLATION

Two methods of jacking of the piles were used in this experimental work:

i) jacking piles singly, which was not only adopted for single pile tests, but also for some of the pile group and pile interaction tests.

ii) jacking of the complete pile-group (i.e., 4-piles or 2-piles connected by a cap), which was used only for some of the pile group tests. The details of the installation procedure for single pile and pile-group tests are as described below.

5.12.1 Installation for Single Pile Tests

An instrumented model pile was used for the single pile tests, which were conducted with the pile at the centre of the large or small vessel. Before jacking the pile, the strain gauges were calibrated by loading the instrumented pile in a testing machine.
as shown in Figure 5.29a.

The pile was connected to the loading machine via a proving ring and suspended freely in the air. Then the testing program was used to take initial load readings for the proving ring and the strain gauges at zero load. These readings were taken as an absolute zero and all readings during the test were calculated relative to these zeros.

A plug, which was placed in the pile collar during consolidation of the sand, was removed and the internal wall of the collar cleaned with a cloth. The test vessel was accurately centred beneath the pile and clamped in position, and then the pile was lowered into the sealing collar until the pile tip was just resting on the soil surface as shown in Figure 5.14. Another set of readings was taken before jacking the pile into the sand, to check on the fluctuations of strain gauge readings. The jacking force was applied by the loading machine and the readings of proving ring and strain gauges were taken regularly for each 25 N of load or at each 5 minutes.

The applied load was released when the penetration depth was between 280 and 290 mm, after which the top strain gauge was still above the sand surface. Two sets of readings along the pile shaft were taken; the first set of readings was taken immediately, and another set about three hours after jacking. The last readings represented the residual load in the pile. The reason for leaving the pile embedded in the soil for a few hours was to allow stabilization and to reduce any creep which might be generated by installation. Little change was generally found in the residual load during the time of monitoring. Similar findings were reported by Allman (1988).

5.12.2 Installation for Pile Group Tests

Two methods were used to install the pile groups in the sand consolidated in the
large vessel. These methods were:

a) jacking the piles individually

The procedure for jacking each pile individually, which was used in both pile-group and pile interaction tests, was the same as that described above for the single pile tests. Each pile was centred under the collar. The loading machine could be positioned over any pile in the vessel by a combination of movement of the loading machine along the frame and movement of the vessel on a trolley across the frame.

The instrumented pile was jacked first and the residual pile load was recorded by the same method used for the single pile tests. Two deflection transducers were held by two magnetic clamps, the centre core of one transducer resting on the top of the first pile, and the other resting on a thin plate which was attached to the collar.

These transducers were connected to the data acquisition system to measure the settlement of both the instrumented pile and the sliding collar during jacking and testing. The proving ring was removed from the first pile and connected to the dummy pile which was centred over the second collar. The testing program was run again and a set of readings for the vertical displacement transducers, the loads along the shaft of the instrumented pile and the proving ring, was taken. The readings at zero load were taken before jacking the second pile in the sand, and all the readings during the test were measured relative to these zeros (i.e. the residual loads were not measured at this stage).

For four pile interaction or 4-pile group tests, the third and fourth dummy piles were jacked into the consolidated soil by the same routine.
In the case of pile group tests, the jacked piles were grouted into the pile cap by mixing four parts of araldite with two parts of hardener and pouring the mixture into the groove between the top piece of the piles and pile cap, as shown in Figure 5.29b.

The loading plunger was lowered down manually until contact was made with each pile group. The three-piece connection between the proving ring and pile cap was then screwed together as shown in Figure 5.30, and the araldite left to harden.

b) jacking two or four piles together.

The group of two or four piles was first assembled by screwing the individual piles into the pile cap. The central three-piece connection was then screwed into both the proving ring, which was attached to the loading machine, and the pile cap. One instrumented pile and one dummy pile, or one instrumented pile with three dummy piles, were screwed to the pile cap to make a 2-pile group and 4-pile group respectively, as shown in Figure 5.31. The whole pile group was then jacked into the consolidated sand via the collars.

The 10kN proving ring attached to the pile cap, the strain gauges in the model pile and the vertical deflection transducer, were connected to the data acquisition system, so that the deflection, the load in the proving ring and the loads in the strain gauges along the pile could be recorded during pile-group jacking.

5.13 TEST PROCEDURE FOR PILES

The small test vessel was always securely fixed to the loading frame, directly below
the loading machine, while the large test vessel was supported by a trolley, which allowed movement across the loading frame and then fixing below the loading machine.

The single piles and pile groups were jacked into the sand using a Wykeham Farrance loading machine of about 10kN capacity and with a maximum travel of 95mm. The device was supplied with a 25 speed gearbox, giving a range of rate of strain from 0.0006mm per minute to 1.524 mm per minute. Both manual and remote control was available in this versatile loading machine. The rate of loading used was of the order of 0.381 mm per minute without allowing for compression of the proving ring. A 5kN capacity proving ring was used for the single pile tests and a 10kN capacity proving ring was employed for the pile group tests. These proving rings were calibrated in both tension and compression, so that the load applied to the pile head or pile cap could be measured.

The deflection transducer was clamped into a fitting attached to the lower end of the proving ring, while the central core rested on a steel block fixed on one end of a vertical steel rod attached to a magnetic clamp. A sufficient distance between the core support and the surface of pile cap was left to allow the static and cyclic movement of the pile cap during testing.

5.14 SUMMARY

The soils tested were subdivided into dense, medium dense, and loose samples, depending on the range of dry density of each type. For calcareous sediments, the dense samples were identified as having a dry density greater than 10.5kN/m$^3$ while loose samples had dry densities less than 9.8kN/m$^3$ and the medium dense samples
had values in between.

The following conclusions can be drawn:

i) It is possible to produce silica sand beds and calcareous sand beds of
controlled and reproducible density under laboratory test conditions by
using a Diffusing Rainer device. The density can be varied within a
certain range by changing the height of precipitation of the sand.

ii) It is possible to determine reasonably accurately the density of each layer
of sand in the testing vessel and to define the problems associated with
sampling from different layers in each type of sampling tin.

These problems may be summarized as follows:

* Rigid-base tins may only strictly be used to measure the density of dry
sand at the base (bottom layer), because both vessel and tin base are
rigid.

* Flexible-base tins should be used to measure the density of dry sand at
the base layer, when the vessel has a flexible base. It also may be used
for other dry sand layers in the testing vessel.

* Permeable-base tins may be used for any layer of wet sand in the
vessel.

* Hollow-base tins can be used for any layer of dry or wet calcareous
sand, especially when this sand is subjected to a change in overburden
pressure which will cause it to be more dense and cohesive.

iii) The accuracy of the measurements of density using different types of tins
is influenced by the diameter and size of tin used and smoothness of the
walls.
iv) Side friction along the test vessel walls reduces the vertical stress in the soil. The major factors affecting the vertical stress at the base of the sand inside the testing vessel are the vessel geometry (L/D ratio), the roughness of the vessel walls, and the base boundary condition.

v) Using two different sizes of test vessel and different base conditions for model pile tests provides a good understanding of the effects of side and base boundaries on the stress condition in the test sand bed.

vi) The modification of the test vessel by adding a pressure chamber at the base of the conventional vessel markedly reduces the loss of stress near the base.

vii) The vertical stress reduction measured at the bottom of the soil was most likely attributable to developed skin friction on the wall surface. A comparison of measured and predicted values of vertical stress at the base of the vessel, showed that the values predicted from the theoretical relationship (eq. 5.5) are slightly less than those measured in the tests for the large and medium vessels and slightly larger than those measured for the small vessel. It is possible that the reason for this difference is that the frictional resistance between the soil and the wall of the vessel has not been fully mobilized for the large and medium vessels. It should also be mentioned that, in the simple theory developed in Section 5.10.1, D/L has no influence on the vertical stress.

viii) The general trends for surface deflections of calcareous sand obtained either from consolidation of sand in the test vessel or from jacking and loading a pile were similar. It was found that all surface deflections increased towards the centre of the vessel (in the direction from the edge
to the vessel centre) for loose and medium-dense sands, while they were more or less constant for dense sands. This behaviour is similar to that observed for driven and buried piles in silica sand by Vesic (1967).

iv) The findings from this series of tests are useful for pile tests, particularly for tests which measured interaction between the piles, and can be summarized as follows:

a) The consolidation time required for the model pile tests was about 24 hours.

b) The precise measurement of the free pile settlement due to jacking or loading an adjacent pile needs to take into account the settlement of the collar for the free pile.

c) An applied overburden pressure at both the top and bottom ends of sands reduces the surface deflection during jacking and loading of piles.

d) the influence of initial sand density on the trend of surface deflection of calcareous sand is similar to that found for silica sand.
<table>
<thead>
<tr>
<th>Dropped Height (mm)</th>
<th>Test No</th>
<th>Density of Calcareous Sand (t/m³) in the Rigid and Flexible Base Tins at Distances 0, 0.44r and 0.88r mm from the Centre Line</th>
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</thead>
<tbody>
<tr>
<td></td>
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<td>Rigid Base Tin</td>
</tr>
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<td></td>
<td></td>
<td>.88r (mm)</td>
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<tr>
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<td>0.941</td>
</tr>
<tr>
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<td>0.942</td>
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<td>Average</td>
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</tr>
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<td>0.958</td>
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</tr>
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<td>1.010</td>
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<td></td>
<td>Average</td>
<td>0.990</td>
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Table 5.2  Density of Silica Sand Using Diffusing Rainer

<table>
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<tr>
<th>Dropped Height (mm)</th>
<th>Test No</th>
<th>Density of Silica Sand (t/m³) in the Rigid &amp; Flexible Base Tins at Distances 0, 0.44r and 0.88r from the Centre Line.</th>
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</thead>
<tbody>
<tr>
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<td><strong>Rigid Base Tin</strong></td>
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<tr>
<td></td>
<td></td>
<td><code>.88r (mm)</code></td>
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<tr>
<td>100</td>
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</tr>
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</tr>
<tr>
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<td>1.532</td>
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<tr>
<td><strong>Average</strong></td>
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<td>1.507</td>
</tr>
<tr>
<td>450</td>
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<td>1.529</td>
</tr>
<tr>
<td></td>
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<td>1.587</td>
</tr>
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<td></td>
<td>3</td>
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<td><strong>Average</strong></td>
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<tr>
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<tr>
<td><strong>Average</strong></td>
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<td>1.588</td>
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</table>

r = Radius of Test Vessel
Table 5.3  Dry Density of Silica Sand in Small Container

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Density of Top Layer (t/m³)</th>
<th>Density of Middle Layer (t/m³)</th>
<th>Density of Bottom Layer (t/m³)</th>
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<tbody>
<tr>
<td></td>
<td>Hollow Tin</td>
<td>Flexible Tin</td>
<td>Hollow Tin</td>
</tr>
<tr>
<td>1</td>
<td>1.487</td>
<td>1.463</td>
<td>1.507</td>
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<td>1.520</td>
<td>1.462</td>
<td>1.516</td>
</tr>
<tr>
<td>3</td>
<td>1.519</td>
<td>1.473</td>
<td>1.502</td>
</tr>
<tr>
<td>4</td>
<td>1.519</td>
<td>1.461</td>
<td>1.502</td>
</tr>
<tr>
<td>5</td>
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<td>1.518</td>
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<tr>
<td>8</td>
<td>1.554</td>
<td>1.477</td>
<td>1.520</td>
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<tr>
<td>Average</td>
<td>1.523</td>
<td>1.465</td>
<td>1.516</td>
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<tr>
<td>S.D.*</td>
<td>0.018</td>
<td>0.006</td>
<td>0.015</td>
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* S.D. = Standard Deviation
Table 5.4  The Bulk and Dry Density of (Wet) Calcareous Sand in Large Vessel

<table>
<thead>
<tr>
<th>Test No.</th>
<th>$\sigma_{vo}$ kPa</th>
<th>Time Sat. hrs</th>
<th>$(\omega_v)$ of Sand in the Vessel</th>
<th>Density Obtained by Tins (t/m$^3$)</th>
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<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Midd. Layer</td>
<td>Bott. Layer</td>
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<td>0.441</td>
<td>0.469</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Rigid Base</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
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</tr>
<tr>
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</tr>
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<td>0.399</td>
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<tr>
<td></td>
<td></td>
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</tr>
</tbody>
</table>

$\sigma_{vo}$ = Overburden Pressure  
$\gamma_{bulk}$ = Bulk Density  
$\gamma_d$ = Dry Density  
$\omega_v$ = Moisture Content of the Sand in the Large Vessel  
$\omega_T$ = Moisture Content of the Sand in the Tin.
<table>
<thead>
<tr>
<th>Vessel Type</th>
<th>D/L*</th>
<th>Wall Type</th>
<th>Overburden Pressure (kPa)</th>
<th>Measured Vertical Stress at the bottom of Soil After (5 minutes)</th>
<th>Measured Vertical Stress at the bottom of Soil After (6 Hours)</th>
<th>Predicted Vertical Stress at the bottom of Soil</th>
</tr>
</thead>
<tbody>
<tr>
<td>Large Vessel</td>
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<td>Smooth</td>
<td>50</td>
<td>35</td>
<td>31.5</td>
<td>37.5</td>
</tr>
<tr>
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<td>73</td>
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<td>150</td>
<td>107.5</td>
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<td>151.5</td>
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<td></td>
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<td>Rough</td>
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<td>Medium Vessel</td>
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<td>68</td>
<td>71.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>200</td>
<td>94</td>
<td>90.5</td>
<td>94.3</td>
</tr>
</tbody>
</table>

* D = Vessel Diameter
L = Length of Vessel

**

Phi = 32° for Smooth Wall & 46° for Rough Wall
k = 0.2
Gamma = 10kN/m³
### Table 5.6 Measured Vertical Stress at the Bottom of Silica Sand

<table>
<thead>
<tr>
<th>Vessel Type</th>
<th>D/L*</th>
<th>Wall Roughness</th>
<th>Overburden Pressure (kPa)</th>
<th>Vertical Stress at the bottom of Soil After (5 minutes)</th>
<th>Vertical Stress at the bottom of Soil After (6 Hours)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Large Vessel</td>
<td>1.23</td>
<td>Smooth</td>
<td>50</td>
<td>37</td>
<td>33</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>100</td>
<td>78.5</td>
<td>74.5</td>
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<td></td>
<td></td>
<td></td>
<td>150</td>
<td>114.5</td>
<td>112</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>200</td>
<td>159</td>
<td>157</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Rough</td>
<td>50</td>
<td>30</td>
<td>29.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>100</td>
<td>69</td>
<td>67</td>
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<td></td>
<td></td>
<td></td>
<td>200</td>
<td>128.5</td>
<td>125</td>
</tr>
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</table>

D = Vessel Diameter  
L = Length of Vessel
<table>
<thead>
<tr>
<th>Test No.</th>
<th>Density (kN/m³)</th>
<th>Type of Overburden Pressure</th>
<th>Overburden Pressure (kPa)</th>
<th>Surface Soil Settlement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Top or * Both</td>
<td></td>
<td>A 13mm From C.L.</td>
</tr>
<tr>
<td>1</td>
<td>10.2</td>
<td>Top</td>
<td>50</td>
<td>13</td>
</tr>
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<td>1b</td>
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<td>28</td>
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<td>19</td>
</tr>
<tr>
<td>2b</td>
<td>10</td>
<td>Top</td>
<td>200</td>
<td>30</td>
</tr>
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<td>9.95</td>
<td>Top</td>
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<td>21</td>
</tr>
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<td>9.94</td>
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<td>15.5</td>
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<td>12</td>
<td>9.96</td>
<td>Both</td>
<td>100</td>
<td>23</td>
</tr>
<tr>
<td>13</td>
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<td>Both</td>
<td>100</td>
<td>21</td>
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<td>14</td>
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<tr>
<td>15</td>
<td>9.8</td>
<td>Both</td>
<td>100</td>
<td>27</td>
</tr>
</tbody>
</table>

* Top = A pressure applied only at the top sand surface
Both = A pressure applied at both top and bottom ends of sand
<table>
<thead>
<tr>
<th>Test No.</th>
<th>Density (kN/m³)</th>
<th>Type of Overburden Pressure</th>
<th>Overburden Pressure (kPa)</th>
<th>Surface Soil Settlement After Jacking</th>
<th>Surface Soil Settlement After Cyclic Loading</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Top or * Both</td>
<td>50</td>
<td>A Pile Collar 13mm From C.L (mm)</td>
<td>A Rod Collar 13mm From C.L (mm)</td>
</tr>
<tr>
<td>1</td>
<td>10.2</td>
<td>Top</td>
<td>50</td>
<td>1.6</td>
<td>0.2</td>
</tr>
<tr>
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<td>Top</td>
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<td>2.1</td>
<td>0.25</td>
</tr>
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<td>Top</td>
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<td>0.3</td>
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<td>Top</td>
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</tr>
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<td>100</td>
<td>1.78</td>
<td>0.19</td>
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<td>0.18</td>
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<td>Top</td>
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<td>2</td>
<td>0.28</td>
</tr>
<tr>
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<td>9.9</td>
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</tr>
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</tr>
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<td>-1.11</td>
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<td>Both</td>
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<td>1.02</td>
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</tr>
<tr>
<td>13</td>
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<td>Both</td>
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<td>1.07</td>
<td>0.14</td>
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<td>Both</td>
<td>200</td>
<td>1.35</td>
<td>0.25</td>
</tr>
<tr>
<td>15</td>
<td>9.8</td>
<td>Both</td>
<td>100</td>
<td>0.9</td>
<td>0.12</td>
</tr>
</tbody>
</table>

* = A pressure applied only at the top sand surface
Both = A pressure applied at both top and bottom ends of sand
Figure 5.1 Schematic Layout of Single Sieve Rainer
Figure 5.2 Schematic Layout of Diffusing Rainer Device
A = Single Sieve Rainer Device  
B = Diffusing Rainer Device

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D = Rigid Base Tin

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D = Body of the Small Vessel Consists of Two Pieces
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F = Trolley

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B = Thick Hard Board

Semi-Rigid Base

C = Rubber Membrane

Pressure-Controlled Base

All dimensions in mm

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F = Trolley
G = Air-Water Exchange Bottle

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a) Connector Between Proving Ring and Loading Machine

b) Connector Between Pile Cap and Proving Ring

\[ r_1 = 14 \quad \text{and} \quad r_2 = 21 \]

All dimensions in mm

Figure 5.22 Two Loading Connectors
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B = Pile Cap for 4-Pile Group  
C = Pile Cap for 2-Pile Group with Small Space Between Two Piles  

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C = Two Collars

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B = Rubber O-ring
C = 60mm thickness Aluminium Ring
D = Rubber membrane
E = Sealing collar
F = Cap screw
G = Support for transducer core
U = LVDT transducer
V = Magnetic clamp
Z = Base of Thompson plunger attached rubber membrane
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D = Four Connectors Between Single Piles and Pile-Cap
E = Proving Ring
F = Grouting the Piles into the Pile Cap
G = Vessel Lid

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Chapter 6 RESULTS OF SINGLE PILE TESTS

6.1 INTRODUCTION

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6.4 STATIC AND CYCLIC (DISPLACEMENT–CONTROLLED) TESTS
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Chapter 6

RESULTS OF SINGLE PILE TESTS

6.1 INTRODUCTION

Model tests on single piles jacked into calcareous sand have been performed to study the influence of various parameters and test conditions on the shaft behaviour under static and cyclic loading.

The majority of tests were performed using a large test vessel, and other tests were performed using the small test vessel, as described in Chapter 5. This chapter presents the results of three types of tests on model instrumented piles which have been jacked into New North Rankin calcareous sand consolidated under various overburden pressures. The results of displacement controlled tests on the pile jacked into "medium–dense" and dense sand are summarized in Tables 6.2, 6.3, 6.4 and 6.5. The results of load controlled tests (uniform amplitude) and storm load tests are shown in Tables 6.6 and 6.7 respectively.

Table 6.1 describes the test programme for single pile tests. The following aspects
have been studied in detail:

a) pile behaviour during installation.

b) pile behaviour under static loading in tension.

c) pile behaviour during and after cyclic loading, particularly the effects of cyclic loading on degradation of the skin friction and modulus of the soil under displacement-controlled conditions.

d) stability of the pile during and after cyclic loading and the effects of cyclic loading on degradation of skin friction under both load-controlled and "storm load" conditions.

The influence of the soil density, overburden pressure, moisture content, size of test vessel and the type of vessel base have been studied for the first three aspects. For the last aspect, the tests have been performed using only the large test vessel in which medium dense and dense samples of sand have been consolidated under two different pressures applied simultaneously to both the top and bottom surfaces of the soil.

An instrumented pile has been used for the tests to enable the distribution of load along the pile to be measured during all stages of the tests. A comparison of the results obtained from the tests with those obtained from previous model and field tests is made to give further insight into the behaviour of jacked piles in calcareous sand. It is felt that these results may provide useful experimental data for theoretical analysis in the future.
6.2 SUMMARY OF TEST PROCEDURE

As described in the previous chapter, the sand samples were prepared carefully in both small and large test vessels at different densities. The adopted procedure of raining sand produced a relatively uniform bed of soil and enabled control of the density, with "loose" and "medium dense" sand samples being produced depending on the height of sand raining. The corresponding range of dry unit weight was 9.5 kN/m³ to 10.5 kN/m³. The denser samples were prepared by the same procedure but the wall of the test vessel was vibrated to achieve higher densities. A unit weight greater than 10.5 kN/m³ was obtained, the actual value depending on the duration of vibration.

At the completion of consolidation, the pile was jacked into the prepared sand sample manually using the loading machine. After taking the final set of residual load readings, the pile was attached to the loading machine and tested.

Each test generally consisted of three stages:

1. An initial static loading to 10% of pile diameter, which was assumed to be the failure criterion (Kulhawy and Carter 1988).

2. Cyclic loading of the pile between pre-determined amplitudes of displacement, for displacement-controlled tests, and predetermined load levels, for load-controlled (uniform amplitude and non-uniform amplitude) tests.

