INVESTIGATIONS OF PILE-SOIL BEHAVIOUR, WITH SPECIAL REFERENCE TO THE FOUNDATIONS OF OFFSHORE STRUCTURES

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by

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Everything should be made as simple as possible, but not simpler.

Albert Einstein

ABSTRACT

The Thesis is concerned with ways of improving predictions of the performance of large piled foundations. Previous work is critically reviewed and the components required for a realistic effective stress analysis identified. Parametric studies using the finite element method are used to show the interelationship between ultimate capacity and displacements at working loads, and the need for complete analyses is discussed.

Experimental programmes are described in which elements of the problem are further investigated. The work includes laboratory and model-scale field tests, prototype instrumentation and numerical experiments. Details are provided of the methods and equipment developed for the work, and comprehensive laboratory test programmes on Magnus and London clay are reported. A special feature of the laboratory tests is the use of precise instrumentation for the measurement of deformation characteristics at small strains, and the data obtained provide new insights into soil behaviour.

The laboratory results are compared with available field data and good agreement is generally found. Non-linear finite element analyses are reported which demonstrate the significance of the determined high-initial stiffness, non-linear, soil characteristics in a range of boundary value problems.

The results of a field scale model pile test programme are presented, where measurements of the effective stresses acting on a steel pile jacked into London clay have been obtained. The data are correlated with the parallel laboratory studies, and attention is drawn to a number of interesting phenomena.

In the final sections of the thesis, the improved understanding gained of the individual components of the piling problem is used to undertake foundation analyses for two important North Sea installations; the Magnus production platform, and the Hutton Tension Leg Platform (TLP). Instrumentation, developed as part of the

research, has been deployed at both sites and comparisons are made between the recorded data and the results predicted by the improved numerical analyses. Surprisingly good agreement is noted, and a number of practical and theoretical conclusions are drawn from the studies.

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CHAPTER 1

INTRODUCTION

1.1 THE SCOPE OF THE THESIS

1.1.1 Over the past decade the exploitation of North Sea oil and gas fields has presented a great variety of challenging engineering problems. At the time of writing a second wave of development is underway. Platforms are planned for previously marginal fields, and extraction is to take place from ever increasing depths and harsher environments. Future projects are thus likely to require both refinements in existing technologies, and the evolution of new designs for structures and foundations.

1.1.2 Most platforms installed in the U.K. Sector are founded on groups of driven piles, and surveys[1] indicate that this trend will probably continue. One of the main applications of soil mechanics in North Sea projects has therefore been in the assessment of driven pile performance; geotechnical engineers have been concerned with developing practical and analytical techniques to cope with the various problems of driveability, ultimate capacity, load-deflection characteristics and response to cyclic loading.

1.1.3 Foundation designers now have a range of tools at their disposal, but many of the approved calculation methods are based on empirical or total stress interpretations of a limited number of mainly onshore pile tests. Experience in other areas of geotechnical engineering, including slope stability and earth pressures, warns against analyses of soil behaviour that are not founded on the central principle of effective stress. In many ways the soil mechanics of piling has not yet been able to properly benefit from the insights provided by Terzaghi.

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[1] See Offshore Engineer (1984)1.

1.1.4 It is clear that design procedures based on a fundamental understanding of pile-soil mechanics would be preferable to the present empirical methods. If properly developed, these procedures could account for the particular site details of stratification, geological history, soil properties, pile type and loading pattern. Such methods would be invaluable in extending present experience to new circumstances, and optimising design in more familiar conditions.

1.1.5 Many attempts have been made to formulate the problems in an appropriate fashion and Terzaghi (1926) was perhaps the first to consider the effects of driven pile installation in terms of effective stresses. However, the analysis of displacement piles has proved to be particularly intractable, and many promising approaches founder when theoretical results are compared with measured behaviour.

1.1.6 Over the past two decades major advances have been made in soil mechanics, particularly in the areas of laboratory and field experimental techniques, soil constitutive modelling and the understanding of continuum behaviour. As a result of these developments, it has recently become possible to consider many elements of a fundamental analysis of pile behaviour with unprecedented realism.

1.1.7 The aim of this thesis is to summarize and extend recent work in order to attempt predictive analysis of the full load-displacement characteristics of the foundations of two important North Sea platforms. The method adopted is to first consider individual components of the problem in Chapters 2 to 9. These elements are then synthesised in complex non-linear numerical studies of the Magnus and Hutton Tension Leg Platform foundations. The locations of these two structures are shown in Figure 1.1.

1.1.8 The two chapters following the introduction therefore include reviews of previously published theoretical and experimental results, and assess their relevance to the considered problems. Particular emphasis is placed on the need for a detailed knowledge of the soil's strength-deformation characteristics and a sound understanding of the processes of pile installation, equilibration and loading to failure.

1.1.9 Chapters 4 to 9 report new investigations of a number of areas, and these include field, laboratory and numerical experiments. In order to carry out the various studies, developments were required in apparatus and techniques, and these are fully described. Wherever possible, checks are made between the results of experimental, or analytical, predictions and high quality measurements of field behaviour.

1.1.10 The full-scale finite element analyses presented in Chapters 10 and 11 incorporate the conclusions of the investigations regarding the geological sequences at the sites, the effective stress conditions developed in the field, the non-linear and elasto-plastic nature of the soil considered, the very high initial soil stiffnesses deduced from the laboratory tests, and the possible influence of residual fabric developed close to the pile. The numerical studies show how these factors combine to determine the axial capacity and load-deflection characteristics of the Magnus and Hutton foundations.

1.1.11 Chapters 10 and 11 also compare the predictions with the load-settlement responses measured on site. Indeed, a major part of the project described in this thesis was concerned with the development and deployment of underwater foundation settlement gauges for the two platforms considered. To help the flow of the central argument, a description of the Magnus instrumentation is not included in the main text, but is reported in the separate Appendix, A6. The further development of the equipment for use on the Hutton TLP is briefly described in a section of Chapter 11.

1.1.12 The final chapter considers the conclusions that may be drawn from the described research. The investigations of pile-soil behaviour give many insights that help understand the characteristics of large piled foundations, and also highlight areas that urgently require further work. Conclusions of a more general nature are also made concerning soil properties, field and laboratory techniques and the analysis of soil-structure interaction.

1.1.13 The thesis effectively summarizes the results of a four year programme sponsored by the Science and Engineering Research Council. The areas of research have been extensive, and as described in the acknowledgements, the author has received assistance in a number of ways. In particular, it should be emphasised that the numerical analyses which form important parts of Chapters 4, 8, 10 and 11 could not have been implemented without the participation of Dr. D. Potts.

1.1.14 The reporting of the research work in this thesis extends to twelve relatively long chapters. To ease cross-referencing for the reader the pages are separated into two volumes, the first contains the text, the second the figures and tables.

CHAPTER 2

A REVIEW OF CURRENT METHODS OF PILE ANALYSIS, AND THEIR USE FOR OFFSHORE STRUCTURES

2.1 INTRODUCTION

2.1.1 For thousands of years driven piles have been used to provide sound foundations in difficult conditions. In each instance decisions had to be taken regarding the size of pile groups, pile type, diameter and length of embedment. Empirical rules and approximate analyses have been developed to guide Engineers making these choices. This chapter reviews the methods of analysis that are currently in use and discusses their application for the foundations of large North Sea oil production platforms. Considerations of driveability are outside the scope of this thesis, and attention is concentrated on piles installed in clays.

2.2 EMPIRICAL METHODS OF PILE ANALYSIS

2.2.1 Lambe and Whitman (1969) noted that even the simplest piled foundation is statically indeterminate to a very high degree, and that the chance of precise analysis is thus more remote than is the case for most problems in soil mechanics. Historically, practical designs have been developed from the combination of empirical correlations and trial loading tests. For relatively long driven piles, such as those often used in offshore applications, the main design problems centre on the prediction of driveability, shaft capacity and displacement response to various components of loading.

2.2.2 The most widely used empirical correlations for shaft capacity in clays are based on proportioning the skin resistance to the original undrained shear strength profile; Tomlinson (1955), (1980), Skempton (1959), American Petroleum Institute (1977). Thus at failure

$$\tau_{rz} = \alpha \, Cu \qquad (1.1)$$

A grave difficulty with the 'alpha' method is the selection of the shear strength parameters for substitution into equation 1.1. It

is well established that the Cu profiles determi ned for a given layer depend on many factors; sampling method, test apparatus and loading rate are all important. The field studies with Boston Blue clay demonstrate this sensitivity in a convenient way, and figure 2.1 shows the range of strengths found by different methods at a single site. In this case, the sampling and test procedures were of research standard, and greater scatter should be expected from commercial site investigations. Morrison (1984) noted the effect of uncertainty regarding Cu on the α values deduced from his pile model pile results, and the reproduction of his interpreted data in figure 2.2 starkly emphasises the problems of the total stress method.