3. Final static loading to failure, to investigate the influence of cyclic loading on the subsequent performance of the pile.
The rate of displacement was set constant for all the tests at about 0.4mm per minute to avoid the effect of the rate of loading on the pile capacity response (Yuan and Poulos 1986). A total of 44 jacking, tensile static and cyclic (displacement controlled) tests were performed under different base conditions. These tests which were carried out using the large and small test vessels are presented in Tables 6.1, 6.2, 6.3, 6.4 and 6.5. Tables 6.2 and 6.3 show the base condition of each test, the density of each sample, the applied pressure on the soil and the results obtained from jacking (e.g. residual load, ultimate and peak skin friction and end bearing capacity). The value of the amplitude of cyclic displacement, number of cycles, degradation factor and soil modulus are shown in Tables 6.4 and 6.5.

There were 15 load-controlled cyclic (uniform amplitude) tests and 8 storm load (non-uniform amplitude) tests which were carried out in this single pile test programme and these results are presented in Tables 6.6 and 6.7 respectively.

6.3 BEHAVIOUR DURING JACKING

The total force and the distribution of force along the pile during jacking were measured by strain gauges attached at different points on the pile shaft. The head load was measured by a strain gauge above the sand surface (SG5) or by a proving ring, while the end bearing force was deduced from the strain gauge near the pile tip (SG1) as shown in Figure 5.21 in the previous Chapter. These measurements were plotted against penetration of the pile in dense and medium dense calcareous sand under different overburden pressures and different types of test vessels.

For the tests (DC 4, DC 5, DC 6, DC 7, DC 9 and DC 10) using the large vessel with the rigid base (R.B), Figures 6.1, 6.2 and 6.3 show the behaviour of the
instrumented pile during jacking in "dense" and "medium-dense" samples under three different overburden pressures (i.e. 50, 100, and 200 kPa). The jacking force versus penetration for both densities increased to a peak and then decreased towards an ultimate value. The peak load was well defined for dense sand samples. The strain softening behaviour appears to be more pronounced for the low overburden pressure of 50kPa than for the higher overburden pressures of 100kPa and 200kPa. The maximum values of head load and end bearing capacity for the dense samples were higher than those for the medium dense samples, as can be seen from the values summarized in Table 6.2 and 6.3.

6.3.1 Influence of Vessel Base Conditions

There appears to be little or no difference between the results from the tests using the rigid-base and the semi-rigid base, as shown in Table 6.2, for a given density. The jacking load and end bearing capacity for both end conditions are quite similar. Table 6.2 shows that, for a given density and overburden pressure, the values of ultimate skin friction obtained from jacking are a little smaller than those from tensile static loading (Table 6.4).

The results of the tests (DC 28, DC 29, DC 30 and DC 32) for jacked piles in both "medium dense" and "dense" sands consolidated in the large vessel with controlled base pressure (C.B.P.) are shown in Figures 6.4 and 6.5. The difference between these results and those obtained from the large vessel with a rigid base (R.B) is that the amount of "strain softening" from the C.B.P tests appears to be slightly greater than those in R.B tests. It might be expected that the sand density near the pile tip is affected by the base condition of the test vessel. However, the results of the tests do not indicate a great influence of the base pressure boundary condition on the behaviour of the pile during jacking. It is suggested that the base
condition would have only a small influence on the end bearing capacity of the piles.

Figure 6.6 shows two linear regression curves for the relationship between peak end bearing capacity and overburden pressure for both "medium–dense" and "dense" sands. The end bearing capacity for both densities increases with overburden pressure, but the slope of the relationship for "dense" samples is greater than that for "medium–dense" samples.

6.3.2 Influence of Vessel Size

Figures 6.7 and 6.8 show that the results of jacking a pile using the small vessel are generally similar to those from the large test vessel, but that the value of peak load from the small vessel is somewhat higher. These figures show the influence of soil density on the peak skin friction and end bearing resistance, normalized with respect to overburden pressure, during jacking of the piles. Both increase with increasing density as would be expected. The results are consistent with those of Poulos and Chua (1985) who observed, for shallow foundations, that the bearing capacity increased with the increasing relative density of both silica and calcareous sands. The tendency for the jacking loads obtained from the tests in the small vessel to be higher than those from large vessel is believed to reflect the greater influence of side restraint in the small vessel, especially on the skin friction.

Typical distributions of residual load along the pile in "medium–dense" and "dense" sands, after the jacking force was removed, are shown in Figures 6.9 and 6.10. The maximum compression load is found near the tip of the pile, and a very small amount of compressive residual load exists near the top of the pile. The maximum value of residual load on the pile jacked in dense sand is higher than that in
medium–dense sand. This residual load will not affect the ultimate tensile load in the piles, which is determined by the strength along the failure surface, Stewart and Kulhawy (1981), and Lee (1988), but it is important in properly interpreting the results of a load test to determine the amount of shaft and base resistance developed along the pile.

Figure 6.11 shows the linear regression lines for residual load tests, as a function of initial soil density using both the small and large test vessels. The residual load increases as density increases, but the rate of increase for tests using the small vessel is higher than for those using the large vessel. Again the residual loads at the base of the pile jacked into "medium–dense" and "dense" calcareous sand increase with increasing overburden pressure, as shown in Figure 6.12. It appears that the slope of the linear regression curve for "medium–dense" samples is lower than that for "dense" samples.

6.3.3 Load Distribution

Typical peak and ultimate load distributions in a pile after full penetration in "medium–dense" and "dense" sands from tests DC 3, DC 4, DC 5, DC 6, DC 9, and DC 10 are shown in Figures 6.13 and 6.14 respectively. The tests were subjected to different overburden pressure which were applied only at the top surface of the soil. The values of ultimate and peak jacking load increase with increasing overburden pressure, and as expected the ultimate load in the dense sands is higher than in the medium–dense sands.

In the case of pressure applied at both the top and bottom ends of the "medium–dense" and "dense" sands, the results of peak and ultimate load distribution after full penetration (from tests DC 13, DC 29, DC 30, and DC 32) are shown in
Figures 6.15 and 6.16. Comparison with Figures 6.13 and 6.14 shows that the results in this case are similar to those in the previous case where the overburden pressure was applied only on the top surface.

The slope of the curves indicates the existence of skin friction during the jacking of the pile. This slope, and therefore the skin friction becomes greater as the overburden pressure increases.

6.4 STATIC AND CYCLIC (DISPLACEMENT-CONTROLLED) TESTS

6.4.1 Static Response

The static skin friction along the pile was obtained from either pre-cyclic displacement-controlled tests or from individual static loading tests. The results of these tests are summarized in Table 6.3.

The head load and the load distribution along the pile during jacking and testing were measured by the strain gauges fixed inside the pile. The change in the load during testing has to be combined with the residual load due to jacking and is called here the "actual load", while without considering the residual load, the measured load change is referred to as the "apparent-load".

Figure 6.17a presents a typical relationship between the "apparent-load" and displacement at different points along the pile shaft from test DC 18 under both static and cyclic loadings. For the same test, the relationship between the "actual-load" and the displacement is illustrated in Figure 6.17b. The non-linear load-displacement curves for static and post cyclic response in both figures indicate
that the value of static shear stress is higher than that of post-cyclic shear stress. Similar behaviour has been found from the tests using the small test vessel. This behaviour is also similar to that observed in the direct shear tests under both CNL and CNS conditions which were described in two previous Chapters (3 and 4).

The average shear stress along sections of the pile shaft can be obtained by subtracting the reading in the upper strain gauge from the lower strain gauge divided by the surface area between these strain gauges. Figure 6.17c shows the t–z curves for the same test presented in Figure 6.17 (a and b) previously. The values of static and post cyclic shear stresses are quite similar to those published by Poulos and Chan (1988) from tests in a small test vessel. However, the shear stresses herein are markedly less than those obtained from model grouted piles (Poulos and Lee 1989).

Figure 6.17d is a plot of the cyclic shear stress at four sections of the pile against the number of cycles, and shows the reduction in the shear stress along the instrumented pile as the number of cycles increases.

Further details of the relationships between the "apparent" static load and deflection during static tensile loading for tests in "medium–dense" sand (DC 13 and DC 32) subjected to 100 kPa and 200 kPa applied pressure at both the top and bottom soil surfaces (C.B.P) are shown in Figures 6.18 (a and b). The tensile load increases to a peak load at 0.5 mm deflection and then maintains a more or less constant value to 2.5mm.

The "actual" loads from these tests (DC 13 and DC 32) are plotted against axial deflection in Figures 6.19 (a and b). The loads in all strain gauges except strain gauge (Sg5) start from compression and change to tension as the tensile load is increased. However, the high compression load appearing in strain gauge (Sg1)
reduces with axial deflection until it reaches a more or less zero load value, indicating an almost zero tip resistance in uplift, as would be expected.

Figure 6.20 and Figure 6.21 show the load–deflection relationship for tests carried out in "medium–dense" and "dense" sands consolidated under different overburden pressures using the large vessel with the rigid–base condition (R.B). Figure 6.20 (a and b) shows the results of tests DC 11 and DC 5 which were conducted under $\sigma_{vo}' = 100$ kPa and Figure 6.21 (a and b) shows the results of tests DC 10 and DC 42, tested under $\sigma_{vo}' = 200$kPa. The tensile load capacity and pile head stiffness of the pile in the "dense" sand (for both overburden pressures) are higher than those of the pile in the "medium–dense" sand.

For the "medium–dense" sand at the lower overburden pressure (Figure 6.20a) the load increases to a maximum value and then remains more or less constant, while for the "dense" sand at the same overburden pressure (Figure 6.20b), the tensile load increases to a peak and then decreases to an ultimate value. At the higher overburden pressure no peak point is discernible for both "medium–dense" and "dense" samples, as shown in Figure 6.21 (a and b).

The comparison between the results of the C.B.P tests and those of the R.B tests shows that for medium–dense sands, the pile head stiffnesses from the C.B.P tests appear to be a little higher than those from the R.B tests.

The comparison is made between load–deflection curves for dense sand tests (eg. DC 24 and DC 17) and those of medium dense tests (DC 21 and DC 11) under two overburden pressures, in Figure 6.22. For the "dense" sand, the tensile load increases to a peak load at about d/5 deflection and then drops to an ultimate value. In the case of the medium–dense sand, the load increases to a maximum
value, with little evidence of a subsequent reduction.

Figure 6.23 shows a plot of the average peak skin friction $f_s$ as a function of applied overburden pressure $\sigma_{vo}'$, and reveals a reasonably linear relationship. The values of $f_s$ are lower than those for piles in silica sands. Expressing the normalized static skin friction as $f_s/\sigma_{vo}'$, Figure 6.24 shows the influence of initial soil density on $f_s/\sigma_{vo}'$ obtained from tests using both small and large vessels. The normalized skin friction in both cases increases with increasing density, but the values from small vessel tests tend to be greater than those from the large vessel, again apparently reflecting the influence of lateral restraint.

Comparison between the present results and the results obtained from the grouted pile tests by Lee and Poulos (1990) shows that the value of skin friction for jacked piles in medium dense calcareous sand is less than for grouted piles in calcareous sand of the same density. The higher value of skin friction for grouted piles was also found from field tests (Angmeer et al, 1973; Nauroy and Le Tirant, 1983 and 1985). However, the difference between the skin friction for grouted piles and jacked piles from field tests is higher than that from the laboratory tests (Nauroy, Brucy and Le Tirant, 1988; and Poulos, 1988a).

From the measured load–settlement behaviour, the initial slope may be used with the elastic theory of Randolph and Wroth (1978a) to backfigure the equivalent Young's modulus of the soil. Figure 6.25 shows the initial tangent soil modulus $E_t'$ versus effective overburden pressure for both medium–dense sand and dense sand. The value of $E_t'$ increases as $\sigma_{vo}'$ increases, and is substantially greater for the dense sand than for the medium–dense sand.

Corresponding values of Young's modulus $E_{50}'$, for a load level of 50% of ultimate,
are shown in Figure 6.26. Again $E_{S0}'$ increases with increasing overburden pressure in both medium–dense sand and dense sand, and is smaller than the tangent value $E_t'$. The $E_{S0}'$ values from the present tests are similar to those obtained by Allman (1988) using a vessel similar to the small vessel used here.

As mentioned above, there is not a big difference between the behaviour of model piles in the standard tests, in which the pressure is applied before jacking the pile (R.B, F.B and C.B.P tests). Further investigation of the model pile response was carried out for three non–standard tests (C.B.P.A) in which the pressure was applied at the top end of the soil before jacking the pile and then the pressure was applied at the bottom end of the soil after jacking the pile. The tests were carried out in "medium–dense" sands consolidated under overburden pressures of 100 and 200 kPa using the large vessel. The results of the tests are given in Table 6.5b. The values of skin friction for both overburden pressures are markedly higher than those from the standard tests. The extra skin friction mobilized in the three non–standard tests may arise from an increased lateral stress around the pile accompanying the introduction of the base pressure boundary condition after jacking. This has shown that any means by which the lateral stress will be increased can improve the skin friction developed along the jacked piles.

6.4.2 Load Distribution

The measured load in each strain gauge during tests DC 4, DC 13 and DC 32 is shown in Figures 6.27a, 6.28a and 6.28b respectively. These tests were conducted on piles in medium–dense sand.

The following observations made from these figures are:
a) For all tests, the load monitored by the strain gauges starts from a compressive, residual value.

b) The load near the tip (Sg1) rapidly rises to a peak at a load level of about 0.3 to 0.5 of the static capacity and then remains relatively constant. Similar trends are found for the rest of the strain gauges.

c) The difference between the head load (Sg5) and the tip load (Sg1) becomes fairly constant during tensile loading tests.

The shear stress at any point on the pile is normally represented by "t", where the "t" value at any point is computed simply as the difference in load between two adjacent strain gauges, divided by the surface area of the segment of pile between these two strain gauges.

The distributions of the "t" with depth for tests DC 4, DC 13 and DC 32 are plotted in Figures 6.27b, 6.29a and 6.29b respectively. The skin friction is smaller near the pile tip and increases with the distance from the tip to a maximum value and then slightly decreases towards the top of the pile. The maximum value of "t" appears at a distance of about 1/3 to 1/4 of the pile length below the soil surface. The skin friction increases as the tensile load level increases and the rate of increase decreases with increasing pile movement.

Distributions of load versus depth at 2.5mm tensile displacement (failure) for medium dense sand samples are shown in Figures 6.30a and 6.30b. Each Figure shows the distribution of load in the pile at various overburden pressures and for different base boundary conditions. The shear stress distributions for these tests are plotted in Figure 6.31. The tests shown in these Figures are as follows: tests DC 11 and DC
21 which represent the semi-rigid base condition; DC 12, DC 13, DC 23 and DC 32 which represent the controlled pressure base; and tests DC 14, DC 15, and DC 24 which represent the rigid base condition. The main features to be noted are:

a) The skin friction increases with increasing distance from the tip until it reaches the maximum value at or near the top.

b) The skin friction increases as the overburden pressure increases.

c) Figure 6.31 (a and b) shows that the distribution of shear stress from the tests with controlled base pressure appears to be a little more uniform than that from the tests using the other boundary conditions base.

The influence of overburden pressure on the load distribution and shear resistance along the pile jacked into medium dense sand consolidated in the small test vessel is shown in Figures 6.32a and Figure 6.32b respectively. It can be noticed that at a point near the soil surface, the values of the load distribution obtained from the tests subjected to 200kPa are more than twice than those from the tests subjected to 100 kPa. However, the differences are less marked near the pile tip.

Both the distributions of load and shear stress obtained from these tests are similar to those obtained using the large test vessel. However, the slope of the distribution load curves for tests from the small vessel is lower than for the curves from the large test vessel (Figure 6.30) particularly for the tests subjected to the higher overburden pressure of 200kPa.

The influence of sand density on skin friction developed along the pile during tensile loading tests for different overburden pressures can be further studied by plotting the
shear stress at various depth.

Figures 6.33a and 6.33b show that, for a given overburden pressure, the skin friction increases with increasing density. Generally, the reduction in the load distribution near the tip of the pile can probably be attributed to a reduction in lateral stress caused by the sand grains caving into the void created below the pile tip as the pile moved upward. The zone of reduced lateral stress progresses up the pile with increasing upward pile movement. This explanation was given by Turner and Kulhaway (1987), who tested large scale model pile shafts in silica sand and found a stress relaxation near the pile tip. It is believed that a similar phenomenon occurred in the tests described in this chapter.

Another factor may be the effect of the wall side resistance of the test vessel. Experimental measurements which were carried out to examine the second hypothesis were described in Chapter 5. It was found that the initial vertical stress varied with depth from 200 kPa at the surface of the soil to 150 kPa at the base of the large test vessel and 120kPa at the base of the small test vessel. The vertical stress reduction at the base for large and small vessels due to friction between a vessel boundary and the soil are 25% and 40% of the overburden pressure respectively. For both 100kPa and 200kPa overburden pressures, the shear stress developed on the lower part of the model pile in the small vessel tests and the large vessel tests follow the same pattern as the developed pressure on the base. It can be seen from Figures 6.32 and 6.33 that the small vessel results have a greater reduction in shear stress with depth than the results from the large vessel.

6.4.3 Cyclic Loading Behaviour

The skin friction reduction factor $D_f$ is defined as the maximum skin friction during
cycling divided by maximum static skin friction $f_s$. The influence of density on the magnitude of the reduction factor is demonstrated in Figure 6.34.

Figure 6.34a shows the relationship between the $D_f$ and number of cycles in both the medium dense sand of test DC 11 and the dense sand of test DC 5 for $\sigma_{vo'} = 100$ kPa and a cyclic displacement $\varepsilon_{pc} = 1.25$mm. Figure 6.34b shows the same relationship as that in Figure 6.34a for $\sigma_{vo'} = 200$ kPa and a cyclic displacement $\varepsilon_{pc} = 2.5$mm for tests DC 22 and DC 24. For all tests, the highest rate of reduction occurs in the first 10 cycles and then the rate reduces. The value of $D_f$ for medium–dense sand is greater than that for dense sand indicating that cyclic degradation is more severe for dense sand. The results of Allman (1988) on a jacked pile in dense calcareous sand are also shown in Figure 6.34b, and are consistent with the present results.

Figure 6.34c shows the influence of cyclic displacement on the reduction factor for two tests (DC 22 and DC 20) conducted in "medium–dense" sand. The amount of reduction for the small cyclic displacement $\varepsilon_{pc} = 1.1$mm from test DC 17 is less than that for the large cyclic displacement ($\varepsilon_{pc} = 2.5$mm) from test DC 20.

The skin friction degradation factor ($D_r$) has been defined as the ratio of the post–cyclic skin friction ($f_c$) to the maximum static skin friction ($f_s$).

The variation of $D_r$ with effective overburden pressure is illustrated in Figure 6.35. The value of $D_r$ decreases as $\sigma_{vo'}$ increases for both sand densities. These results differ from some earlier tests (Chan 1986) in which the degradation factor appeared to be more or less independent of $\sigma_{vo'}$.

The influence of cyclic displacement on the value of $D_r$ for medium–dense sand under $\sigma_{vo'} = 100$ kPa and $\sigma_{vo'} = 200$ kPa is shown in Figure 6.36. As found in
previous tests, the value of $D_\tau$ decreases (more degradation) as $\pm \rho_c$ increases. The values of $D_\tau$ for 0.5mm and 0.25mm are similar to those obtained from jacked piles in calcareous sand by Chan (1986). It is noteworthy that relatively low values of $D_\tau$ (0.6 or less) can occur for cyclic displacements in excess of about $\pm$ 0.5 mm, i.e. cyclic loading can cause a severe reduction of skin friction. However, the cyclic degradation for jacked piles is not as severe as for the grouted model piles tested by Lee and Poulos (1990), for which degradation factors less than 0.1 were obtained.

6.4.4 Correlation Between CNS and Model Pile Tests

It was demonstrated in Chapter 4 that the reduction in shear stress during cycling from CNS shear box tests has been predicted from standard CNL cyclic shear box tests. The reduction in shear stress along a model pile can be predicted from the results of the CNS tests performed in Chapter 4. This correlation is illustrated in Figures 6.37 and 38.

Figures 6.37a 6.37b show plots of the shear stress reduction factor "$D_\tau$", which has been defined previously, against number of cycles for different types of tests (model pile and CNS tests) under displacement controlled (\( \pm 1 \) mm ) cyclic loading. The model pile test and CNS test were carried out in "dense" sand.
Figure 6.37a shows the reductions in shear stress at different levels along the pile and the reduction measured in a CNS test, while the average shear stress reduction factors for the model pile are illustrated in Figure 6.37b. For both types of tests, the reduction factor decreases with increasing number of cycles. The rate of reduction in average skin friction of the pile with increasing cycles is similar to that observed in the CNS tests.

The correlation between the shear stress acting on the shaft of a model pile at
failure after cycling and the shear stress from CNS tests is displayed in Figure 6.38 in which the shear stress degradation factor "$D_\tau$" is plotted against cyclic displacement "$\rho_c\)". Seven CNS tests for very dense sands, which were carried out in Chapter 4, and the same number of model pile tests performed in dense sand, are shown in Figure 6.38. This Figure shows that there is good agreement between the response of the model pile and the results of the CNS shear box tests. The general trend shows that $D_\tau$ decreases as $\rho_c$ increases and the rate of decrease for both types of test is high for small cyclic displacement ($\rho_c$). However, the values of $D_\tau$ for the model pile test appear to be slightly lower than those for the CNS tests at \( \pm \rho_c = 0.5 \text{ mm} \), about the same at \( \pm \rho_c = 1 \text{ to } 1.2 \text{ mm} \), and slightly higher than those of the CNS tests at \( \pm \rho_c > 2 \text{ mm} \).

6.4.5 Cyclic Load Distribution

Figures 6.39 and 6.40 show the effects of cycling the pile displacement on the load distribution in the pile after loading to the maximum static load, for four different types of tests. Figures 6.39 (a and b) show the load distribution along the pile shaft after various numbers of cycles for test in "medium-dense" sand subjected to $\sigma_{vo'} = 100\text{kPa}$ applied at the top and bottom boundaries of the sand. The top Figure 6.39a shows the results of test DC 15 with $\rho_c = \pm 2.5 \text{ mm}$. The results of test DC 11 with $\rho_c = \pm 1.25 \text{ mm}$ are shown in Figure 6.39b.

The load and the skin friction (as indicated by the slope of the load distribution) appear to decrease during cycling for both tests. The reduction in skin friction for the test with $\pm \rho_c = 2.5\text{mm}$ is more marked than that for the test with $\pm \rho_c = 1.25\text{mm}$, particularly after the first few cycles. However, the rate of reduction reduces with increasing numbers of cycles for both tests. The reduction in the skin friction can be seen to be very small between the 30th and 50th cycles in test DC.
15 with \( \pm \rho_c = 2.5 \text{mm} \).

Figure 6.40 shows the cyclic load versus depth for a test in "medium–dense" sand (DC 4) and a "dense" sand test (DC 28). Both tests were performed under 200kPa applied pressure and for 1.1mm cyclic displacement (\( \pm \rho_c \)). It can be seen from the slopes that the reduction of skin friction for the pile in the "dense" sample is greater than that in the "medium–dense" sample.

A feature of all the tests described in this section is that the skin friction along the upper part of the pile appears to be reduced more by cycling than that along the lower part, and that the reduction is more pronounced for dense samples.

6.5 EFFECTS OF WATER IN SOIL

To examine the effect of moisture in the sand on the behaviour of the pile, four tests were performed in which the sand was rained into water from a height of 400 mm above the water surface in the test vessel. This height was the same as that used in the model pile tests in dry sand. In the case of the samples containing water, the mean unit weight was found to be greater than that for dry samples as shown in Table 6.4. The skin friction determined from jacking and testing (see Tables 6.2 and 6.4) was found to be a little higher than that for the dry samples, but this was consistent with the higher density of the samples. Thus, it is concluded that moisture does not have a significant effect on the results of pile tests. Similar conclusions were made on the effect of water on the strength of sands in direct shear box tests, which were reported in Chapter 3.
6.6 LOAD–CONTROLLED CYCLIC TESTS

Two series of tests were performed to study the cyclic behaviour of a single instrumented pile jacked into medium–dense calcareous sand. The first series was conducted under uniform amplitude cyclic loading conditions, while the second series was conducted under non–uniform amplitude (storm) loading. The results of all tests with various mean load levels \( P_o \) and cyclic load levels \( \pm P_c \) are summarized in Table 6.6.

6.6.1 Uniform Amplitude Cyclic Loading Tests

A total of 15 load controlled tests were carried out and a summary of the results is presented in Table 6.6. Figure 6.41a shows typical apparent load–deflection curves at the pile head and strain gauge positions along the pile shaft for one–way (tensile) cyclic controlled–load tests. The permanent accumulated displacement increases with the number of cycles.

Figure 6.41b shows the actual load–deflection curves which were obtained from the previous figure after subtracting the residual load. A similar trend to that of Figure 6.41a can be seen in this figure. According to the definition of shear stress which was mentioned in Section (6.4.1), Figure 6.41c shows the \( t–z \) curves which were obtained from Figure 6.41b. The value of shear stress at the upper and lower parts of the pile was less than that at the middle part.

The accumulated displacement developed during cycling increases with increasing mean load level \( P_m/P_s \) and cyclic load level \( \pm P_c/P_s \) as shown in Figure 6.42, where \( P_s \) is the static load capacity. The highest rate of displacement increase is in the first few cycles and the rate of increase decreases with increasing number
of cycles. This observation is supported theoretically by Andersen et al. (1976).

A useful way to represent the results of load controlled tests is the Stability Diagram which was developed by Poulos (1988b). The cyclic and mean load components are normalized by the static load capacity ("Pc/ Ps" and "Pm/Ps"), and are plotted in this diagram. The purpose of the stability diagram is to define the regions in which the pile is stable or unstable under cyclic loading. More modifications of this diagram were suggested by Poulos and Lee (1990) who represented the results of model grouted pile tests in calcareous sand. His diagram also included the results of some field tests which were found to be consistent with the model tests.

The cyclic stability diagram is plotted in Figure 6.43. The line plotted in this diagram represents the approximate boundary between the "failure" and "non-failure" tests. It appears that the accumulated displacement increases as the boundary line is approached and the pile fails under the load combinations on or beyond this line. However, the cyclic degradation may be very small when the pile is subjected to small values of "Pc/Ps" and "Pm/Ps", and the pile may be essentially in the elastic range. These results are consistent with the results of tests on model grouted piles by Lee (1988) and model jacked piles by Allman (1988). Furthermore, these results are supported theoretically by analyses using the computer program "SCARP" which assumes that cyclic degradation begins after reverse slip occurs at a point along the pile. Comparisons between theory and experiment are discussed further in Chapter 8.

6.6.2 Storm Loading Tests

"Storm loading" represents the extreme environmental load condition imposed on offshore piles by storm waves. To simulate storm loading multi-parcel cyclic loading was applied to the model piles. The maximum value and the minimum value of the
load for each parcel has been defined as a percentage of the static ultimate load, similar to that used for uniform cyclic loading.

Two series of storm loading tests which were performed are presented in Table 6.7. The first series (Loading pattern 1) consisted of three to four load parcels in which the maximum load increased gradually. The second series (Loading pattern 2) consisted of six to eight load parcels, with the first three or four parcels of loading being similar to those in the first series, while the last three or four parcels reduced the load gradually. Table 6.7 also shows the maximum recorded loading, maximum accumulated displacement and the number of cycles to cause failure for each of the loading cases. It is observed that the maximum accumulated displacement increases with increasing maximum load level. No post-cyclic loading was carried out for those cases where cyclic loading did not cause failure.

These two test series were performed to collect some information about the effects of each loading pattern on the skin friction along the pile. From the first series, a typical load settlement curve and load pattern for test (ST3) are as shown in Figure 6.44a and 6.44b and a typical relationship for accumulated displacement and number of cycles is shown in Figure 6.44c for the same test. Further data from three tests (ST1, ST2, and ST3) in the first series are summarized in Figures 6.45, 6.46 and 6.47. Each Figure shows the effect of the number of cycles and the magnitude of loading on the accumulated displacement of the pile due to each storm loading parcel.

Figures 6.48 and 6.49 show the same relationships as in Figures 6.45, 6.46, and 6.47 for two tests (ST6 and ST7) in the second series of storm loading tests.
The observations from these tests are as follows:

a) For each load parcel, the accumulated displacement in the first cycle is markedly higher than for the subsequent cycles.

b) For a specific load parcel, the accumulated displacement increases with increasing number of cycles, but the rate of increment decreases with the number of cycles. A similar observation has been made in the uniform load–controlled cyclic tests.

c) The accumulated displacement increases with increasing load level. It appears that the increment of accumulated displacement depends on the load level more than on the number of cycles.

d) It is also noted that the observations made in the first three points are supported theoretically by Lee and Poulos (1991b).

e) It appears that there is an influence of the shape of the loading pattern of the "storm load" on the accumulated displacement developed by the pile. The decreasing load in the second load pattern did not reduce, or contribute much additional displacements to, the accumulated displacement.

6.7 SUMMARY

Model tests on piles jacked in New North Rankin calcareous sand have been carried out to investigate the effects of various test conditions on the behaviour of the pile during jacking, static loading and cyclic loading. The main conclusions may be
summarized as follows:

6.7.1 Behaviour During Jacking

1) The behaviour of an instrumented pile during jacking in vessel containing "dense" and "medium–dense" sand samples under different overburden pressures is summarized in Tables 6.1 and 6.2.

2) The jacking resistance increased as the initial soil density increased, with both the end–bearing and shaft resistance increasing.

3) The jacking resistance (both end bearing and shaft) increased as the overburden pressure increased.

4) The difference between the results obtained from rigid base tests (R.B) and those from (C.B.P) tests is that the strain softening from the first type of test appeared to be slightly more pronounced than from the controlled base pressure tests (C.B.P) tests. This may reveal some small influence of the base condition on the sand density underneath the pile tip, and consequently, the influence of the base condition on the end bearing capacity of the piles.

5) The jacking force obtained in the small vessel was higher than that from the large vessel. This was considered to reflect the influence of increased side restraint in the small vessel, especially on the skin friction.

6) The maximum value of developed residual load on the pile jacked in dense sand was higher than that for a pile jacked in medium–dense sand.
6.7.2 Static Response

1) The tensile load increased to a peak load at about 0.5 mm deflection and then dropped slightly to an ultimate value for medium-dense sand.

2) For dense sand, the peak load appeared to be more pronounced and was developed at a smaller displacement than that for medium dense sand.

3) Both skin friction and soil Young's modulus increased with increasing soil density and increasing overburden pressure.

4) The shear stress along the middle part of the pile appears to be higher than that along the upper and lower parts of the pile.

5) The pile head stiffness from C.B.P tests appeared to be somewhat higher than that from R.B tests.

6) The tests carried out in the smaller test vessel showed generally higher values of skin resistance, indicating that the results of these tests may be influenced by the side-wall restraint of that vessel.

7) The comparison between the present results and the results obtained from grouted piles by Lee and Poulos (1990) showed that the value of skin friction on jacked piles in medium dense calcareous sand was much less than that on grouted piles in the same sand.

8) The skin friction was very small near the pile tip and increased with increasing distance from the tip until it reached a maximum value near to the top.
9) The results for stress distributions along the shaft from the tests with a C.B.P condition show a more uniform shear stress than those from the tests with other boundary base conditions.

6.7.3 Cyclic Displacement-Controlled Response

1) Under cyclic loading, the skin friction tended to decrease or "degrade". Degradation, as measured by \( D_7 \) became more severe as the soil density increased.

2) The degradation of skin friction also became more severe as the cyclic displacement increased and as the overburden pressure increased.

3) The rate of reduction of skin friction appeared to be most severe in the first 10 cycles and then it significantly decreased with subsequent cycles.

4) The reduction in shear stress along a model pile shaft can be predicted from the results of constant normal stress (CNS) shear box tests. The results show a good agreement between the degradation of skin friction of the model pile and predictions using the results of the CNS shear box tests.

5) The degradation of skin friction appeared to be relatively uniform along the pile length.

6.7.4 Cyclic Load-Controlled Response

a) For Uniform Amplitude Tests:

1) The accumulated displacement increased with increasing load level and increasing
number of cycles.

2) A cyclic stability diagram can be used to depict cyclically stable and unstable zones. The stable zone in which the cyclic loading did not reduce the pile capacity appeared to be bounded by a line with a maximum cyclic load of about 0.6 times the static ultimate tensile load. The results here were reasonably consistent with other laboratory and field results.

b) For Storm Load Tests:

1) For each parcel of load, the accumulated displacement in the first cycle is markedly higher than that for subsequent cycles.

2) For a specific load parcel, the accumulated displacement increased with increasing number of cycles, but the rate of increase reduces with the number of cycles. A similar observation was made for the uniform load-controlled cyclic tests.

3) The accumulated displacement increased with increasing load level. The increment of accumulated displacement appeared to depend on the load level more than on the number of cycles.

4) It appears that there is some influence of load pattern on the pile skin friction. The reducing magnitude of load does not greatly reduce the accumulated displacement.
Table 6.1 Summary of Single Pile Test Programme

Single Pile Tests

Jacking Static Cyclic Tests Tests Tests

Disp.- Controlled Load-Controlled Load-Controlled Tests Tests Tests

Uniform Amplitude Non-Uniform Amplitude (Storm Load)
Table 6.2a Results from Jacking Single Piles in "Large Test Vessel"

<table>
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<tr>
<th>Test No.</th>
<th>Base Condition</th>
<th>Dry Density (kN/m³)</th>
<th>Vertical Pressure (kPa)</th>
<th>Embedded Length (L, mm)</th>
<th>Residual Base Load After Jacking</th>
<th>Residual Base Load Prior to Test</th>
<th>Penetration at Peak Load (Z, mm)</th>
<th>Peak Base Load (Sg1, N)</th>
<th>Peak End Bearing Capacity (Sg1, kPa)</th>
<th>Peak Head Load (Sg5, kPa)</th>
<th>Peak Skin Friction During Jacking (kPa)</th>
<th>Ultimate Base Load (Sg1, kPa)</th>
<th>Ult. End Bearing Capacity (Sg5, kPa)</th>
<th>Ultimate Head Load (Sg5, kPa)</th>
<th>Ultimate Skin Friction (kPa)</th>
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<td>75</td>
<td>285</td>
<td>–</td>
<td>–</td>
<td>87</td>
<td>1120</td>
<td>2281.6</td>
<td>1200</td>
<td>11.7</td>
<td>935</td>
<td>1904.8</td>
<td>1050</td>
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<td>75</td>
<td>288</td>
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<td>37</td>
<td>87</td>
<td>1110</td>
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<td>920</td>
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* Rigid = Rigid Base Condition
S.Rigid = Semi- Rigid Base Condition
C.B.P = Controlled Base Pressure (Applied Pressure at Both Top and Bottom Ends)
C.B.P.A = Controlled Base Pressure After Installing Pile

** Wet Sand
### Table 6.2b Results from Jacking Single Piles in "Large Test Vessel"

<table>
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<th>Embedded Length (mm)</th>
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<th>Peak Base Load (N)</th>
<th>Peak End Bearing Capacity (kPa)</th>
<th>Peak Head Load (N)</th>
<th>Peak Skin Friction During Jacking (kPa)</th>
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* Rigid = Rigid Base Condition
S. Rigid = Semi-Rigid Base Condition
C.B.P. = Controlled Base Pressure (Applied Pressure at Both Top and Bottom Ends)
C.B.P.A = Controlled Base Pressure After Installing Pile
**Table 6.3  Results from Jacking Single Piles in "Small Test Vessel"**

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<th>Embedded Length L (mm)</th>
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* Rigid = Rigid Base Condition
  S.Rigid = Semi–Rigid Base Condition
  C.B.P. = Controlled Base Pressure (Applied Pressure at Both Top and Bottom Ends)
  C.B.P.A = Controlled Base Pressure After Installing Pile
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<th>Vertical Pressure (kPa)</th>
<th>Cyclic Displacement (mm)</th>
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<th>Static Skin Friction (kPa)</th>
<th>Cyclic Load (N)</th>
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* Rigid = Rigid Base Condition
S. Rigid = Semi-Rigid Base Condition
C.B.P. = Controlled Base Pressure (Applied Pressure at Both Top and Bottom Ends)
C.B.P.A = Controlled Base Pressure After Installing Pile

** Wet Sand
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<th>Cyclic Displacement</th>
<th>No. of Cycles</th>
<th>Static Load (N)</th>
<th>Static Skin Friction (kPa)</th>
<th>Cyclic Load (N)</th>
<th>Cyclic Skin Friction (kPa)</th>
<th>Degradation Factor</th>
<th>Initial Tangent Soil Modulus Et (MPa)</th>
<th>Secant Soil Modulus Ec50 (MPa)</th>
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*Rigid = Rigid Base Condition
S.Rigid = Semi-Rigid Base Condition
C.B.P. = Controlled Base Pressure (Applied Pressure at Both Top and Bottom Ends)
C.B.P.A = Controlled Base Pressure After Installing Pile
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<th>Base Condition</th>
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<th>Cyclic Displacement</th>
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Rigid = Rigid Base Condition
S. Rigid = Semi-Rigid Base Condition
C.B.P. = Controlled Base Pressure (Applied Pressure at Both Top and Bottom Ends)
C.B.P.A = Controlled Base Pressure After Installing Pile
Table 6.6  Summary of Single Pile Results from Load—Controlled Tests

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<th>Density (kN/m$^3$)</th>
<th>Overburden Pressure (kPa)</th>
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*Ps = Ultimate Static Load (about 210 N)
Figure 6.1  Strain Gauge Loads Versus Penetration During Pile Jacking
Figure 6.2 Strain Gauge Loads Versus Penetration During Pile Jacking
Figure 6.3 Strain Gauge Loads Versus Penetration During Pile Jacking
Figure 6.4 Strain Gauge Loads vs Penetration During Jacking of Single Pile
Figure 6.5  Strain Gauge Loads Versus Penetration During Pile Jacking
Figure 6.6 The Influence of Overburden Pressure on Peak End Bearing Capacity During Jacking
Figure 6.7 The Influence of Soil Density on Normalized Peak End Bearing Capacity During Jacking

Figure 6.8 The Influence of Soil Density on Normalized Peak Skin Friction During Jacking
Figure 6.9  Residual Load Distribution Along the Length of Single Pile (Medium–Dense Sand)

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$P_s = \text{Ultimate Static Load}$

$P_{st} = \text{Storm Load}$


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Chapter 7

RESULTS OF PILE GROUP TESTS

7.1 INTRODUCTION

Several series of static and cyclic tests on two different types of model pile groups jacked into calcareous sediment have been performed to investigate the factors that influence the skin friction of pile groups in this sediment. The first type of test used a group of two piles, and the second type used a group of four piles, as described in Chapter 4.

This chapter presents the results of the tests on both types of pile group jacked into medium dense and dense New North Rankin calcareous sand which was consolidated in a large test vessel. Table 7.1 summarizes the pile group test programme, while the results are summarized in Tables 7.2 to 7.11.

The aims of the investigation included the following aspects:

1) to study the effects of sand density and overburden pressure on pile-group
behaviour during installation.

2) to study the effects of the method of installation on the behaviour of the pile groups.

3) to determine the jacking load and the skin friction generated when the piles are jacked individually and jacked as a group that is connected by a rigid cap, as described in Chapter 5.

4) to determine the skin friction of the pile group under tension loading and the degradation of the skin friction when the pile is subjected to various types of cyclic loading (displacement-controlled, load-controlled and storm loading).

5) to enable these results to be compared with the results of single pile tests which were reported in Chapter 6.

6) to enable these results to be compared with other results of tests for groups installed in clays or silica sand which were obtained from previous investigators.

All the investigations carried out herein employed a constant pile diameter of 25 mm and total pile length of 450mm. The embedded pile length in the sand was about 290 mm. As described in Chapter 4, a proving ring was used to enable determination of the total load on the pile-group (group capacity) during installation as well as during subsequent static and cyclic loading. The instrumented pile allowed determination of the head load and load distribution along one pile within the pile groups.
7.1.1 Scope of the Tests

The test programme which is summarised in Tables 7.2 to 7.11 consisted of 5 series of tests which were performed using the large testing vessel with applied pressure at both ends of the sand sample. The five series of tests involved:

a) Jacking piles individually followed by tensile static load to failure and then followed by controlled-displacement cyclic loading. The tests were performed with different sand densities (4 tests). The test results are summarized in Tables 7.2 and 7.5.

b) Jacking piles as group followed by tensile static load to failure and then followed by controlled-displacement cyclic loading. The tests were performed with different sand densities (15 Tests). Test results are presented in Tables 7.3 to 7.7.

c) Jacking piles followed by controlled-load cyclic loading; performed on "medium-dense" sand (18 Tests). The test results for cyclic loading are summarized in Tables 7.8 and 7.9.

d) Jacking piles followed by one-way Storm loading; performed on medium-dense sand (8 Tests). The test results are summarized in Tables 7.10 and 7.11.

e) Jacking piles followed by two-way Storm loading; one test was carried out and the test results are shown in Table 7.11.

In the first series of tests, the piles were jacked individually, while the piles were jacked together for the other four series of tests. The following aspects of pile
behaviour have been investigated in detail:

1) Pile Installation.
2) Static Loading.
3) Cyclic Loading (Controlled Displacement).
4) Cyclic Loading (Controlled Load).
5) Storm Loading.

7.2 EFFICIENCY AND SETTLEMENT OF PILE GROUP

It is worthwhile defining the terms used to describe the efficiency of pile groups with respect to bearing capacity and settlement, before the discussion of the results of the pile group tests commence.

The behaviour of the pile group depends mainly on the type of the soil surrounding and underlying the group, and on the type of pile group which may be classified as explained below:

The two types of pile groups which are considered are:

i) Free standing group; in which the pile cap is above ground and not in contact with the soil.

ii) Piled foundation; in which the pile cap is placed directly on the ground or buried under the ground.

The pile group which is dealt with in this thesis is the free standing type.

For the first type of pile group, the interaction only occurs between the piles and
surrounding and underlying soil, while the interaction system includes the pile cap for the second type of pile group. However, for both types of pile group, the average load capacity of a pile in the group \( P_{av} \) is defined as the ratio of ultimate load capacity (total load) of the group to the number of the piles in the group:

\[
P_{av} = \frac{P_g}{N} \quad (7.1)
\]

where

- \( P_g \) = ultimate load capacity of the pile group
- \( N \) = number of piles in the group

It is well known that there are two parameters commonly used to relate the behaviour of pile groups under the static loading condition. The first is group efficiency (\( \eta \)) which is defined as the ratio of average load \( P_{av} \) obtained from equation (7.1) to the single pile load capacity, where,

\[
\eta = \frac{P_{av}}{P_s} \quad (7.2)
\]

and

\( P_s \) = ultimate load capacity of single pile

The second parameter, for both a "free standing group" and a "piled foundation group", is settlement ratio (\( R_s \)) which is defined as the ratio of settlement of the pile group to the settlement of a single pile at the same average load per pile, where,

\[
R_s = \frac{\rho_g}{\rho_s} \quad (7.3)
\]
where

\[ \rho_g = \text{settlement of group for group load of } (N \times P) \]

\[ \rho_s = \text{settlement of single pile for a load of } P \]

Several empirical correlations have been used to determine the value of \( \eta \), among which is the "Converse–Labarre" formula (Poulos and Davis, 1980),

\[ \eta = 1 - \xi \left[ \frac{(n-1)m + (m-1)n}{90 \frac{m}{n}} \right] \ldots (7.4) \]

where

\( m = \text{number of rows} \)

\( n = \text{number of piles in row} \)

\( \xi = \arctan \frac{d}{s}, \text{in degrees} \)

\( d = \text{pile diameter} \)

\( s = \text{centre to centre spacing of piles} \)

The most widely used formula for estimating the load capacity of a pile group is that given by Terzaghi and Peck (1948) in which the possibility of the group failing as a single "block" is considered, together with the single pile mode in which the piles act individually.

The static group efficiency is generally found to be slightly less than 1 in dense sand and more than 1 in loose sand (Ranganatham and Kaniraj, 1978). The settlement ratio due to static load is found to be greater than 1 for pile groups. This means that the settlement of a pile group is more than that of a single pile for the same average load per pile. Skempton (1953) explained that the higher settlement for the pile group may be attributed to the larger extent of the pressure
bulb underneath the pile group. This greater influence of a pile group will result in greater compression of the sand mass and hence a larger settlement for the group of piles.

The two parameters $\eta$ and $R_s$ have been introduced for pile groups under static load conditions and these parameters are difficult to calculate accurately, especially for sands. Hence, these two parameters need to be studied more. The change in the efficiency and settlement of a pile group when subjected to cyclic loading also needs study.

7.3 BEHAVIOUR DURING JACKING

The pile jacking tests have been classified into two types, as follows:

i) tests on piles jacked individually, and

ii) tests in which jacking of all the piles together is carried out.

For the first type, each pile is installed individually, one after another, using a loading machine to push the piles into a consolidated sand bed through special collars, as described previously in Chapter 5. This type of installation is similar to the normal sequence of installation of piles. The influence of this sequence of installation allows investigation of not only the pile group capacity under static and cyclic loading, but also of the interaction between two piles for different spacings. The interaction behaviour of two piles at different spacings will be discussed in detail in Section 7.7.
The second type of installation involves pushing all of the piles in the group together by an applied jacking load on the pile cap, using the same loading machine as for the individually jacked piles. The reason to use this type of installation will be discussed in Section 7.2.2. However, this type of installation was similar to that previously used by Vesic (1967) for driving pile groups into silica sand.

The value of these two series of tests is to observe and understand the behaviour of piles in a group under various conditions, so that a comprehensive understanding of the static and cyclic response of a pile group may be obtained.

7.3.1 Piles Jacked Individually

These tests were carried out on four piles in a symmetrical square group with a centre-to-centre spacing of 4 pile diameters, as shown in Figure 7.1.

The behaviour of four piles in a group installed in this way, obtained from tests 4PDC–1, 4PDC–4 and 4PDC–3, has been studied by plotting the jacking load versus the order of pile jacking in the group for medium–dense and dense sand under different overburden pressures in Figures 7.2a and 7.2b. The jacking force generally increases as the sand density increases or as the overburden pressure increases for the same density, as expected.

For medium–dense sand consolidated under overburden pressures of 100 and 200kPa, the general trend is for the jacking load to increase from the 1st jacked pile to the 4th jacked, as shown in Figure 7.2a. However, the ratio of the jacking load between two consecutive piles (i.e., 1st to 2nd pile, 2nd to 3rd pile and 3rd to 4th pile) appears to be more or less constant. All of these load ratios were less than 1. This trend is attributed to the change in the density of the soil between the piles due to
jacking of the piles.

The jacking load behaviour of the pile group in dense sand is shown in Figure 7.2b. The jacking load of each pile in this case is higher than that in the case of the piles in medium dense sand, as expected. It can be seen that the ratios between the jacking forces for two consecutively jacked piles increases until the ratio between the forces for the 3rd to 4th piles becomes greater than 1. This perhaps suggests that some critical density has been achieved.

7.3.2 Piles Jacked Together in a Group

This series included tests of 4-pile and 2-pile groups. The four pile group was jacked in a symmetrical square pattern and the spacing was 4d. For the 2-pile group, tests were performed with centre to centre spacings of the piles equal to that of the 4-pile group. Each group contained one instrumented pile.

During jacking, the total load on the group was measured by a proving ring which was connected between the loading machine and the pile cap, while the strain gauges along the instrumented pile measured the force distribution for various amounts of penetration. For each test, the mean jacking load is defined as the total load $P_g$ divided by the number of piles in the group.

Strain gauge "Sg5" located at the top of the instrumented pile, and above the soil surface, measured the jacking head load of that pile. The base load carried by a single pile in the group has been assumed to be measured by the bottom strain gauge "Sg1", and this assumption has been explained previously in Chapter 5. This base load was considered to be the average base load of the group because there was no direct means for measuring the total base load for the group.
The change in the jacking load from one pile to another within a group jacked into both "medium–dense" and "dense" sands has been shown in Figures 7.2a and 7.2b respectively. Thus the distribution of load along each pile in the group is expected to be dissimilar. It is reasonable to believe that jacking the piles in a group (all together) may produce a more similar distribution of load along these piles than that obtained by the first type of jacking. It has been found from the first few tests for piles jacked together, that the value of the mean load is about the same as the value of the individual jacking load for the instrumented pile measured by the strain gauge "Sg5". This argument was confirmed by the results of the tests for both types of installation, which are presented in Tables 7.3 and 7.4.

For a particular sand density and overburden pressure all the tests carried out here conform to a general trend where sharp increases in jacking load and load at strain gauge locations occurred as the penetration depth increases until the peak was reached, after which there was a decrease to an ultimate value.

The average load and base load responses from the jacking of the 2–pile and 4–pile groups from tests 2PDC–2 and 4PDC–5 respectively were studied by plotting jacking force against penetration curves for medium dense sand with an overburden pressure of 100kPa (see Figures 7.3 and 7.4). These two figures and the results from other tests presented in Tables 7.3 and 7.4 indicate that the values of mean head loads and base loads for the 4–pile group are slightly greater than those of the 2–pile group, which is in turn slightly higher than those of single piles. The results for single piles have been presented in the previous Chapter.

A comparison of the head and base load responses of 4 piles in a group obtained from test 4PDC–5 for the same overburden pressure and similar density is given in Figure 7.5. As shown in Figure 7.5, the stiffness of a single pile is higher than that
of a pile in a 2–pile group which is in turn stiffer than that measured in the 4–pile group.

As expected, from the previous test results on single piles and from other published data (Chan 1986 and Allman 1988), the jacking load required for penetration increases significantly as the overburden pressure and density increase.

It has been found that the strain softening behaviour for one pile in a group jacked into soil of different densities is similar to that for a single pile. Results for the peak load distribution at full penetration obtained from tests 2PDC–2, 2PDC–3, 2PDC–5, and 2PDC–6 for the 2–pile groups and from tests 4PDC–5, 4PDC–4, 4PDC–7, and 4PDC–8 for the 4–pile groups, are shown in Figures 7.6 and 7.7. The tests were performed under overburden pressures of 100 and 200 kPa. As expected the peak skin friction increases as overburden pressure increases for the same soil density.

The influence of overburden pressure on peak skin friction capacity for tests carried out on a single pile in medium–dense sand are shown in Figure 7.8. It can be seen that the regression curve of these tests significantly increases with increasing overburden pressure. The reason for representing this figure is to clarify how the dotted regression curve for the single pile tests was defined, which is shown again in Figure 7.9.

Figure 7.9 shows the peak skin friction versus overburden pressure for several tests on single piles, 2 pile groups, and 4 pile groups, for comparison purposes. The general trend is for the peak skin friction to increase with increasing overburden pressure. However, the regression curve for the 4–pile group is seen to be higher than for the 2–pile group and single pile. The regression curve for the 2–pile group appears to be slightly lower than for the single pile at the low overburden pressure,
and increases gradually until it becomes slightly higher than that of the single pile at higher overburden pressures. Figure 7.9 indicates that the number of piles in the group may slightly influence the peak skin friction capacity of the piles.

The end bearing capacity of single piles jacked into medium–dense sand under different overburden pressures is shown in Figure 7.10. The general trend is represented by a linear regression line which increases as overburden pressure increases, as expected. This regression line again appears in Figure 7.11 for comparison purposes with pile groups.

Figure 7.11 shows the peak end bearing capacity against overburden pressure for single piles and groups. The general trend is for end bearing capacity for all groups to increase with increasing overburden pressure. The regression line for 4–pile groups plots slightly higher than that of 2–pile groups which is also a little higher than that for single piles.

The influence of overburden pressure on the residual load distribution in the instrumented pile in the 2–pile group and 4–pile group tests under overburden pressures of 100 kPa (eg. 2PDC–4 and 4PDC–5) and 200 kPa (tests 2PDC–5 and 4PDC–8) is shown in Figures 7.12 and 7.13 respectively. Figure 7.12 shows that the residual load observed from each instrumented pile in the 2–pile and the 4–pile groups are about the same. In contrast, it appears that the residual load from the 4–pile group in test "4PDC–8" is slightly higher than that from the 2–pile group in test "2PDC–5". The number of piles in the group may have a smaller influence on the residual load for the tests at lower overburden pressure than at higher overburden pressure.

Two 4–pile group tests, 4PDC–3 and 4PDC–5, were performed in "dense" and
"medium–dense" sands, and results are shown in Figure 7.14. It can be seen that the residual load mobilized along one pile in dense sand is higher than that in medium dense sand.

Figure 7.15 shows a comparison of the maximum residual load mobilized along one pile for different pile groups. This is studied by plotting maximum residual load versus overburden pressures for medium–dense sand. The regression lines for the residual load for three types of group increase with increasing overburden pressure. However, the residual load for the 4–pile group is greater than that for the 2–pile group, which is greater than for the single piles (Figure 7.15).

7.4 STATIC BEHAVIOUR

As mentioned in the previous chapter the load–settlement behaviour for tensile loading of single piles considered the residual (compressive base and tensile shear) load due to jacking. In the current chapter, the load–settlement behaviour of a pile group also takes into account the residual load mobilized from jacking of the group. The measured load change less the residual load is referred to as "actual load", while the "apparent load" is used for the measured static load before subtracting residual load as described in Chapter 6. The results of static tests carried out on both 4–pile groups and 2–pile groups are summarized in Tables 7.5, 7.6a and 7.7a. These tables show the values of mean static load, and the head and base load. The various sand densities and overburden pressures used in the tests are also included.

Typical actual load–settlement behaviour from static loading of a 4–pile group (4PDC–5) is shown in Figure 7.16. The sand used for this test was medium–dense. When the tension load increased, as shown in Figure 7.16, the loads measured from
all strain gauges in the instrumented pile (except the top strain gauge Sg5), started from compression but became tensile. However, the total load on the group measured by the proving ring started from zero. As was observed for the case of a single pile, the high compressive load observed at the bottom strain gauge Sg1 decreased with axial deflection until it reached a more or less zero tensile load value, indicating essentially zero tip resistance in uplift, as would be expected. This behaviour was similar for all types of pile groups.

Two groups of load–deflection curves obtained from tensile loading of 2–pile groups (from 2PDC–1 to 2PDC–6) and of 4–pile groups (from 4PDC–1 to 4PDC–7) are shown in Figures 7.17 and 7.18. The general trend is for the tensile load to increase with increasing axial deflection. However, as expected the load capacity (total load) of the 4–pile groups is higher than that of 2–pile groups for the same overburden pressure and soil density.

Figure 7.19 shows a plot of the load capacity of a group versus overburden pressure for the tests on 2–pile and 4–pile groups. The trend for both types of group is for load capacity to vary linearly with overburden pressure. The regression line for the 4–pile group is higher than that for the 2–pile group, and the slope is also greater.

From Figures 7.17 and 7.18 for a particular overburden pressure and group type, the difference in load capacity of the group is not more than 15%. This indicates the consistency of the tests.

One load–settlement curve from each type of pile group which were shown in Figures 7.17 and 7.18, as well as one curve from a single pile test (DC 15) presented in Chapter 6, are shown in Figure 7.20. The 4–pile group shows the highest capacity as expected.
Figure 7.21 shows the single pile load capacity, and the average load capacity $P_{av}$ for both 2-pile and 4-pile groups. It can be seen that the average load for a 4-pile group is higher than that for a 2-pile group which is higher than that of a single pile. Similar behaviour has been predicted from theoretical analyses by Poulos, Randolph and others, and from experimental work summarized by various authors e.g. Skempton 1953; Vesic 1967; Hanna 1963; Ranganatham Kaniraj, 1978.

The influence of number of piles in a group on the group efficiency ($\eta$) under the same overburden pressure of 100 kPa is given in Figure 7.22. The mean values joined by a line for the single pile, 2-pile group and 4-pile group are also shown in this figure. It appears that the group efficiency increases slightly as the number of piles in the group increases.

Figure 7.23 shows the relationship between the group efficiency $\eta$ of 2-pile and 4-pile groups and sand density. It appears that $\eta$ increases with increasing sand density. Similar observations on the jacking of pile groups in silica sand have been made by Vesic (1967) and Ranganatham Kaniraj (1978), again indicating that the soil density plays an important role in determining the pile capacity. Vesic (1967) reported that the soil density may affect pile group efficiency more than the number of piles in a group.

Figure 7.24. shows that at failure, the average load of the 4-pile group is slightly higher than that of the 2-pile group which is also slightly higher than that of the single pile. This indicates that the sand among the piles is densified due to jacking of the piles.

The settlement of a pile group can be studied through the stiffness of the group. The stiffnesses obtained from the average load $P_{av}$ of three tests are also shown.
in Figure 7.24. The stiffness of a single pile is slightly higher than that of the 2-pile group which is higher than that of the 4-pile group. This behaviour is shown more clearly in Figure 7.25, in which the secant pile stiffness is plotted against load level for the same three tests. The general trend for the three types of group is that the secant stiffness decreases with increasing load level, and the highest stiffness occurs for single pile tests.

Figure 7.26 summarizes results of all the tests performed on single piles, 2-pile and 4-pile groups, which were carried out for a particular value of load level, \( P/P_s = 50\% \). All the tests were performed on medium dense sand and at the same overburden pressure of 100kPa. Again, it is clearly seen that the secant pile stiffness reduces with increasing numbers of piles in a group.

The secant stiffness of the pile group \( (K_g) \) is defined as the average load per pile divided by pile settlement. The normalized secant stiffness of the pile group is defined as the secant stiffness of the pile group \( (K_g) \) divided by the secant stiffness of a single pile \( (K_s) \). The relationship between the mean value of \( K_g/K_s \) and number of piles in the group is presented in Figure 7.27. Again the normalized secant pile stiffness decreases as the number of piles in the group increases. This indicates that, interaction between piles in the group has occurred, thus reducing pile group stiffness.

From the last four figures, it appears that, despite the slight densification of the sand due to jacking of the piles in a group, interaction between piles in a group has occurred, and increasing the number of piles in the group has reduced the pile group stiffness.
7.4.1 Load Distribution

The distribution of load along the instrumented pile in each pile group has been measured by the strain gauges during tensile loading, in a similar manner to that for the single pile tests.

Figures 7.28 and 7.29 present the relationship between the tensile load and depth during tensile loading for a 2-pile group (2PDC−4) and a 4-pile group (4PDC−5), both jacked in medium dense sands subjected to 100kPa applied pressure at both ends of the sand bed. The following observations from these two Figures may be made:

1) the load at each point along the instrumented pile starts from a compressive residual value, except that measured by strain gauge "Sg5".

2) the base load of the pile decreases with increasing pile movement towards failure until it becomes close to zero at failure.

3) the lower part of the pile at failure carries very little load, and this load increases towards the upper part which carried the majority of load. Noting that the slope of the load distribution in the upper part of the piles is nearly constant, it can be inferred that pile-soil slip generally commences near the top part of the pile and progresses downward with increasing pile group movement.

4) slip at the lower part of the pile may not occur unless a large pile settlement of about 10% of pile diameter occurs. This behaviour is similar to that of the single pile discussed in the previous Chapter and it is also supported theoretically by Poulos (1981).
The development of skin friction along one pile in 2-pile and 4-pile groups during tensile loading can be further studied by converting the relationships in Figures 7.28 and 7.29 to shear stress versus depth relationships; these are presented in Figures 7.30 and 7.31. It can be seen that the general behaviour for these tests appears to be similar, in that the skin friction increases with increasing tensile displacement. The skin friction for the 2-pile group from test 2PDC-4 in Figure 7.30 appears more uniform than that of the 4-pile group from test 4PDC-5 in Figure 7.31. The uniformity of skin friction was observed in a single pile test performed under the same condition of applied pressure on medium dense sand samples. This indicates that the uniformity of skin friction may be influenced by the jacking of piles in a group.

7.5 CYCLIC LOADING BEHAVIOUR (DISPLACEMENT-CONTROLLED)

The skin degradation friction factor $D_T$ is defined as the ratio of the average post-cyclic skin friction ($f_{gc}$) to the average static skin friction of a group ($f_{gs}$), i.e.

$$D_T = \frac{f_{gc}}{f_{gs}} \quad \text{................. (7.5)}$$

where the average static skin friction ($f_{gs}$) is calculated by dividing the average static tensile load capacity of a pile in a group ($P_{avS}$) by the surface area of the embedded length of one pile in the group.

$$f_{gs} = \frac{P_{avS}}{\pi d z} \quad \text{................. (7.6)}$$

where
\[ d = \text{pile diameter} \]
\[ z = \text{embedded pile depth} \]

and the average post-cyclic skin friction \( f_{gc} \) is calculated by dividing the average post-cyclic load capacity \( P_{ave} \) by the surface area of the embedded length of one pile in the group.

\[ f_{gc} = \frac{P_{ave}}{\pi d z} \quad \ldots \ldots \ldots \ldots \quad (7.7) \]

Values of \( D_r \) for two types of pile groups are included in Tables 7.6b and 7.7b. Figures 7.32 and 7.33 show the degradation factors for the three types of groups (i.e. single pile, 2-pile group and 4-pile group) versus number of cycles for different values of cyclic displacement and overburden pressure.

Figure 7.32 shows the influence of the number of piles in a group on the degradation factor for three typical tests (i.e. 4-pile group 4PDC-5, 2-pile group 2PDC-3, and single pile DC 11) in medium-dense sand subjected to 100 kPa overburden pressure and about the same cyclic displacement (i.e. \( \pm \rho_c \) for 4PDC-5 and 2PDC-3 = 1.2mm, and \( \pm \rho_c \) for DC 11 = 1.25mm). From Figure 7.32, it can be seen that the general trend is for the degradation factor "\( D_r \)" for all types of group to decrease with increasing number of cycles, with the highest rate reduction of \( D_r \) occurring in the first few cycles. However, \( D_r \) for the 4-pile group is slightly less than for the 2-pile group which is in turn slightly less than for a single pile.

Figure 7.33 shows three tests (i.e. single pile DC 22, 2-pile group 2PDC-6, and 4-pile group 4PDC-7) in a medium-dense sand subjected to 200 kPa overburden pressure and 50 cycles. The cyclic displacement was \( \pm \rho_c = 2.5 \text{ mm} \) for both the single pile and 2-pile group tests, and \( \pm \rho_c = 1.2 \text{ mm} \) for the 4-pile group test.
Comparison may be made between the single pile test and the 2-pile group, and it can be seen that $D_r$ for the 2-pile group test is less than that for the single pile test. Comparison between the 2-pile group and the 4-pile group tests can also be made in Figure 7.33. It appears that the $D_r$ of a 2-pile group is lower than that of a 4-pile group, indicating that the degradation factor may depend on the cyclic displacement more than it depends on the number of piles in the group.

The influence of overburden pressure on degradation factor can be revealed from the comparison between two tests carried out on the 4-pile group. The first test (4PDC-5) was performed under 100 kPa (Figure 7.32) and the second test (4PDC-7) was performed under 200 kPa (Figure 7.33). After 50 cycles, the value of $D_r$ for the test under 100kPa overburden pressure is 0.56 which is higher than that for the test under 200kPa overburden pressure which is 0.43. Similar behaviour was observed previously in the single pile tests. This indicates that the overburden pressure influences the degradation factor for single piles and pile groups.

From the results discussed in this section, it can be inferred that the overburden pressure, sand density, and cyclic displacement affect the load capacity of a pile group subjected to cyclic loading more than does the number of piles in the group.

### 7.5.1 Cyclic Load Distribution

Figures 7.34 and 7.35 show the effects of cyclic loading on the load distribution along the instrumented pile for a 2-pile group test (2PDC-4) and a 4-pile group test (4PDC-5). The general trend for the behaviour of pile groups for both tests appears similar to the single pile behaviour.

Both figures show a reduction in the load along the pile during cyclic loading, and
the rate of the degradation decreases as the number of cycles increases. The highest reduction in load occurs near the top part of the pile. However, the reduction in the load near the lower part of the instrumented pile for test (4PDC-5) on the 4-pile group is higher than that for test (2PDC-4) on the 2-pile group.

It has been observed from the single pile test in Section 6.4.4 that the reduction in skin friction near the tip of a pile jacked into dense sand is higher than that in medium-dense sand. Thus, the difference between the skin friction near the tip of the pile in tests 4PDC-5 and 2PDC-2 may be attributed to the more densified sand layer near the tip of piles caused during jacking the higher number of piles in the 4-pile group.

7.6 CYCLIC LOADING BEHAVIOUR (LOAD-CONTROLLED)

This section presents results of two series of model pile group tests in calcareous sand subjected to different types of load-controlled cyclic loadings. The pile groups involved in this study are 2-pile and 4-pile groups. One test series was performed on both types of groups under a uniform amplitude of cyclic loading. The other series was performed under two different patterns of storm loading. The study herein is concerned mainly with the accumulation of pile displacement, and the capacity of the pile group after a particular number of cycles is also considered.

7.6.1 Uniform Amplitude Cyclic Loading Tests

Table 7.8 summarizes the results of 10 tests on 2-pile groups and Table 7.9 summarizes the results of 8 tests on 4-pile groups. Both tables show density, overburden pressure, ratio of mean load capacity of the group to the static load
capacity \( (P_m/P_s) \), ratio of cyclic load capacity of the group to the static load capacity \( (P_m/P_s) \), and the number of cycles to cause failure.

It was found that the load transfer at different points along the instrumented pile in a group for cyclic controlled load tests was similar to that of a single pile, as described in Section 6.4.1. The accumulation of pile head displacement increases as the number of cycles increases.

The influence of the number of piles in a group on the accumulated displacement for a particular load level and number of cycles is shown in Figure 7.36, where the displacement amplitude is plotted against number of cycles. For all types of group, the accumulation of displacements increases as number of cycles increases, but the rate of accumulation decreases with number of cycles. The highest displacement increment can be seen in the first cycle, which was also observed from cyclic simple shear tests (Andersen, 1976).

The accumulation of displacement for the 2-pile group appears slightly lower than the 4-pile group which is about the same as the single pile at 10 cycles. However, the trend is changed after 10 cycles and the accumulated displacement of the 4-pile group appears slightly higher than the 2-pile group which is a little higher than a single pile at 50 cycles. This indicates that there is some effect of pile group interaction on the accumulated displacement of a group. This behaviour is similar to that observed from model pile groups in silica sand (Moey, 1983) and in clay (Hewitt 1988) as reported in the literature review in Chapter 2. However, it appears that the difference in the magnitude of accumulated displacement between a pile group and a single pile as reported in the literature is greater than that observed in the present tests. The full scale pile test in silica sand under cyclic loading conditions of Van Weele (1979) confirmed that the cyclic displacements depend more
on cyclic load level than number of cycles.

Figures 7.37a and 7.37b show the cyclic stability diagram for 2-pile groups and 4-pile groups respectively. In each diagram, the mean load normalized by the static load capacity \( \left( \frac{P_m}{P_C} \right) \) is plotted against cyclic load normalized by the static load capacity \( \left( \frac{P_C}{P_S} \right) \). The line in each stability diagram represents the approximate boundary between "failure" and "non-failure". The purpose and the function of the stability diagram for a pile group is similar to that for a single pile (see Chapter 6). The results for pile groups appear consistent with those obtained from single pile tests.

Comparisons between the results of uniform cyclic loading for pile groups and theory are discussed further in Chapter 8.

7.6.2 Storm Loading Tests

This section describes the results of the non-uniform amplitude (storm) cyclic loading tests carried out on 2-pile and 4-pile groups. The storm loading of multiple parcels of cyclic loading applied to the model pile group was the same as that applied to the model single pile, and this type of loading is meant to simulate more realistically the loading imposed on an offshore pile group.

Two series of storm loading tests (Loading pattern 1 and Loading pattern 2) were performed, which are similar to those which were performed on a single pile and for which results were presented in Chapter 6. The results of the first series, on the 2-pile group, are summarized in Table 7.10, while the results of the second series (on the 4-pile group) are summarized in Table 7.11. Each Table shows the maximum recorded loading, maximum accumulated displacement, and the number of
cycles to cause failure.

It appears from the pile group tests, that the behaviour of the pile group is similar to that of the single pile. The behaviour of a single pile subjected to one-way storm loading has been presented in Chapter 6. In this chapter, the typical behaviour of the 4-pile group subjected to two-way storm loading from test (2PTS–2) is demonstrated in Figures 7.38, 7.39, and 7.40.

Figures 7.38 and Figure 7.39 show the storm load against number of cycles, and the accumulated displacement versus number of cycles relationships respectively. The storm load versus accumulated displacement curves for cyclic and post cyclic loadings are plotted in Figure 7.40. It can be seen that the accumulated displacement increases with number of cycles. For a particular load parcel, the change in displacement in the first cycle is higher than that in subsequent cycles. However, the maximum accumulated displacement increases as the maximum load level increases.

For comparison purposes, the accumulated displacement plotted versus number of cycles, for a single pile (Test ST1), a 2-pile group (Test 2PST–3) and a 4-pile group (Test 4PST–1) subjected to one-way storm loading (Loading pattern 1) can be seen in Figure 7.41 (a and b). The differences in the accumulated displacement among the three types of group can not be recognized in the first load parcel (low load level), but these differences increase with number of cycles until they become more clear in the cycles of the last parcels. The accumulated displacement for the single pile is slightly less than that of the 2-pile group which is in turn slightly less than the 4-pile group.

The results therefore suggest that the accumulated displacement for pile group is greater than that for a single pile.
Figure 7.42 shows the behaviour of the three types of group from three different tests (single pile, 2-pile group and 4-pile group) subjected to storm loading pattern 2. The maximum load for the first few parcels increases gradually until reaching the maximum value then the maximum load decreases for the last two parcels (Figure 7.42a). Figure 7.42b shows that the general trend of accumulated displacement versus number of cycles for the first four parcels is similar to that shown in Figure 7.41b. However, the rate of increase of accumulated displacement becomes very slow after the 4th load parcel until it becomes more or less constant at the last load parcel. This reveals that the last three parcels, in which the load levels were reduced, have very little influence on the accumulated displacement of the piles.

7.7 INTERACTION TESTS

Numerous static and cyclic laboratory and field tests on piles in calcareous sediments have been undertaken in the last decade under different boundary test conditions (i.e. Angemeer et al, 1975; Nauroy and Le Tirant, 1983; Poulos and Lee, 1988; Randolph et al, 1988a; and Poulos and Al-Douri, 1992) to reach a better understanding of the behaviour of offshore piles in calcareous sand. However, the previous experimental work has not investigated the behaviour of pile groups and the interaction between piles in calcareous sands.

This section presents the observed results of some tests on an unloaded (previously jacked) pile during jacking of other piles, and the effects of static and cyclic loading of an adjacent pile at different distances from the unloaded pile. In all the tests, the instrumented pile was used for the unloaded pile and an uninstrumented pile was
used for the loaded pile. The following observations have been made:

a) The jacking force required to install the piles in different sand densities.

b) The pile head movement of the unloaded pile during jacking, static loading and cyclic loading of an adjacent pile at different spacings.

c) The stress along the unloaded instrumented pile during jacking and loading of another pile at different spacings.

The head deflection of the unloaded pile was determined by means of a deflection transducer which was positioned on the pile head. Only the axial deflection was recorded.

7.7.1 Results of Interaction Tests

A total of 12 jacking tests were carried out on model piles in different densities of calcareous sand under two different overburden pressures. The first 4 tests involved the jacking of four piles to observe the deflection of the first pile during jacking of the three other piles as shown in Table 7.2. The last 8 tests were performed on 2 piles jacked at different spacings to study the interaction between two piles during jacking, and static and cyclic loading (see Table 7.12). The location of the first unloaded pile was at a distance 3d (d = pile diameter) from the centre of the vessel for all tests, as shown in Figure 7.43a. The centre to centre spacings between two piles is shown in Figure 7.43b, the minimum spacing being 3d. This spacing could not be reduced because of the present design of the test vessel and model pile.

a) Interaction During Jacking

The results for the jacking force of four piles jacked in different sand densities
under two different overburden pressures have been discussed in Section 7.3.1 (Figures 7.2 and 7.3). The ratio of the jacking force between two consecutive piles (ie. 1st to 2nd pile, 2nd to 3rd pile and 3rd to 4th pile) appeared to be dependent on the density and the rate of increasing ratio became more pronounced with increasing sand density.

The evidence for the existence of interaction between these four piles which were jacked was via monitoring of the change in head movement of the 1st jacked pile. Figure 7.44 shows the influence of the number of jacked piles on the head movements of the unloaded pile for four tests (4PDC1, 4PDC2, 4PDC3 and 4PDC4). The overburden pressure and density for each test is shown in Table 7.2. The main features observed from Figure 7.44 are:

i) It appears from tests 4PDC-1 and 4PDC-4 that the 1st pile jacked (unloaded pile) in medium dense sand moved downward when the 2nd pile was jacked. The 1st pile continued moving downward during jacking of the 3rd and 4th piles with a generally decreasing movement rate.

ii) The overburden pressure has some effect on the movement of the unloaded pile, with a larger movement occurring for a higher overburden pressure.

iii) The first installed pile in dense sand (from test 4PDC-2 with a low overburden pressure of 100kPa) moved upward as the next pile was jacked and it continued moving upward with each successively jacked pile. Similar behaviour can be seen in this Figure for test 4PDC-3 with a higher overburden pressure 200kPa.

iv) From Figure 7.44 it can be seen that the direction of the deflection (either upward or downward) is significantly influenced by the initial sand density and the
magnitude of the deflection is influenced by overburden.

Further results from the tests on two piles jacked in sand of different densities at different spacings are shown in Figure 7.45. The "s/d" ratios were 3, 4 and 5. For the lower sand density, it can be seen that the deflection of the unloaded pile after jacking the pile at 3d spacing is greater than that at 4d and there is very little deflection of the unloaded pile after jacking the pile at 5d spacing. However, the direction of deflection has changed from downward to upward for dense sand, particularly for the case of small spacing ratio. The direction and the value of the head movement of the unloaded pile thus depends on the sand density and spacing between the piles respectively.

The load mobilized along the instrumented unloaded pile during jacking of the 2nd pile at a spacing of 3d has been measured by strain gauges in the instrumented pile. Typical results for medium dense sand subjected to 100 kPa and 200 kPa overburden pressure were obtained from two tests (Int–2 and Int–3) and are illustrated in Figures 7.46 (a and b). Both Figures show that the lower load appears at the top part of the pile near the soil surface (Sg4) as expected. However, the maximum load is developed between the middle and bottom part of the pile (Sg2), and not at the tip (Sg1) as expected. Among other things, this may be a result of lateral movement of the soil altering the load in the unloaded pile during jacking of the adjacent pile.

Figure 7.47 shows the maximum loads along the unloaded pile versus the "s/d" ratio during jacking of the 2nd pile for medium dense sand subjected to 100 kPa and 200 kPa overburden pressures. It can be seen that for both overburden pressures the maximum load decreases as the s/d ratio increases, reflecting the smaller interaction between the two piles. Also, the maximum load for the higher overburden pressure
is larger, commensurate with the higher mean stress involved.

b) Static and Cyclic loading

The effect on the unloaded pile of the loaded pile during both static and cyclic loading is shown in Figure 7.48, where the deflection of the unloaded pile is plotted against the spacing to diameter ratio "s/d" of the two piles. Similar behaviour to that observed previously from jacking is evident. The deflection of the pile head increases as the "s/d" ratio decreases for medium–dense sand. For dense sand only one test for s/d = 3 is shown, and again the deflection is now directed upward. The deflection of the unloaded pile due to static and cyclic loading may be attributed to pile–soil–pile interaction.

Figure 7.49 shows the influence of cyclic displacement on interaction between piles from two tests that were performed on medium–dense sand and subjected to 100 cycles. The pile head deflection from the test where the pile was subjected to 2.5mm cyclic displacement is higher than that for two tests involving a 1.25mm cyclic displacement. This suggests that the stress in the soil between two piles was greater in the case of the higher cyclic displacement.

Figure 7.50 shows the effect of number of cycles on the deflection of the unloaded pile from test 1nt–1 conducted on medium dense sand. The deflection increases with increasing number of cycles. The highest rate of deflection occurs in the first 20 cycles and then the rate reduces.

Typical load–deflection curves at different points along the unloaded pile shaft under both static and cyclic loading are illustrated in Figure 7.51. The mobilized static load at all the points on the pile increases for a displacement of about 1mm and then
becomes more or less constant with increasing displacement. The minimum value of static load appears at the top part of the pile and the maximum load at the lower part of the pile. From this Figure, the interface shear stress can be computed along the unloaded pile and the minimum and maximum skin friction should be at the top and bottom of the pile respectively.

It can also be seen in Figure 7.51 that a reduction in force along the unloaded pile occurs due to cycling the load on the loaded pile. It appears that the higher reduction occurred at the bottom part of the pile. This again indicates the existence of interaction between the two piles, not only during jacking and static loading, but also during cyclic loading.

7.8 SUMMARY

Several series of static and cyclic tests on model 2-pile groups and 4-pile groups jacked into New North Rankin calcareous sand have been carried out to investigate the factors that influence the load-settlement behaviour and the skin friction behaviour along piles in pile groups in this sand. The main conclusions are summarized below.

7.8.1 Behaviour During Jacking

1) The end bearing resistance and the skin friction developed during jacking generally increase as the initial sand density increases or overburden pressure increases.

2) For piles jacked individually, the jacking load behaviour of the pile group in
dense sand shows a different trend to that in medium dense sand. For medium–dense sand, the ratio of the jacking load between two consecutive piles (ie 1st to 2nd pile, 2nd to 3rd pile and 3rd to 4th pile) appears to be more or less constant and all of these load ratios were less than 1. In contrast, for dense sand, the ratio between the jacking load of two consecutively jacked piles increases until the ratio between the 3rd to 4th piles becomes greater than 1.

3) From the comparison between the results of piles jacked individually and together, it has been found that jacking the piles together may result in a more uniform distribution of load among and along these piles than if the piles are jacked individually.

4) It has been found that the residual load mobilized along one pile in dense sand is higher than that in medium dense sand. The trend of the residual load against depth for the medium–dense sand test is nearly linear, while that for dense sand is not quite linear.

5) The maximum value of developed residual load on one pile in a group in dense sand is higher than that in medium–dense sand. This behaviour is similar to that for jacking of a single pile.

7.8.2 Static Response

1) The tensile group load increases to a peak load and then drops to an ultimate value for both dense and medium–dense sands. For dense sand, the peak appears to be more pronounced and is developed at a smaller displacement than that for medium dense sand.
2) When the tensile load increases, the loads measured for all points along the pile embedded in the soil start reducing the residual compression load and increasing the tensile load.

3) It appears that the group efficiency ($\eta$) increases as the number of piles in group increases, and $\eta$ increases with increasing sand density.

4) For single piles and pile groups, the secant stiffness decreases with increasing load level.

5) It has been found that the stiffness of single piles is higher than that of the 2–pile group which again is stiffer than the 4–pile group.

6) The residual load along one pile in a group decreases with increasing group movement towards failure, until the tip load is zero at a failure displacement of about 10% pile diameter.

7) The uniformity of skin friction may be influenced by the number of jacked piles in a group.

7.8.3 Cyclic Displacement–Controlled Response

1) The skin friction of the piles tended to degrade under cyclic loading. The degradation for the pile groups is more pronounced as the sand density increases.

2) The degradation of skin friction increases as the cyclic displacement increases and as the overburden pressure increases.
3) The rate of reduction of skin friction appeared to reduce with increasing number of cycles.

4) The degradation of skin friction appears to be slightly more pronounced as the number of piles in the group increases.

7.8.4 Cyclic Load–Controlled Response

a) For Uniform Amplitude Tests:

The general behaviour of pile groups is reasonably consistent with single pile results.

1) The accumulated displacement of pile groups increases as both load level and number of cycles increase.

2) The accumulated displacement in the first cycle is markedly higher than for subsequent cycles.

3) A cyclic stability diagram for both 2–pile and 4–pile groups can be used to define the stable and unstable zones. Group effects do not appear to have a major effect on the cyclic stability diagram.

4) It appears there is some effect of number of piles in a group on the group capacity and group settlement.

b) For Storm Load Tests:

1) For each parcel of load, the accumulated displacement in the first cycle is
higher than for any subsequent cycles.

2) As observed in single pile tests, it also appears that, for a specific load parcel, the accumulated displacement increases with increasing number of cycles, but the rate of increase reduces with number of cycles.

3) The accumulated displacement increases with increasing load level. The increment of accumulated displacement appears to depend on the load level more than on the number of cycles. This is also observed in the single pile tests.

4) It appears that the small influence of the number of piles in the group on the group capacity and group settlement in the case of uniform amplitude cyclic load is also repeated for the storm loading case.

7.8.5 Interaction Tests

a) During Jacking

1) The head deflection of the unloaded (1st jacked) pile increases with increasing numbers of subsequently jacked piles, and the deflection also increases as the relative spacing "s/d" decreases. This clearly demonstrates the interaction between piles during installation of the piles.

2) The direction of the deflection depends on the sand density, upward in dense sand or downward in loose sand, and the magnitude of the deflection is influenced by overburden pressure.
b) During Static and Cyclic Loading

1) The head deflection of the unloaded pile depends on spacing between the piles. The deflection increases with decreasing spacing between two piles. This may be attributed to the effects of interaction between the piles through the soil (i.e. pile-soil-pile interaction).

2) The influence of the sand density on the direction of deflection of the unloaded pile, and the influence of overburden pressure on the magnitude of the deflection, are similar to those observed during jacking.

3) The minimum value of induced load in a pile due to loading of adjacent piles appears at the top part of the pile and the maximum load occurs in the lower part of the pile.

4) The force along the unloaded pile reduces due to cycling the load on the loaded pile. It appears that the greater reduction occurs at the bottom part of the pile. This indicates the existence of pile-soil-pile interaction during cyclic loading, as well as during jacking and static loading.
Table 7.1 Summary of Pile Group Test Programme

Pile Group Tests
(2-Pile & 4-pile Group)

- Jacking Tests
- Static Tests
- Cyclic Tests

- Jacking Piles
- Disp.- Controlled Tests
- Load- Controlled Tests
- Load- Controlled Tests
- Individually in Group
- Together Tests
- Uniform Amplitude
- Non-Uniform Amplitude
- (Storm Load)
### Table 7.2: Results from Installation of Pile Groups (By Jacking the Piles Individually)

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Dry Density (kN/m³)</th>
<th>Vertical Pressure (kPa)</th>
<th>Jacked Pile No.</th>
<th>Embedded Length L (mm)</th>
<th>Residual Load at Base Prior to Test SG1 (N)</th>
<th>Peak Base Load</th>
<th>Peak End Bearing Capacity</th>
<th>Peak Head Load</th>
<th>Peak Skin Friction During Jacking (kPa)</th>
<th>Ultimate Base Load</th>
<th>Ult. End Bearing Capacity</th>
<th>Ultimate Head Load</th>
<th>Ultimate Skin Friction</th>
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<td>1st</td>
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<td>85.5</td>
<td>1254.0</td>
<td>2554.6</td>
<td>1360.0</td>
<td>1520.0</td>
<td>1280.0</td>
<td>2607.6</td>
<td>1445.0</td>
<td>7.3</td>
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<td></td>
<td></td>
<td>1571.0</td>
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<td>Penetration at Peak Load Z (mm)</td>
<td>Total Jacking Load * P.R (N)</td>
<td>Mean Jacking Load * (P.R)/4 (N)</td>
<td>Peak Head Load Sg5 (N)</td>
<td>Peak Base Load Sg1 (N)</td>
<td>Peak End Bearing Capacity (kPa)</td>
<td>Peak Skin Friction During Jacking (kPa) Sg5 (N)</td>
<td>Ultimate Head Load Sg5 (N)</td>
<td>Ultimate Base Load SG1 (N)</td>
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</table>

* P.R = Proving ring reading for total load on pile group during jacking
* P.R/4 = Reading of total load on jacked group / No. of piles
<table>
<thead>
<tr>
<th>Test No.</th>
<th>Dry Density (kN/m³)</th>
<th>Vertical Pressure (kPa)</th>
<th>Embedded Length L (mm)</th>
<th>Residual Load at Base Prior to Test SG1 (N)</th>
<th>Penetration at Peak Load Z (mm)</th>
<th>Total Jacking Load * P.R (N)</th>
<th>* P.R/2 (N)</th>
<th>Mean Jacking Load</th>
<th>Peak Head Load Sg5 (N)</th>
<th>Sg1 (N)</th>
<th>Peak End Bearing Capacity (kPa)</th>
<th>Peak Skin Friction During Jacking</th>
<th>Ultimate Head Load Sg5 (N)</th>
<th>Ultimate Base Load SG1 (N)</th>
<th>Ultimate Skin Friction (kPa)</th>
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P.R = Proving ring reading for total load on pile group during jacking
P.R/2 = Reading of total load on jacked group / No. of piles
Table 7.5a  Results from Displacement Controlled Tests on 4-Pile Group (Piles Jacked Individually)

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Dry Density (kN/m³)</th>
<th>Vertical Pressure (kPa)</th>
<th>Total Static Load * P.R (N)</th>
<th>Mean Static Load * (P.R)/4 (N)</th>
<th>Head Load Sg5 (N)</th>
<th>Static Skin Friction fs (kPa)</th>
<th>Secant Soil Modulus Ec50 (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4PDC–1</td>
<td>10.40</td>
<td>100</td>
<td>1415.0</td>
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<td>295.0</td>
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<td>13.0</td>
</tr>
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<td>100</td>
<td>1900.0</td>
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<td>383.0</td>
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</tr>
<tr>
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<td>200</td>
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<tr>
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<td>548.0</td>
<td>486.0</td>
<td>24.9</td>
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*P.R = Proving ring reading for total load on pile group during jacking
P.R/4 = Reading of total static load on jacked group / No. of piles
### Table 7.5b
Results from Displacement Controlled Tests on 4-Pile Group (Piles Jacked Individually)

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Cyclic Load (kN)</th>
<th>Cyclic Disp. (mm)</th>
<th>Vertical Pressure (kPa)</th>
<th>Cyclic Skin Friction (kPa)</th>
<th>Degradation Factor</th>
<th>f_c/f_s</th>
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<tbody>
<tr>
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<td>100</td>
<td>50</td>
<td>0.62</td>
<td>9.9</td>
</tr>
<tr>
<td>4PDC-2</td>
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<td>50</td>
<td>100</td>
<td>50</td>
<td>0.46</td>
<td>10.0</td>
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<td>50</td>
<td>200</td>
<td>50</td>
<td>0.54</td>
<td>17.1</td>
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<tr>
<td>4PDC-4</td>
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<td>50</td>
<td>200</td>
<td>50</td>
<td>0.43</td>
<td>10.7</td>
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Table 7.6a  Results from Displacement Controlled Tests on 4–Pile Group  
(All Piles in the Group Jacked Together)

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Dry Density (kN/m³)</th>
<th>Vertical Pressure (kPa)</th>
<th>Total Static Load</th>
<th>Mean Static Load</th>
<th>Head Load</th>
<th>Static Skin Friction (kPa)</th>
<th>Secant Soil Modulus Ec50 (MPa)</th>
<th>Secant Pile Stiffness (N/mm)</th>
<th>Group Efficiency (Pg / NPs)</th>
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<td>11.5</td>
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<td>18.9</td>
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* P.R = Proving ring reading for total load on pile group during jacking.

P.R/4 = Reading of total static load on jacked group / No. of piles
### Table 7.6b  Results from Displacement Controlled Tests on 4-Pile Group
(All Piles in the Group Jacked Together)

<table>
<thead>
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<th>Test No.</th>
<th>Dry Density (kN/m³)</th>
<th>Vertical Pressure (kPa)</th>
<th>Cyclic Disp. (mm)</th>
<th>No. of Cycles</th>
<th>Cyclic Load Pc (N)</th>
<th>Cyclic Skin Friction fc (kPa)</th>
<th>Degradation Factor fc/ls</th>
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<td>117.0</td>
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<tr>
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Table 7.7a  Results from Displacement Controlled Tests on 2–Pile Group
(All Piles in the Group Jacked Together)

<table>
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<tr>
<th>Test No.</th>
<th>Dry Density (kN/m³)</th>
<th>Vertical Pressure (kPa)</th>
<th>Total Static Load * P.R (N)</th>
<th>Mean Static Load * (P.R)/2 (N)</th>
<th>Head Load (N)</th>
<th>Static Skin Friction (kPa)</th>
<th>Secant Soil Modulus Ec50 (MPa)</th>
<th>Secant Pile Stiffness (N/mm)</th>
<th>Group Efficiency (Pg / NPs)</th>
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<td>3742.8</td>
<td>1.00</td>
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</tbody>
</table>

* P.R = Reading of total static load of pile group from proving ring
P.R/2 = Reading of total static load on pile group / No. of Piles
Table 7.7b  Results from Displacement Controlled Tests on 2–Pile Group
(All Piles in the Group Jacked Together)

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Dry Density (kN/m²)</th>
<th>Vertical Pressure (kPa)</th>
<th>Cyclic Disp. (mm)</th>
<th>No. of Cycles</th>
<th>Cyclic Load (N)</th>
<th>Cyclic Skin Friction (kPa)</th>
<th>Degradation Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>2PDC–1</td>
<td>10.10</td>
<td>75</td>
<td>2.50</td>
<td>50</td>
<td>84.0</td>
<td>3.7</td>
<td>0.52</td>
</tr>
<tr>
<td>2PDC–2</td>
<td>10.20</td>
<td>100</td>
<td>0.50</td>
<td>50</td>
<td>140.0</td>
<td>6.2</td>
<td>0.66</td>
</tr>
<tr>
<td>2PDC–3</td>
<td>9.90</td>
<td>100</td>
<td>1.10</td>
<td>50</td>
<td>127.7</td>
<td>5.6</td>
<td>0.60</td>
</tr>
<tr>
<td>2PDC–4</td>
<td>10.50</td>
<td>100</td>
<td>1.10</td>
<td>50</td>
<td>109.4</td>
<td>4.9</td>
<td>0.44</td>
</tr>
<tr>
<td>2PDC–5</td>
<td>10.20</td>
<td>200</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>2PDC–6</td>
<td>10.30</td>
<td>200</td>
<td>2.50</td>
<td>100</td>
<td>128.0</td>
<td>5.7</td>
<td>0.28</td>
</tr>
<tr>
<td>2PDC–7</td>
<td>10.00</td>
<td>100</td>
<td>1.10</td>
<td>50</td>
<td>102.6</td>
<td>4.7</td>
<td>0.53</td>
</tr>
</tbody>
</table>
Table 7.8  Summary of Two Pile Group Results from Load—Controlled Tests

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Density kN/cu.m</th>
<th>Overburden Pressure kPa</th>
<th>Pm/Ps</th>
<th>Pc/Ps</th>
<th>Number of Cycles to Cause Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>2PCL1</td>
<td>10.2</td>
<td>100</td>
<td>0.29</td>
<td>0.29</td>
<td>–</td>
</tr>
<tr>
<td>2PCL2</td>
<td>10.1</td>
<td>100</td>
<td>0.35</td>
<td>0.35</td>
<td>–</td>
</tr>
<tr>
<td>2PCL3</td>
<td>10.6</td>
<td>100</td>
<td>0.00</td>
<td>0.85</td>
<td>148</td>
</tr>
<tr>
<td>2PCL4</td>
<td>10.4</td>
<td>100</td>
<td>0.38</td>
<td>0.42</td>
<td>140</td>
</tr>
<tr>
<td>2PCL5</td>
<td>10</td>
<td>100</td>
<td>0.37</td>
<td>0.25</td>
<td>–</td>
</tr>
<tr>
<td>2PCL6</td>
<td>9.9</td>
<td>100</td>
<td>0.57</td>
<td>0.00</td>
<td>–</td>
</tr>
<tr>
<td>2PCL7</td>
<td>10.5</td>
<td>100</td>
<td>0.15</td>
<td>0.54</td>
<td>–</td>
</tr>
<tr>
<td>2PCL8</td>
<td>10.3</td>
<td>100</td>
<td>0.67</td>
<td>0.24</td>
<td>156</td>
</tr>
<tr>
<td>2PCL9</td>
<td>10</td>
<td>100</td>
<td>0.21</td>
<td>0.27</td>
<td>–</td>
</tr>
<tr>
<td>2PCL10</td>
<td>10.2</td>
<td>100</td>
<td>0.70</td>
<td>0.10</td>
<td>–</td>
</tr>
</tbody>
</table>

Table 7.9  Summary of Four Pile Group Results from Load—Controlled Tests

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Density kN/cu.m</th>
<th>Overburden Pressure kPa</th>
<th>Pm/Ps</th>
<th>Pc/Ps</th>
<th>Number of Cycles to Cause Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>4PCL1</td>
<td>9.9</td>
<td>100</td>
<td>0.35</td>
<td>0.35</td>
<td>–</td>
</tr>
<tr>
<td>4PCL2</td>
<td>10</td>
<td>100</td>
<td>0.20</td>
<td>0.44</td>
<td>–</td>
</tr>
<tr>
<td>4PCL3</td>
<td>10.6</td>
<td>100</td>
<td>0.46</td>
<td>0.46</td>
<td>135</td>
</tr>
<tr>
<td>4PCL4</td>
<td>10.4</td>
<td>100</td>
<td>0.28</td>
<td>0.65</td>
<td>150</td>
</tr>
<tr>
<td>4PCL5</td>
<td>10.2</td>
<td>100</td>
<td>0.39</td>
<td>0.39</td>
<td>–</td>
</tr>
<tr>
<td>4PCL6</td>
<td>10.4</td>
<td>100</td>
<td>0.14</td>
<td>0.69</td>
<td>–</td>
</tr>
<tr>
<td>4PCL7</td>
<td>9.9</td>
<td>100</td>
<td>0.70</td>
<td>0.14</td>
<td>–</td>
</tr>
<tr>
<td>4PCL8</td>
<td>10.2</td>
<td>100</td>
<td>0.70</td>
<td>0.10</td>
<td>–</td>
</tr>
</tbody>
</table>
### Table 7.10 Summary of 2-Pile Group Results from Storm Load Tests

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Pattern</th>
<th>Number of Parcels Per Pattern</th>
<th>Maximum Load</th>
<th>Number of Cycles to Cause Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>2PST1</td>
<td>1st</td>
<td>4</td>
<td>0.5Ps</td>
<td>–</td>
</tr>
<tr>
<td>2PST2</td>
<td>1st</td>
<td>4</td>
<td>0.65Ps</td>
<td>260</td>
</tr>
<tr>
<td>2PST3</td>
<td>2nd</td>
<td>6</td>
<td>0.40Ps</td>
<td>–</td>
</tr>
<tr>
<td>2PST4</td>
<td>2nd</td>
<td>6</td>
<td>0.45Ps</td>
<td>–</td>
</tr>
</tbody>
</table>

### Table 7.11 Summary of 4-Pile Group Results from Storm Load Tests

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Pattern</th>
<th>Number of Parcels Per Pattern</th>
<th>Maximum Load</th>
<th>Number of Cycles to Cause Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>4PST1</td>
<td>1st</td>
<td>4</td>
<td>0.5Ps</td>
<td>–</td>
</tr>
<tr>
<td>4PST2</td>
<td>1st</td>
<td>4</td>
<td>0.75Ps</td>
<td>320</td>
</tr>
<tr>
<td>4PST3</td>
<td>1st</td>
<td>4</td>
<td>0.65Ps</td>
<td>–</td>
</tr>
<tr>
<td>4PST5</td>
<td>2nd</td>
<td>6</td>
<td>0.45Ps</td>
<td>–</td>
</tr>
<tr>
<td>4PST6*</td>
<td>2nd</td>
<td>5</td>
<td>0.70Ps</td>
<td>–</td>
</tr>
</tbody>
</table>

*2-way Cyclic Loading*
Table 7.12 Results from Interaction Tests Between Two Piles

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Dry Density (kN/m³)</th>
<th>Cyclic Displac. (mm)</th>
<th>Over-burden Pressure (kPa)</th>
<th>Number of Cycles</th>
<th>Spacing Between Two Piles (mm)</th>
<th>Vertical Deflection During Jacking (mm)</th>
<th>Vertical Deflection During Cyclic Loading (mm)</th>
<th>Max. Load Along Unloaded Pile from Jacking (N)</th>
<th>Max. Load Along Unloaded Pile from Testing (N)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Int-1</td>
<td>10.20</td>
<td>1.25</td>
<td>100</td>
<td>100</td>
<td>3d</td>
<td>-0.23</td>
<td>-0.25</td>
<td>120.0</td>
<td>40.0</td>
</tr>
<tr>
<td>Int-2</td>
<td>10.40</td>
<td>2.50</td>
<td>100</td>
<td>82</td>
<td>3d</td>
<td>-0.29</td>
<td>-0.36</td>
<td>116.0</td>
<td>51.0</td>
</tr>
<tr>
<td>Int-3</td>
<td>10.10</td>
<td>1.25</td>
<td>200</td>
<td>100</td>
<td>3d</td>
<td>-0.42</td>
<td>-0.30</td>
<td>174.0</td>
<td>62.0</td>
</tr>
<tr>
<td>Int-4</td>
<td>11.50</td>
<td>2.50</td>
<td>100</td>
<td>100</td>
<td>3d</td>
<td>0.12</td>
<td>0.11</td>
<td>140.0</td>
<td>-</td>
</tr>
<tr>
<td>Int-5</td>
<td>11.40</td>
<td>2.50</td>
<td>200</td>
<td>100</td>
<td>3d</td>
<td>0.14</td>
<td>0.33</td>
<td>234.0</td>
<td>98.0</td>
</tr>
<tr>
<td>Int-6</td>
<td>10.00</td>
<td>2.50</td>
<td>200</td>
<td>100</td>
<td>4d</td>
<td>-0.18</td>
<td>-0.19</td>
<td>145.0</td>
<td>67.0</td>
</tr>
<tr>
<td>Int-7</td>
<td>11.30</td>
<td>1.25</td>
<td>100</td>
<td>100</td>
<td>4d</td>
<td>0.06</td>
<td>-0.14</td>
<td>54.0</td>
<td>-</td>
</tr>
<tr>
<td>Int-8</td>
<td>10.40</td>
<td>2.50</td>
<td>200</td>
<td>100</td>
<td>5d</td>
<td>-0.08</td>
<td>-0.09</td>
<td>49.0</td>
<td>-</td>
</tr>
</tbody>
</table>

*  

\(d =\) Pile diameter
Note

\( d = \text{Pile Diameter} \)

\( \Theta_1 = \text{1st Pile Jacked (Monitored Pile)} \)

\( \Theta_2 = \text{2nd Pile Jacked} \)

\( \Theta_3 = \text{3rd Pile Jacked} \)

\( \Theta_4 = \text{4th Pile Jacked} \)

Figure 7.1 Configuration of Four Jacked Piles
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Figure 7.2b Number of Piles vs Jacking Load (Dense Sand)
Figure 7.3 Mean Pile Load During Jacking (2-Pile Group)

Figure 7.4 Mean Pile Load During Jacking (4-Pile Group)
Medium—Dense Sand
Overburden Pressure = 100kPa

![Graph showing the jacking load vs. penetration for different pile configurations.

- Single Pile
- 2—Piles in Group
- 4—Piles in Group

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Figure 7.9  The Influence of Overburden Pressure on Peak Skin Friction of Pile Groups During Jacking
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Figure 7.33 Effect of Number of Piles in the Group on Degradation Factor
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Case 1

Case 2

Case 3

Note

d = Pile Diameter

○1 = 1st Pile jacked (Unloaded Pile)

○2 = 2nd Pile jacked (Loaded Pile)

Figure 7.43b The Spacings Between The Centres of Two Piles
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Figure 7.50 Number of Cycles Versus Deflection of Pile Head
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Chapter 8 COMPARISINS OF THEORETICAL AND EXPERIMENTAL BEHAVIOUR OF PILES IN CALCAREOUS SAND

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8.2 THEORETICAL ANALYSIS
   8.2.1 Static Analysis
   8.2.2 Cyclic Response Analysis

8.3 PILE GROUP

8.4 PROGRAM "SCARP"
   8.4.1 Program SCARP Capabilities

8.5 DETERMINATION OF PARAMETERS REQUIRED FOR ANALYSIS

8.6 COMPARISONS BETWEEN ANALYSIS AND MEASURED RESULTS
   8.6.1 Pile Installation
   8.6.2 Static Behaviour
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8.7 SUMMARY AND CONCLUSIONS
Chapter 8

COMPARISONS BETWEEN THEORETICAL AND EXPERIMENTAL BEHAVIOUR OF PILES IN CALCAREOUS SAND

8.1 INTRODUCTION

This chapter presents two aspects of the behaviour of piles in calcareous sediments. The first deals with a numerical analysis and how it was used to predict the behaviour of an isolated pile or a pile within a group. The second aspect presents some comparisons between the measured and theoretical performance of single piles and pile groups in calcareous sand under both static and cyclic loading.

A numerical analysis of a single pile and a pile group is described. The extension of the static loading analysis to consider cyclic loading is also described. The theoretical results are obtained using a computer program "SCARP" developed by Poulos (1990). The objective of the theoretical analysis was: i) to examine the behaviour of a single pile or a pile within a group by using input parameters derived from the experimental results, and ii) to investigate how and why the predicted results either fit or do not agree with experimental results during all stages of the tests, from jacking to the end of cyclic loading.
A simple modification has been performed on the "SCARP" analysis to simulate the accumulation of permanent displacement under uniform and non-uniform amplitude cyclic loading, and is described herein.

The comparisons include the model pile and pile group tests described in Chapters 6 and 7 respectively, together with a number of published test results.

8.2 THEORETICAL ANALYSIS

A simple boundary element analysis for both static and cyclic axial response is described and has been used to predict the tensile loads along the pile, and the soil displacement which arises from pile-soil interaction. This analysis is of a single pile, but is also applicable to groups of equally loaded piles. The following assumptions are made in this analysis:

1) the pile is circular and behaves as a uniaxially loaded elastic column with a closed end;
2) each element is acted upon by a uniform vertical pile-soil stress;
3) only compatibility of vertical displacements of the soil and pile are considered;
4) the applied load is static or quasi-static, i.e. inertia effects are not considered.

8.2.1 Static Analysis

The analysis for static response is described by Poulos (1990). A boundary element analysis is used in which the pile is represented as an elastic cylinder and the surrounding soil mass as an elastic continuum. The Poisson's ratio ($\nu_s$) and Young's modulus ($E_s$) of the soil are here assumed constant along the pile. The geometry of
a pile and the surrounding soil medium, and the representation of the pile as a series of elements are shown in Figure 8.1. The pile can be divided into three different types of elements:

i) cylindrical shaft elements,
ii) annular base elements,
iii) annular elements at discontinuities between shaft elements.

Consideration is given to compatibility between the displacements of the pile and the soil at each element, but allowance is also made for the possibility of pile–soil slip or soil yield by specifying limiting values of the interaction stress at each element. The analysis is formulated incrementally so that the complete sequence of loading, unloading and reloading can be simulated. Consequently, the residual displacements and stresses upon unloading to zero load may also be computed.

a) Incremental Soil Displacement Equations

Considering a typical element i shown in Figure 8.1, the incremental axial soil displacement $\Delta \rho_{\text{sij}}$ adjacent to element i due to the incremental stress $\Delta \rho_j$ on an element j can be expressed as:

$$\Delta \rho_{\text{sij}} = \frac{I_{ij}}{E_S} \Delta \rho_j$$  \hspace{1cm} (8.1)

where:

$I_{ij} =$ axial–displacement influence factor for element i due to shear stress at element j.

As a result of loads at all n side elements and the base, the incremental axial soil
displacement at element \( i \) is:

\[
\Delta \rho_{si} = \frac{1}{E_s} \sum_{j=1}^{n} I_{ij} \Delta p_j + \frac{1}{E_s} I_{ib} \Delta p_b
\] (8.2)

where:

\( I_{ib} \) = axial displacement factor for element \( i \) due to uniform stress on the base

\( \Delta p_b \) = incremental stress at the base element.

The incremental axial soil displacements for the base and for all the pile elements can be expressed more generally as:

\[
(\Delta \rho_s) = \left[ \frac{1}{E_s} \right] (\Delta p)
\] (8.3)

where:

\( (\Delta \rho_s) \) = incremental axial soil displacement vector

\( (\Delta p) \) = incremental pile-soil interaction stress vector

\[
\left[ \frac{1}{E_s} \right] = \text{matrix of soil influence factors divided by the soil modulus, and the influence factors are most conveniently evaluated by integration of Mindlin's equation (Mindlin, 1936) for the displacement caused by a point load within a semi-infinite mass. Details of the relevant integration are given in Poulos and Davis (1980).}

b) Incremental Pile Displacement Equations

Considering a typical shaft element \( i \) as shown in Figure 8.1, the incremental axial compression \( \Delta \rho_i \) is given as:
\[ \Delta \rho_i = \frac{\Delta P_i}{E_{pi}} \frac{l_i}{A_{pi}} \tag{8.4} \]

where:

- \( \Delta P_i \) = incremental average load in pile element \( i \)
- \( l_i \) = length of pile element \( i \)
- \( E_{pi} \) = Young's modulus of pile element \( i \)
- \( A_{pi} \) = cross-sectional area of pile element \( i \).

The incremental compression vector \((\Delta \rho)\) for the pile shaft and base element can be expressed as:

\[ (\Delta \rho) = [FE] (\Delta p) \tag{8.5} \]

where:

- \([FE]\) = pile compression matrix (See Poulos, 1990, for details).

The total incremental axial displacement vector for all the pile elements is given as:

\[ (\Delta \rho_p) = \Delta \rho_b \{1\} + [AD] (\Delta \rho) \tag{8.6} \]

where:

- \( \Delta \rho_b \) = incremental pile base displacement
- \( \{1\} \) = vector whose elements are unity
- \([AD]\) = summation matrix (see Poulos, 1990, for details).

By combining eqs. (8.5) and (8.6), the incremental axial displacement of the pile elements may be expressed as follows:

\[ (\Delta \rho_p) = \Delta \rho_b \{1\} + [AD][FE] (\Delta p) \tag{8.7} \]
c) Displacement Compatibility Condition

While purely elastic conditions exist at the pile–soil interface, the axial displacement of adjacent points along the interface must be equal; thus,

\[(\Delta \rho_s) = (\Delta \rho_p)\]  \hspace{1cm} (8.8)

Substituting eqs. (8.3) and (8.7) into eq. (8.8), gives:

\[
\left[ \frac{1}{E_s} - AD.FE \right] (\Delta p) = \Delta \rho_B \hspace{1cm} (8.9)
\]

d) Vertical Equilibrium Condition

The vertical force equilibrium condition requires:

\[
\sum_{i=1}^{n} A_i \Delta \rho_i = \Delta P \hspace{1cm} (8.10)
\]

where:

- \(A_i\) = surface area of pile–soil interface for element \(i\)
- \(\Delta \rho_i\) = interaction stress increment on element \(i\)
- \(n\) = total number of elements
- \(\Delta P\) = increment of applied load on pile head.

Equations (8.9) and (8.10) may be solved for the unknown interaction stress increments \(\Delta p\) and the incremental base displacement, \(\Delta \rho_B\). Then equation 8.7 can
be used to compute incremental pile displacements. Consequently, the current overall
pile displacements and interaction stresses are determined by adding the incremental
values to the existing values computed at the previous load level.

8.2.2 Cyclic Response Analysis

The analysis for cyclic axial response is described by Poulos (1990). This analysis
considers cyclic degradation of skin friction and accumulation of permanent
displacements.

a) Cyclic Degradation of Skin Friction

This is assumed to depend upon the amplitude of cyclic displacement and the
number of cycles. The limiting pile shaft resistance \( \tau_{fc} \) after cycling can be
expressed as:

\[
\tau_{fc} = D_\tau \tau_f
\]  

(8.11)

where,

\( \tau_f \) = limiting pile shaft resistance for static loading

\( D_\tau \) = cyclic degradation factor for shaft resistance

The determination of the degradation factor for skin friction, \( D_\tau \), is discussed below.

Reverse-Plastic-Stress (Matlock and Foo, 1980) Model

This model postulates that cyclic degradation of the limiting shear stress occurs over
an element on the shaft when that element is subjected to reverse slip. Each
subsequent application of reverse slip causes additional degradation until some lower limiting value of skin friction is reached.

This model can be expressed as follows:

\[ D_τ = (D_τ' - D_{τ_{\text{min}}})(1 - λ) + D_{τ_{\text{min}}} \]  

(8.12)

where

- \( D_τ \) = current value of degradation factor
- \( D_τ' \) = degradation factor for previous cycle
- \( D_{τ_{\text{min}}} \) = minimum possible degradation factor
- \( λ \) = degradation rate parameter.

The merit of this model is that it involves only two parameters, \( λ \) and \( D_{τ_{\text{min}}} \), which are affected by soil type and the method of installation of the piles. However, comparisons by Lee (1988) show that the model is over-simplified and does not always accurately reflect the rate at which the skin friction degrades with increasing numbers of cycles.

b) Accumulation of Permanent Displacements

Accumulation of permanent displacement under cyclic loading depends on the cyclic load level and the number of cycles. As a means of representing the accumulated displacements, the incremental displacement of soil adjacent to each pile element can be considered to consist of two components (Poulos, 1989b) which are:

1) Due to the incremental pile–soil stresses on soil elements;
2) Due to the accumulation of permanent displacement caused by cyclic loading.
Hence equation (8.3) can be modified as follows:

\[(\Delta \rho_s) = \left[ \frac{1}{E_s} \right] (\Delta p) + \{\Delta S_p\} \quad (8.13)\]

where,

\[\{\Delta S_p\} = \text{Vector of incremental permanent soil displacements due to cyclic loading.}\]

Thus, equation (8.9) be modified as follows:

\[\left[ \frac{1}{E_s} - \text{AD. FE} \right] (\Delta p) = \Delta \rho_b \left( 1 - \{\Delta S_p\} \right) \quad (8.15)\]

The incremental permanent displacement, \(\Delta S_p\), occurring between cycles \(N\) and \(N+\Delta N\), can be approximated by the equation of Diyaljee and Raymond (1982) which was originally based upon the results of triaxial tests:

\[S_p = B N^m e^{nX} \quad (8.16)\]

where:

\[X = \text{applied cyclic load level (a function of mean load and cyclic load amplitude)}\]

\[N = \text{number of cycles}\]

\[B,n,m = \text{experimentally-determined parameters}\]

\[S_p = B \text{ for } N = 1 \text{ and } X \to 0, \text{ and } n \text{ can be derived from the variation of } S_p \text{ with load level } X.\]

Then the increment of permanent displacement, \(\Delta S_p\), can be expressed as:
\[ \Delta S_p = S_{pN} (n\Delta X + m\Delta N/N) \]  

(8.17)

where

\[ S_{pN} = \text{permanent displacement at cycle N} \]
\[ \Delta X = \text{change in representative stress level X between N and N+\Delta N} \]
\[ n,m = \text{experimentally-determined dimensionless parameters.} \]

The program "SCARP" uses the above method of analysis and incorporates Eq. 8.17 into Eq. 8.15. It also allows for the consideration of pile-soil slip once the interaction shear stress has reached the current value of limiting skin friction (Poulos and Davis, 1980).

### 8.3 PILE GROUP

The analysis of a single pile can be extended to the analysis of the behaviour of piles in a group in two cases. The first is when the piles in the group are distributed in a number of symmetrical annuli, as is the case for an offshore pile group. The second is when the number of piles in the group is small (i.e. 2 piles or 4 piles in the group), as is the case for the experimental tests which were performed in this study. In order to accomplish a similar solution, the analysis considers the behaviour of all the piles in the group to be identical, under both static and cyclic loadings.

The simple modification required for the analysis of the behaviour of piles in a group is to include components due to the surrounding piles in the group in addition to the subject pile itself. These additions are made to the calculation of the soil displacements at each element as described earlier in this chapter and expressed by
(eq. 8.7). Solution of the resulting displacement compatibility equations, the checks
for pile–soil slip and yield and incorporation of cyclic degradation, are then carried
out as for a single pile.

8.4 PROGRAM "SCARP"

The program "SCARP" can be used to compute the axial, static and cyclic
displacement and load distribution along a pile subjected to specified sequences of
static and/or cyclic loading.

Static Loading can be specified as either displacement–controlled or load–controlled.
For cyclic loading, load–controlled loading can be specified, and one can consider
both uniform amplitude controlled load and non–uniform amplitude (storm) load, in
which each loading set involves a specified number of cycles with specified values of
mean and cyclic load.

The program uses the simplified boundary element formulation described above, with
the soil being represented by an elastic continuum. Allowance is made for pile soil
slip along the interface and end bearing failure by specifying limit values of the
pile–soil interaction stresses.

In Applying eq. 8.16 for storm loading, in which the load level "X" changes or
several parcels of cyclic loading are applied, care should be taken to determine the
appropriate value of the number of cycles N. An equivalent number of cycles "N_{ke}"
of the new load level X_{k+1} should be determined to represent all previous cycles of
load at other stress levels. This is given by:
\[ N_{ke} = \sum_{j=1}^{k} N_j e^{\alpha(X_j - X_{k+1})} \]  \hspace{1cm} (8.18)

where,

- \( N_j \) = number of cycles of cyclic load "parcel" j
- \( X_j \) = load level for parcel, j assumed here to be given by \((P_{oj} + 0.5P_{cj})/P_u\)
- \( P_{oj} \) = mean load for parcel j
- \( P_{cj} \) = cyclic load (half-amplitude) for parcel j
- \( P_u \) = peak static pile capacity
- \( X_{k+1} \) = load level for current cyclic loading parcel \((k+1)\)
- \( \alpha = n/m \) = ratio of empirical permanent displacement parameters.

Eq. 8.17 can be written and applied most conveniently in incremental form, as follows:

\[ \Delta S_p = S_{pN} \left( \frac{m \Delta N}{N_{ke}} \right) \] \hspace{1cm} (8.19)

where,

- \( \Delta S_p \) = increment in permanent displacement between cycles \( N_{ke} \) and \( N_{ke} + \Delta N \) (at the current load level \( X_{k+1} \))
- \( S_{pN} \) = permanent displacement of the pile at cycle \( N_{ke} \).

In the "SCARP" analysis described by Poulos (1990), the pile displacement for the first cycle of loading has been computed via elastic-plastic theory, using the input values of Young's modulus of the soil and allowing for any pile-soil slip which may occur. The immediate change in cyclic shear strain due to a change in cyclic shear stress should be considered in a similar manner to that in the method of predicting
accumulated settlement of an offshore foundation developed by Andersen et al (1976). This immediate change in cyclic shear strain is not considered in the program "SCARP". A modification to the "SCARP" analysis has therefore been developed, in which the pile displacement at the first cycle (i.e. the immediate displacement) has been calculated from the empirical relationship:

\[ S_{p1} = B \ e^{nX} \]  \hspace{1cm} (8.20)

The values of "B" and "n" are obtained by fitting the above relationship to the measured pile displacement after the first cycle from pile tests carried out at different load levels "X". A subsequent change in displacement which occurs if a change in load level takes place after \( N \) cycles, is computed as:

\[ \Delta S_p = S_N \ - \frac{m}{N} + B^n \ e^{nX} \Delta X \]  \hspace{1cm} (8.21)

where

\[ S_N = \text{pile displacement after } N \text{ cycles} \]
\[ \Delta X = \text{change in load level between cycles } N \text{ and } N+1 \]

This modified approach to calculation of permanent displacements will be referred to as the "modified SCARP" analysis. Both the "SCARP" and the "modified SCARP" analysis will be used in the comparisons between the model test results and theory.

For storm loading, the immediate pile displacement for each load parcel which was due to a change in the load level of storm loading has been considered in this modification.
8.4.1 Program SCARP Capabilities

The capabilities and limitations of program SCARP include:

a) variable pile conditions, i.e. a maximum of 31 elements per pile including base and discontinuity elements.

b) all piles in a group behave identically and a maximum group size of 12 piles can be analysed.

c) a maximum of 1000 increments may be used within a static loading sequence.

8.5 DETERMINATION OF PARAMETERS REQUIRED FOR ANALYSIS

This section describes the parameters required for the SCARP analysis, and how they were assessed or derived for the model pile test comparisons.

Two groups of parameters are required to analyse static and cyclic loadings for both single piles and piles in a group:

a) Parameters Required to Analyse Static Loading

1- embedded length of the pile (LEN = 290mm ),

2- diameter of pile base and cross-sectional area of element I (DB = 25mm, AR(I) = 490.87mm ),

3- number of shaft and base elements into which pile is divided (NS = 10, NB = 1 ),

4- Young's modulus of the pile (EP = 46647 MPa),

5- Poisson's ratio of soil (PR = 0.3),
6- Young's modulus of the base of the vessel (E_BASE = 100 MPa assumed),
7- Thickness of soil layer (H = 480mm),
8- The values of Young's modulus of the soil and skin friction were obtained as follows:

It has been found that the theoretical analysis of the uniform cyclic and storm loading tests using the "SCARP" program is sensitive to the Young's modulus of the soil "E_s" and the parameters "m" and "n". The evaluation of the "m" and "n" values has been presented in Section 8.4. The value of E_s was found to be dependant on the soil density, and for each sample, the Young's modulus obtained from load-deflection curve data varied with load level.

Figure 8.2 shows the relationship between the value of E_s and the maximum load level normalized by the ultimate load "P_s" for tensile load tests on model piles jacked into calcareous sand with different soil density. The results of these tests are presented in Chapter 6. The general trend is for Young's modulus to decrease with increasing load level. At a load level of P/P_s = 0.2, the range of E_s values is 9 to 36 MPa for the looser and denser samples respectively, while E_s values are between 2 to 9 MPa for the looser and denser samples respectively at a load level of P/P_s = 0.7. The effect of density on the value of E_s at P/P_s = 0.2 is similar to that at P/P_s = 0.7, regardless of load level.

b) Parameters Required to Analyse Cyclic Loading (Load-Controlled)

The extra parameters are defined as m, n and A, and have been obtained from the measured data for a single pile under constant amplitude cyclic loading.

The approach for obtaining the parameters for use in calculating the permanent
accumulation of displacement of the single pile or a pile within a group when subjected to specified cyclic loading is similar to that used by Diyaljee and Raymond (1982). The permanent displacement depends on the level of cyclic loading which is defined by a mean load and cyclic load for a specified number of cycles and the displacement during the previous cycle.

As described in Chapter 6, some results of controlled-load tests on a single model pile for different load levels have been presented in Figure 6.46. These results are replotted on logarithmic scales in Figures 8.3 and 8.4.

Figure 8.3 shows a plot of the logarithm of accumulated displacement against logarithm of number of cycles as a curve with slope "m" and intercept "A" for a specified stress level. The resulting slopes from the plotting of curves for all the tests are nearly constant. There is little difference between the values of the intercept "A", and thus the average of the values can be used for program "SCARP".

Plots of the logarithm of permanent cyclic displacement and the load level at the pile head for a certain number of cycles of load are presented in Figure 8.4. The value of both "m" obtained from Figure 8.3 and the intercept "A" obtained from Figure 8.4 are adopted in eq (8.16) to predict the accumulation of permanent cyclic displacement of a pile or pile group.

The tests carried out at various combinations of mean (P₀) and cyclic (±P₀) load provide average values of m = 0.35, n = 3, and A = 0.08.

The other parameters needed for the cyclic loading model are UEDFMIN (D_{min}), DLAMB (λ), and BDFMIN (D_{min} for base element). These parameters have been
derived from the experimental results on single piles. The following parameters have been used:

\[
\text{UTDFMIN} = \text{minimum degradation parameter } D_{\text{min}} \text{ for skin friction for all elements } (D_{\text{min}} = 0.2) \\
\text{DLAMB} = \text{degradation rate parameter } \lambda \text{ for base and discontinuity elements } (\lambda = 0.55) \\
\text{BDFMIN} = \text{minimum degradation parameter } D_{\text{min}} \text{ for base and discontinuity elements } (D_{\text{min}} = 1 \text{ has been assumed here i.e. no degradation of pile base capacity.})
\]

In the analysis, ten shaft elements were used to represent the pile, the distribution of skin friction along the shaft was assumed to be uniform.

8.6 COMPARISONS BETWEEN ANALYSIS AND MEASURED RESULTS

8.6.1 Pile Installation

Analysis of the results of installation from the preceding two Chapters (6 and 7) gives an indication of fairly uniform skin friction distribution down the pile during and after jacking. This is reflected in the assumption of uniform skin friction employed in the theoretical analysis of the residual load which is developed after the jacking force is removed.

Generally the ultimate skin friction for "medium dense" sand has been found to be less than that for dense sand for both single piles and a pile within a group, as discussed in Chapters 6 and 7 previously.
However, from experimental pile test results, the skin friction from static loading is about 12 kPa acting upwards and the average shear stress remaining on the pile after jacking is about 4 to 5 kPa acting downwards when calcareous sand is subjected to the 100 kPa overburden pressure. However, the average shear stress increases to be about 7 to 8 kPa when the sand is subjected to 200 kPa overburden pressure.

In order to model the residual load, it has been assumed that: i) the limiting shear stress is uniformly distributed along the pile. ii) The value of shear stress and Young's modulus used in the analysis are the same as those mentioned above. iii) The installation has involved the generation of full slip and end bearing before removal of the load.

The program SCARP has been used to model the installation of the pile, in the manner described by Poulos (1987). The pile has been loaded to failure statically, and then unloaded to zero head load in the analysis.

Figure 8.5 (a and b) shows the predicted and observed relationships between residual load versus depth relationship of a single pile jacked into medium-dense sand under 100 kPa and 200 kPa overburden pressure. The theoretical residual load increases linearly from the base to the top of the pile, and the maximum value is about 100 N for the sample subjected to 100 kPa (Figure 8.5a) and about 150 N for the one subjected to 200 kPa (Figure 8.5b). The general trend and the maximum predicted value of residual load are similar to those obtained from the tests.

The predicted results for residual load in a single pile, and a pile within a group, are the same, if the same soil parameters are used. The experimental results also showed no significant difference between a single pile and a pile in a group, under
the same conditions of sand density and overburden pressure.

8.6.2 Static Behaviour

The static load–deflection behaviour for different pile groups plotted from experimental data presented in the preceding Chapter is replotted in Figure 8.6 and 8.7. These results can be compared with those obtained from the computer program "SCARP".

The average value of ultimate static shear stress and Young's modulus for each group was calculated from the results of a series of tests carried out on single piles, 2-pile groups and 4-pile groups. The shear stress was again assumed to be uniformly distributed along the pile in this modelling. For the single pile modelling, two different densities (medium–dense and dense) and two overburden pressures (100 kPa and 200 kPa) have been used. However, for modelling a pile within a 2-pile group or a 4-pile group, only the case of medium–dense sand and an overburden pressure of 100 kPa has been considered. The pile dimensions, number of piles the in group, soil modulus and other parameters which are listed in section 8.5 have been used to model the static–load deflection behaviour.

Figure 8.6 compares predicted and measured load–deflection curves for single pile tests in medium dense sand subjected to an overburden pressure of 100 kPa. The results show that the stiffness and the pile capacity obtained from experimental test correspond very well with that from theoretical analysis, but failure develops more gradually than is indicated by the theory.

The same relationships for dense sand subjected to 100 kPa are demonstrated in Figure 8.7. It can be seen that good agreement is obtained between the stiffness
from theory and experiment while the predicted pile capacity is slightly lower than that measured. The stiffness for dense sand for both predicted and measured results (Figure 8.7) are higher than those for medium dense sand (Figure 8.6). Figure 8.7 also shows that the strain softening behaviour of the measured results is not reproduced in the predicted results, which employs an elastic–perfectly plastic analysis. The same relationships for medium dense sand subjected to a high overburden pressure of 200 kPa are shown in Figure 8.8. It can be seen again that good agreement occurs between the theoretical and experimental stiffness and load capacity.

Figure 8.9 shows typical predicted load–deflection curves for the three types of group in medium dense sand under 100 kPa overburden pressure. The stiffness of the single pile is higher than that of the 2–pile group which is higher than that of the 4–pile group. However, the average values of the maximum static load (per pile) for the three groups are equal to 210 N.

Comparisons between the predicted and measured results for the three types of pile group are shown in Figure 8.10. It can be seen that the stiffness from the predicted result corresponds well to that from measurement. It can also be seen that the static load capacity from the predicted results corresponds well with that from the experimental results from single pile tests and very well with that measured from 2–pile and 4–pile groups.

Figures 8.11a and 8.11b show the measured and predicted load distributions at failure along the pile jacked into medium dense sand and subjected to two different overburden pressures. For both overburden pressures, the general trend of load distribution for both measured and predicted results is for an almost linear decrease from the pile head to the pile tip. The predicted and measured distribution of load
are about the same, indicating that the distribution of ultimate skin friction is close to uniform with depth.

The influence of overburden pressure on measured and predicted load distribution along the pile can be seen from the comparison between the results shown in Figures 8.11a and 8.11b. The value of the load in the pile at points near the soil surface obtained from the test conducted under 100 kPa overburden pressure is about half of that from the test conducted under 200 kPa.

8.6.3 Cyclic Behaviour

Since the end bearing measured during the static and cyclic tensile loading tests was zero, the end bearing in the theoretical analysis was also assumed to be zero for static and cyclic tensile loading.

The SCARP program has to be calibrated by using specific values of $E_s$ and $m$ in the input data in order to predict the results of the model pile and pile group tests. The calibration of the program was achieved by one parameter being held constant and varying the other, as illustrated in Figures 8.12 and 8.13.

Both Figures 8.12 and 8.13 show the relationship between accumulated displacement and number of cycles for uniform load-controlled cyclic loading tests. When $m$ is constant at 0.35 and $E_s$ is variable, Figure 8.12 shows theoretical results and one typical experimental result. When $E_s$ is constant at 2 MPa and $m$ is variable the results are shown in Figure 8.13. It appears that the best fit between the theoretical curves and the experimental curve is with $m = 0.35$ and $E_s = 2$ MPa.

It can be seen that the influence of $E_s$ on the theoretical results from the SCARP program is more significant than that of "m". The parameter "n" only influences the
results of accumulated displacement from storm loading and a value of 3 will be shown to be adequate later. Consequently, the m and n parameters used for predicting uniform cyclic loading and storm loading results have been taken to be 0.35 and 3 respectively (see Figures 8.3 and 8.4).

a) Uniform Cyclic Loading

Figure 8.14 compares the computed and experimental accumulated displacement for two cases in which a single pile was subjected to 100 cycles of uniform loading, (tests CL10 and CL12). Considering the maximum cyclic load level for test CL10 is \( X = 0.45 \) and for test CL12 \( X = 0.40 \), the analysis can simulate reasonably well the results of the model pile when \( m = 0.35 \) and \( E_S = 2 \text{MPa} \). For the same value of "m" and increasing the value of \( E_S \) to 4 MPa the predicted and measured static displacement values are the same, but during cycling the analytical result underpredicts the measured results. Figure 8.14 also shows that the displacements after the first 3 cycles for the observed results agree well with those for the results computed using the lower \( E_S \).

Figures 8.15 and 8.16 demonstrate that the analysis can simulate reasonably well the results of the model 2-pile and 4-pile groups when \( m = 0.35 \) and \( E_S = 2 \text{MPa} \). Again, when "m" is constant and \( E_S \) is increased to 4 MPa, the analytical result underpredicts the observed results. For \( E_S = 2 \text{MPa} \), it appears that the predicted accumulated displacement is slightly higher than that from measurement for the 2-pile group (Figure 8.15) and significantly higher for the 4-pile group (Figure 8.16).

From Figures (8.14, 8.15 and 8.16) it can be seen that the differences between predicted and measured accumulated displacements tend to increase as the number of
piles in the group increases. The reason for these increases may be attributed to the
assumed Young's modulus of the soil "E_s" between the piles in the group used in
the program "SCARP" being constant. However, in reality E_s between two piles is
higher than that near each pile shaft because of the decreasing strain level away
from the piles. As demonstrated by Poulos (1988a) a higher E_s between the piles
reduces interaction and therefore settlement. As a result, assuming a constant
modulus will tend to lead to an over-prediction of group settlement.

b) Storm Loading

All the same parameters used to predict pile response to uniform cyclic loading have
been used for predicting the behaviour of piles and pile groups to storm loading.

Figures 8.17 and 8.18 show the relationship between accumulated displacement and
number of cycles for a single pile subjected to the 1st pattern of storm loading
using a constant "m" of 0.35 and values of "E_s" of 2 and 4 MPa. Figure 8.17
shows the predicted and measured results under four parcels of cyclic load at levels
of 0.1P_s, 0.2P_s, 0.25P_s and 0.35P_s. It appears that the predicted results at
E_s = 2MPa are in very good agreement with the experimental results, while the
experimental results are under-predicted when using E_s = 4MPa. Similar behaviour
can be seen from the comparison between experimental and predicted results for the
higher load levels (i.e. 0.2P_s, 0.35P_s, 0.45P_s and 0.65P_s), in Figure 8.18.

The comparison of predicted and measured results for 2-pile and 4-pile groups
under the first pattern of storm loading is demonstrated in Figures 8.19 and 8.20
respectively.

Figure 8.19 compares the computed accumulated displacement for a 2-pile group
subjected to 100 cycles of storm loading, with that determined from the model test. It can be seen that the computed results using $E_S = 2\text{MPa}$ slightly over-predict the experimental results in the first parcel and subsequent load parcels. The computed results using $E_S = 4\text{MPa}$ generally under-predict the experimental results.

Figure 8.20 shows that for a four-pile group, the computed accumulated displacement with $E_S = 4\text{MPa}$ agrees well with that from measurement. At $E_S = 2\text{MPa}$ the difference between computed and measured accumulated displacement is significant for the 4th parcel of load. The difference between the predicted and measured behaviour of piles in a group under storm load is similar to that for uniform cyclic loading, because the value of $E_S$ used in the SCARP program is assumed to be constant between the piles, and hence the interaction effects are over-estimated in the program.

Comparisons between predicted and experimental results for the settlement in the first cycle for each load parcel show a generally similar trend and reveal that the "immediate" settlement increases with increasing load level. However, the rate of increase for the predicted result is higher than that from the experimental results.

Figures 8.21, 8.22 and 8.23 show the experimental and computed results for a single pile, 2-pile and 4-pile groups, for the second pattern of storm loading, which consists of six parcels (i.e. $0.17P_S$, $0.25P_S$, $0.35P_S$, $0.45P_S$, $0.35P_S$, and $0.25P_S$). The results computed for the single pile (Figure 8.21) over-predict the settlement with $E_S = 2\text{MPa}$ and under-predict settlement with $E_S = 4\text{MPa}$ for the first four load parcels. However, the two predicted results both under-predict the displacement for the last two parcels in which the load level decreases. The behaviour of the 2-pile group appears to be similar to those of the single pile (Figure 8.22). Figure 8.23 shows that the measured accumulated displacement for the four pile group agrees
very well with computed results with \( E_s = 4 \text{MPa} \) in the first 3 load parcels, but then the measured displacement shifts during subsequent parcels to approach to the computed results for \( E_s = 2 \text{MPa} \) at the last load parcel.

From the last three Figures, it can be seen that the analysis assumes that a change in accumulated displacement accompanies any load change, but this does not seem appropriate for loading that decreases. It would seem very likely that a significantly higher soil modulus would be relevant to unloading than to first loading. A modification would be possible to the program "SCARP" that would, take this effect into account and reduce this "jump" in displacement when the load level decreases.

8.7 SUMMARY

The predicted residual load in the pile after jacking increases linearly from the base of the pile, and the maximum value is about 100 kPa for the sample subjected to 100 kPa and about 150 kPa for the one subjected to 200 kPa. The general trend and the maximum predicted value of residual load is similar to that obtained from experimental tests on a single pile.

The predicted and experimental results for residual load within a pile in a group, are similar, and are also similar to those for a single pile.

Comparisons between the predicted and experimental load–deflection behaviour during tensile static loading of single piles and pile groups jacked into medium dense and dense sands under both 100 and 200 kPa overburden pressure have been made. These show that the stiffness and the load capacity obtained from experimental tests
correspond very well with those from theoretical analyses.

The influence of overburden pressure on the measured load distribution along the pile is similar to that predicted from theory. The value of the load at a point near the soil surface obtained from the tests conducted under 100 kPa overburden pressure is about half of that from test conducted under 200 kPa.

The computed and measured accumulated displacements have been found to be in good agreement for a single pile, while the computed results slightly over-predict the displacement for a 2-pile group and significantly over-predict that for a 4-pile group. The differences between the computed and measured results may be due to the assumption (in the theory) that the Young's modulus of the soil between the piles in the group is constant. In fact, the Young's modulus of the soil increases with increasing distance from the pile shaft, and leads to a smaller interaction factor between two piles, and hence a smaller displacement than predicted.

In the case of the first pattern of storm loading, the predicted immediate settlement in the first cycle for each load parcel increases with increasing load level in a similar way to that obtained from measured results, but the rate of increase for the predicted results is higher than that from the experimental results. However, for the second pattern of storm load, the measured results did not show a significant rebound during the first cycle of each decreasing load parcel. Consequently a modification of the (SCARP program) may be required to reduce the predicted upward "jump" in the pile displacement.
Figure 8.1 Model of Axially Loaded Pile

a) Stresses Acting
   on Pile Elements

b) Stresses Acting
   on Soil Elements
Figure 8.2 Backfigured Young’s Modulus Versus Normalized Load Level (P/Ps)
Figure 8.3 Accumulated Displacement Normalized with Respect to Pile Diameter vs Number of Cycles

Figure 8.4 Accumulated Displacement Normalized with Respect to Pile Diameter vs Load Level
Figure 8.5  Measured and Predicted Residual Load Versus Depth for Single Pile
Figure 8.6  Measured and Predicted Load–Deflection Curves During Static Loading (Single Pile)

Figure 8.7  Measured and Predicted Load–Deflection Curves During Static Loading (Single Pile)
Medium–Dense Sand
Overburden Pressure = 200kPa

- Experiment
- Theory

Figure 8.8  Measured and Predicted Load–Deflection Curves During Static Loading (Single Pile)
Figure 8.9  Typical Predicted Load–Deflection Curves for different Pile Groups

Figure 8.10  Predicted and Measured Load–Deflection Behaviour of Different Pile Groups
Figure 8.11 Measured and computed Load Distributions at Failure During Static Loading
Figure 8.12 Sensitivity of Prediction of Displacement to Parameter (m) (Single-Pile)

Figure 8.13 Sensitivity of Prediction of Displacement to Young's Modulus (Es) (Single-Pile)
Figure 8.14 Analytical and Measured Accumulated Displacement Versus Number of Cycles
Figure 8.15 Analytical and Measured Accumulated Displacement Versus Number of Cycles (2-pile Group)

Figure 8.16 Analytical and Measured Accumulated Displacement Versus Number of Cycles (4-pile Group)
Overburden Pressure = 100kPa

Ps = Ultimate Static Load
Pst = Storm Load

Normalized Storm Load "Pst/Ps"

0.35Ps
0.25Ps
0.2Ps
0.1Ps

Storm Load Range

Number of Cycles

(a)

(m = 0.35 & n = 3)

Experiment
Theory (Es=2MPa)
Theory (Es=4MPa)

Test No. ST1
Medium—Dense Sand
Overburden Pressure = 100kPa

(b)

Figure 8.17 Predicted and Measured Accumulated Displacement vs Number of Cycles for Single Pile
Figure 8.18 Effect of Storm Load on Accumulation of Displacement for Single Pile
Medium—Dense Sand
Overburden Pressure = 100kPa
$P_s =$ Ultimate Load for the Group
$m = 0.35$ & $n = 3$

(a)

Normalized Storm Load $\frac{P_s}{P_s}$

(b)

Figure 8.19 Comparison Between Predicted and Measured Accumulated Displacement for Two Pile Group
Medium-Dense Sand
Overburden Pressure = 100 kPa
$P_s = \text{Ultimate Load for the Group}$
$(m = 0.35 \& n = 3)$

Figure 8.20 Comparison Between Predicted and Measured Accumulated Displacement for Four Pile Group
Medium—Dense Sand
Overburden Pressure = 100kPa
$P_s =$ Ultimate Load for the Group
$(m = 0.35 \& n = 3)$

![Normalized Storm Load Plot](image)

Figure 8.21 Predicted and Measured Accumulated Displacement for Single Pile
(2nd Pattern of Storm Loading)
Medium-Dense Sand
Overburden Pressure = 100 kPa
Ps = Ultimate Load for the Group
(m = 0.35 & n = 3)

Figure 8.22 Predicted and Measured Accumulated Displacement for Two-Pile Group (2nd Pattern of Storm Loading)
Figure 8.23 Predicted and Measured Accumulated Displacement for Four-Pile Group (2nd Pattern of Storm Loading)
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SUMMARY AND CONCLUSIONS

9.1 INTRODUCTION

Many difficulties have arisen in the design and construction of foundations for offshore structures installed in calcareous sediments in the last decade. This has motivated research to try to achieve a better understanding of the behaviour of offshore piles in calcareous sand. Static and cyclic laboratory and field tests on piles in these soils under different boundary test conditions (i.e. Poulos and Lee, 1988; Randolph et al, 1988; Khorshid, 1990) have been performed in the past. However, the behaviour of pile groups, the interaction between piles in calcareous sands, and also many aspects of boundary conditions in pile tests, have not been addressed in the past.

The major purpose of this thesis has been to perform an extensive static and cyclic laboratory test programme on model piles and pile groups under different soil densities, test boundaries, and cyclic loading conditions in order to investigate their influence on the behaviour of the piles. The test programme also included an
extensive number of static and cyclic shear box tests to determine the strengths of the soil and the soil–pile interface. The results of these tests may be used to develop methods of analysis which are capable of simulating the important aspects of the behaviour of piles and pile groups in calcareous sand.

The general literature review in Chapter 2 deals with the properties of calcareous soil and with the behaviour of offshore isolated piles and pile groups. The former were investigated by various triaxial tests (CID and CIU), and shear box tests (CNL and CNS), while the latter were investigated using experimental and theoretical models. The conventional theoretical methods used in the design of onshore foundations may not be directly applicable for offshore foundations because of the unusual nature of calcareous sediments and the unique loading condition imposed on the piles by wave loading.

9.2 ENGINEERING CHARACTERISTICS

9.2.1 Static and Cyclic "CNL" Direct Shear Tests

As described in Chapter 3, several series of static and cyclic direct shear tests described in Chapter 3, were carried out on calcareous and silica sands under the constant normal load "CNL" condition. The results of static shear tests for both soil–soil and soil–metal interfaces indicate that:

1) The peak and ultimate shear strengths depend on the normal stress, void ratio, and angularity of the soil particles. The peak shear strength of dense dry calcareous sand was developed at relatively large shear displacements, whereas that of dense silica sand was developed at a smaller shear displacement. This is consistent with the fact that the friction angle was fully mobilised at a smaller shear displacement in a sand of lower initial void ratio, as observed by Semple
(1988).

2) For a particular void ratio and normal stress, the effect of particle size on shear behaviour is relatively small.

3) Both the internal angle of friction and the soil-metal interface friction angle decrease as the initial void ratio increases. For the same density, the angle of internal friction (\(\varphi\)) and the interface friction angle (\(\delta\)) for calcareous sand are greater than those for silica sand. The magnitude of \(\varphi\) is greater than that of \(\delta\) for each type of sand.

The results of cyclic shear tests on sands show that the reduction in shear stress during cyclic loading depends on the normal stress, cyclic horizontal displacement, void ratio, the size and shape of the particles, and the number of cycles. However, the rate of change of shear stress decreases with increasing numbers of cycles. Similar behaviour is observed in sand-metal interface tests. All the calcareous sands show greater compressibility than the silica sand. Reasonable agreement was found between the results of the present tests performed on carbonate and non-carbonate sediments, and those obtained by other investigators.

The results of the tests to investigate particle damage due to grain crushing indicated that the coarse particles of calcareous soil are more prone to particle crushing than finer particles and the damage to particles decreases as the initial void ratio decreases. Silica sand exhibited minimal damage from shearing.

9.2.2 Static and Cyclic "CNS" Direct Shear Tests

The results obtained from a series of cyclic shear box tests on very dense calcareous sand samples under constant normal stiffness (CNS) conditions are described in Chapter 4. It is demonstrated that the reduction in shear stress in the CNS shear
box tests can be predicted from the results of the standard CNL shear box tests. Thus, it is suggested that standard shear box tests may be used to estimate the degradation of pile skin friction during cyclic pile loading.

The value of the degradation factor \( (D_r) \) for skin friction depends on the influence of cyclic loading on the skin friction. When \( D_r = 1 \), cyclic loading has no influence on the skin friction; if \( D_r < 1 \) reduction in skin friction due to cyclic loading occurs; and for \( D_r > 1 \) cyclic loading increases skin friction (this may occur in the first few cycles for loose sand under small amplitudes of cyclic loading).

The main features discussed in Chapter 4 were:

1) The CNS shear box tests provide a good understanding of the effect of cyclic loading on the degradation of ultimate shear stress of calcareous sand and confirmed the results from the CNL shear box tests. These results are similar to those reported for CNS cyclic simple shear tests performed by Morrison et al (1988).

2) The CNS tests have indicated that the greatest degradation of skin friction occurred in the first few cycles. This pattern of behaviour is similar to that shown for model and full scale pile tests.

3) The CNS test gives reasonable estimates of the behaviour of piles in sand, taking into account the stiffness of the surrounding soil. The degradation of the pile skin friction can be obtained from simple cyclic shear CNS tests. A comparison between the results of skin friction degradation for a model pile and CNS tests for 50 cycles and different cyclic displacement \( (\rho_C) \) is presented in Chapter 6. The results show that the reduction in skin friction and shear stress in the model pile tests can be predicted from the results of the CNS tests. This method will also allow comparison between different soil responses and give an
indication of the relative loss of skin friction. It also follows that the degradation of pile skin friction can be assumed from the results of conventional CNL direct shear tests.

9.3 EFFECT OF BOUNDARY ON TEST RESULTS

Chapter 5 presents a series of tests that were carried out to investigate the effects of the uniformity and density of the soil, size of the test vessel and boundary conditions of the vessel base on the behaviour of model piles.

It was found that to produce sand beds of controlled and reproducible density in the laboratory, a diffusing rainer device had to be used. The density of the sand bed varied between loose and medium–dense depending on the height of precipitation of the sand. It is possible to determine reasonably accurately the density of each layer of sand in the testing vessel by using different types of tins (rigid–base, flexible–base, permeable–base, and hollow–base tins). Dense samples can be prepared by vibrating the sand bed with a vibrator before consolidation.

Using two different test vessel sizes, and different base conditions, allowed a study to be made of the effects of side and base boundaries on the stress conditions in the test sand bed. An investigation dealing with the transfer of the overburden pressure to the bottom of the soil and the influence of shear resistance between the soil and internal wall of the test vessel revealed a good agreement between the measured and predicted values of vertical stress at the base of the vessel. For the larger vessel, the pressure at the bottom of the vessel was about 75% of the applied overburden pressure. This would affect the distribution of skin friction along the jacked pile. Thus, the base of the vessel was modified to apply pressure at the bottom end of the sand sample to provide a more uniform distribution of load along
the shaft and make the end bearing capacity of the pile more repeatable.
All surface deflections increased from the edge of the vessel toward the centre for
loose and medium-dense sands, but they were more or less constant for dense sands.
In the case of model pile tests, the time for complete consolidation was about 20
hours, and it was therefore important to allow sufficient time for consolidation prior
to testing, so that the loading tests on jacked piles would not be influenced by soil
consolidation movements.

An applied overburden pressure at both the top and bottom ends of the sand was
found to reduce the surface deflection during jacking and pile loading. The influence
of initial sand density on the magnitude and direction of the surface deflection of
calcareous sand was similar to that found for silica sand by Vesic (1967).

9.4 SINGLE PILE BEHAVIOUR

The response of jacked model piles in medium-dense and dense New North Rankin
calcareous sands was studied by performing: i) displacement-controlled tests, ii)
load-controlled tests, and iii) storm load tests (Chapter 6).
The behaviour of a single pile during jacking can be summarized as follows:

1) The jacking force increased to a peak at about 3 to 4 pile diameters
penetration and then decreased to an ultimate value. The peak load was well
defined for dense sand samples. The jacking resistance increased as both initial
soil density and overburden pressure increased.

2) The jacking force obtained from the tests in the small vessel was higher than
from tests in the large vessel. This reflects the influence of the side restraint
on the skin friction in the small vessel.
3) The maximum value of the developed residual load at the base of the jacked pile in dense sand was higher than that in medium–dense sand.

4) The residual compressive loads along the pile decrease with increasing pile movement towards failure, until the tip load is zero at a failure displacement of about 10% of the pile diameter.

5) The difference between the results obtained from the rigid base tests (R.B) and those from the controlled base pressure (C.B.P) tests is that the strain softening from the former appeared to be slightly more pronounced than for the latter. This may reveal the influence of the base condition on the sand density underneath the pile tip, and consequently, the influence of the base condition on the end bearing capacity of the piles during jacking.

When a static tensile load was applied on a single pile, the following features were shown:

1) In medium–dense sand, the tensile load increased to a peak load at about 0.5 mm deflection and then dropped slightly to an ultimate value. For dense sand, however, the peak point appeared to be more pronounced and developed at a smaller displacement.

2) Both skin friction and soil Young’s modulus increased with increasing soil density and increasing overburden pressure.

3) The shear stress along the middle of the pile appears to be higher than that at the top and bottom of the pile.

4) The tests carried out in the smaller test vessel showed generally higher values of skin friction, indicating that the results of these tests may be influenced by the side–wall restraint of that vessel.

5) The pile head stiffness from the C.B.P tests appeared to be somewhat higher than from the R.B tests. The distributed shear stress from the C.B.P tests was
more uniform than that from the tests with other boundary base conditions.

6) For comparison purposes, the static skin friction values observed from laboratory tests tended to be larger than those from field tests. However, the comparison between the present results and the results obtained for a model grouted pile by Lee and Poulos (1990) showed that the value of skin friction on a jacked pile in medium dense calcareous sand was much less than that on grouted piles.

When a pile was subjected to a displacement-controlled loading, the tests showed the following:

1) Cyclic loading causes significant reduction in skin friction and this reduction depends mainly on the cyclic displacement.

2) The rate of reduction of skin friction appeared to reduce with increasing number of cycles. This behaviour has also been observed in the direct cyclic shear box tests described in Chapters 3 and 4.

3) The degradation of skin friction ($D_\tau$) was influenced by increasing overburden pressure and sand density and it appeared to be relatively uniform along the pile length.

The cyclic load-controlled response of a pile was investigated by performing two types of test.

a) Uniform amplitude tests showed that the accumulated displacement increased with increasing load level and increasing number of cycles. However, the rate of increase reduced with increasing number of cycles.

b) Storm load tests, showed similar response to the case of uniform amplitude cyclic loading. The accumulated displacement in the first cycle was markedly higher than
for subsequent cycles for each parcel of load. For a specific load parcel, the accumulated displacement increased with increasing number of cycles, but the rate of increase reduced with the number of cycles. The accumulated displacement increased with increasing load level. The increment of accumulated displacement appeared to depend on the load level more than on the number of cycles.

Comparison between results of different loading patterns (which are defined in Chapter 6) showed little or no influence of load pattern on the pile skin friction.

9.5 PILE GROUP BEHAVIOUR

Static and cyclic tests on model 2-pile groups and 4-pile groups jacked into calcareous sand were carried out to investigate the factors that influence the load settlement behaviour and the skin friction development along piles.

The behaviour found during the jacking process of the pile groups is summarized below:

1) The end bearing and the skin friction developed during jacking generally increase as the initial sand density or the overburden pressure increases.

2) For piles jacked individually in medium–dense sand, the jacking force increases as the number of jacked piles increases, but the rate of increase decreases with the number of piles. The ratio of force for the 4th pile to that for the 3rd pile is slightly greater than 1. For dense sand, the jacking force increases during jacking the final three piles, but at a decreasing rate. The jacking load for the 4th pile appears less than that for the 3rd pile, hence the force ratio of 4th to 3rd is less than 1. This behaviour may be attributed to the change of
soil density during jacking of the piles in medium–dense and in dense sands. The change in density may be moving the soil towards a critical state that is largely governed by the normal stress developed, which in turn is dependent on the stiffness of the surrounding soil.

3) From the comparison between the results for piles jacked individually or together, it appears that jacking the piles together results in a more uniform distribution of the load among and along the piles.

4) The maximum value of developed residual load on one pile in a group in dense sand is higher than that in medium–dense sand. This behaviour is similar to that for jacking of a single pile.

When a static tensile load was applied to a pile group, the following behaviour was observed:

1) The tensile group load increases to a peak load and then drops to an ultimate value for both dense and medium–dense sands. For dense sand, the peak appears to be more pronounced and is developed at a smaller displacement than that for medium dense sand.

2) When the tensile applied load increases, the loads measured along the pile shaft reduce from residual compression loads and become tensile loads. The compression load observed near the pile tip, decreases with axial deflection until it reaches practically zero, indicating no tip resistance in uplift, as would be expected. This behaviour is similar for all types of pile group.

3) It appears that the group efficiency (\( \eta \)) increases as the number of piles in the group increases, and \( \eta \) increases with increasing sand density.

4) For both single piles and pile groups, the secant stiffness decreases with increasing load level.

5) It has been found that the stiffness of single piles is higher than that of the
2—pile group which again is stiffer than the 4—pile group.

6) The residual load along one pile in a group decreases as the group is loaded towards failure, this is similar to that of a single pile.

7) The uniformity of skin friction may be influenced by the number of jacked piles in a group.

The cyclic displacement—controlled response of a pile group displays the following characteristics:

1) The skin friction developed along the piles tends to reduce under cyclic loading, and the rate of reduction of skin friction appeared to reduce with increasing number of cycles.

2) The degradation factor $D_r$ for the pile groups is increased as the cyclic displacement, number of cycles, overburden pressure, and soil density increases. This behaviour is similar to that of single pile. However, $D_r$ appears to be more influenced by cyclic displacement increases than by the number of piles in group.

The cyclic load—controlled response measured while performing uniform amplitude tests showed a consistency with single pile results in that:

1) The accumulated displacement of pile groups increases as both load level and number of cycles increase. However, the accumulated displacement in the first cycle is markedly higher than for subsequent cycles.

2) A cyclic stability diagram for both 2—pile and 4—pile groups can be used to define the stable and unstable zones. Group effects do not appear to have a major influence on the cyclic stability diagram.
Under storm loading condition, the tests showed that:

1) For each parcel of load, the accumulated displacement in the first cycle is higher than for any subsequent cycles.

2) As observed in single pile tests, it also appears that, for a specific load parcel, the accumulated displacement increases with increasing number of cycles, but the rate of increase reduces with the number of cycles.

3) The accumulated displacement increases with increasing load level. The increment in accumulated displacement appears to depend on the load level more than on the number of cycles. This is also observed in the single pile tests.

4) It appears that the small influence of the number of piles in the group on the group capacity and group settlement in the case of uniform amplitude cyclic load is also repeated for the storm loading case.

9.6 INTERACTION TESTS

The behaviour of an unloaded pile during jacking of another pile showed the following:

1) The head deflection of the unloaded (1st jacked) pile increases with an increasing number of subsequently jacked piles, and also increases as the relative spacing "s/d" decreases. This clearly demonstrates the interaction between piles during their installation.

2) The direction of the deflection is either upward in dense sand or downward in medium–dense sand, demonstrating a significant influence by the sand density, and the magnitude of the deflection is influenced by overburden pressure.
During static and cyclic loading of a loaded pile, the unloaded pile showed these characteristics:

1) The head deflection of the unloaded pile depends on the spacing between piles. The deflection increases with decreasing spacing between two piles. This may be attributed to an interaction between the piles through the soil (i.e. pile-soil-pile interaction).

2) The influence of the sand density on the direction of deflection of the unloaded pile, and the influence of overburden pressure on the magnitude of the deflection, is similar to that observed during jacking.

3) The minimum value of induced load in an unloaded pile due to loading of adjacent piles appears in the top part of the pile and the maximum load occurs in the lower part of the pile.

4) The force along the unloaded pile reduces due to load cycling on the loaded pile. It appears that the greatest reduction occurs at the bottom part of the pile. This indicates the existence of a pile-soil-pile interaction during cyclic loading, as well as during jacking and static loading.

9.7 COMPARISON BETWEEN PREDICTED AND MEASURED RESULTS

This section summarises the comparison between the behaviour of predicted and experimental results of a single pile and pile groups during jacking, tensile loading and cyclic loading.

The predicted and experimental behaviour for load-deflection for single pile and pile groups jacked into medium dense and dense sands under both 100 and 200 kPa overburden pressure, show a good agreement. The predicted and experimental results
for residual load for a single pile and a pile within a group are similar.

The influence of overburden pressure on predicted load distribution along the pile is similar to that which was measured experimentally.

For a specific value of Young's modulus, the predicted and measured accumulated displacement (for both uniform cyclic and storm loading) have been found to be in good agreement for a single pile, but agreement was not as good for pile groups. The differences between predicted and measured results increases with the number of piles in the group. This behaviour may be attributed to the assumption of a constant Young's modulus for the soil "E_s" between the piles in the group which is used in the program "SCARP". In reality E_s between two piles is higher than that near each pile shaft because of the smaller strain level, and this can lead to a decrease in the amount of interaction between the piles.

The predicted immediate settlement in the first cycle for each load parcel increases with increasing load level. This behaviour is similar to that obtained from experimental results, but the rate of increase for the predicted results is higher than that found from experiment.

9.8 Suggestions for Future Research

The work described in this thesis is considered to have contributed to a better understanding of the behaviour of piles and pile groups in calcareous sediments, under both static and cyclic loading. The importance of the volume change characteristics of the soil on pile skin friction degradation under cyclic loading has been demonstrated, and it has been found that it may be possible to predict this
degradation using conventional CNL direct shear tests. Some of the findings of this work are, as follows:

1) The model pile tests have been carried out on relatively small diameter piles. Ideally tests on larger model piles (e.g. up to 150mm diameter) would be desirable to examine scale effects on skin friction, end bearing, Young's modulus, and cyclic degradation.

2) Further modification of the vessel design may be made, firstly to the lining of the side wall to reduce the friction between the wall and soil, and secondly in the way the surcharge pressure is applied. Consideration might be given to applying controlled pressure around the circumference, as well as at the top and base, to produce more uniform stress within the soil profile.

3) This study should be extended to larger model pile groups than those used here and the influence of pile position within a group on the individual pile response should be studied.

4) The behaviour of piles in non-homogeneous soil requires investigation. Experimental conditions could be modified to simulate variation of strength and stiffness of the soil within the test vessel.

5) Comparisons between theoretical predictions of pile behaviour and the measured response have also identified several shortcomings of the present theory which should be the subject of further research. In particular, the following aspects require attention:

a) the modelling of progressive cyclic degradation of skin friction; the
Matlock and Foo model presently used has limitations in its ability to predict the rate of degradation with increasing cycles.

b) the modelling of the accumulation of permanent displacements. At present, the procedure used is empirical and far from satisfactory, ideally it requires a proper applied mechanics basis.

c) the present group analysis assumes that the soil between the piles is homogeneous, it should be extended to allow for the soil between the piles to be stiffer than it is near the piles.
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