2.2.3 Terzaghi (1928) first considered the effective stress processes that control skin friction, and Zeevaert (1957), (1960), Eide et al (1961), Chandler (1968) and Burland (1973) proposed approximate formulations for calculating shaft friction in terms of effective stress. Chandler proposed Equation 1.2 for bored piles, assuming that σ'_r is not changed by installation or loading.

$$\tau_{rz} = K_o \sigma'_v \tan\delta' \qquad (1.2)$$

Burland suggested a similar equation which could cover a wider range of pile types;

$$\tau_{rz} = \beta \sigma'_v \qquad (1.3)$$

However, summaries of load tests often show large scatters in β and Vijayvergiya and Focht (1972) proposed a hybrid formulation which gives a better fit to the API collection of pile tests;

$$\tau_{rz} = \lambda(\sigma'_v + 2Cu) \qquad (1.4)$$

This expression includes both 'total' and 'effective' stress terms. All of the above methods separate the calculation of ultimate resistance from the consideration of changes in stresses and strains that occur in the soil from its undisturbed condition through the processes of installation, consolidation and loading to failure. Whilst the failure conditions can probably be written in the form of Equation 1.5, Engineers have been reluctant to assume values for σ'_r and δ'. The alpha method thus remains the most popular, even though there is no proven effective stress rationale for the choice of α values.

$$\tau_{rz} = \sigma'_r \tan\delta' \qquad (1.5)$$

2.2.4 In the simplified empirical design approaches total axial capacity is obtained by summing shaft and base resistances. The latter component is calculated by considering the base to be a separate deep foundation; Skempton (1951), Meyerhof (1963). The ultimate base bearing pressure, q_b, is usually taken as 9.0 Cu; Whitaker and Cooke (1966), Vijayvergia and Focht (1972), Burland (1973). Thus the total axial capacity is written as;

$$Q_T = \int_0^L \tau_{rz} . 2\pi r_o \, d\ell + 9.Cu_b . \pi . r_o^2 \qquad (1.6)$$

2.2.5 Approximate methods for calculating pile vertical load displacement relationships have also been developed. Poulos and Davis (1968) and Poulos (1968) obtained linear elastic solutions for the loading of incompressible single piles and pile groups, but as noted by Meyerhoff (1976) and Kezdi (1975) major difficulties remained in the choice of soil deformation modulus. Coyle and Reese (1966) proposed an approximate non-linear calculation routine. In their method the pile is considered as an elastic column on whose sides and base vertical tractions and forces act. The pile is divided into a number of elements and the soil forces, are applied as functions of pile displacement, u, so that the governing equation can be written as;

$$E_p A_p \frac{d^2 u}{d\ell^2} + F(u) = 0 \qquad (1.7)$$

St John (1980) referred to the shaft friction term, F(u), as a shear stress transfer function.

2.2.6 Coyle and Reese (1966) proposed methods of finding such functions from instrumented pile and laboratory model tests. Bea (1975), Holmquist and Matlock (1976) and Grosch and Reese (1980) present further model studies of resistance-displacement relation ships, and Vijayvergia (1977)1 summarizes laboratory and field records to produce general 'T-Z' curves. Once the curves have been derived, a numerical routine is employed to solve Equation 1.7 for all the segments and hence deduce a load-settlement relationship.

2.2.7 A number of objections may be raised to the use of the T-Z column routines;

(i) The methods neglect any interaction between the soil layers.

(ii) The laboratory models used to define T-Z relationships often impose severely unrepresentative boundary conditions on the pile-soil system. Difficulties also exist in the interpretation of field tests.

(ii) The methods were derived for single piles only. Pile group effects must be found from further empirical relationships or elastic solutions.

2.2.8 Similar procedures have been developed for the analysis of horizontal loading on piles in clay, and both vertical and horizontal loading on piles installed in sand. Readers are referred to Meyerhof (1976), Heins and Barends (1979) and St. John (1980) for considerations on piling on sands. St. John also reviews methods of estimating the response of piles to lateral loads.

2.2.9 Empirical design approaches are only valid when they are used to interpolate solutions between established results. They must therefore be based on carefully conducted experiments. In the case of piling these should properly represent pile type, method of instalation, loading history and geological conditions. Many useful collections of such data have been made including those of Chandler (1968), Burland (1973), Meyerhof (1976), Weltmam and Healy (1978) and Kraft et al (1981). However, there can be serious difficulties in interpreting pile test data due to:

(i) Considerable variations of capacity and stiffness to load with time; for effects with driven piles see Cummings et al (1950), Seed and Reese (1955), Eide et al. (1961), Gallagher and St. John (1980).

(ii) The practice of carrying out multiple loadings to failure as a means of evaluating the influences of time and loading direction on capacity or stiffness. As discussed by St. John et al (1983),

post peak reductions in capacity can lead to false conclusions being drawn from tests such as those described by Heerema (1979) or Cox and Kraft (1979).

(iii) The frequent monitoring of pile head displacements from datum positions which fall within the zone of influence of the piles.

(iv) Possible interactions between test piles and their reaction systems.

(v) The absence of good quality site investigations, and inconsistencies in sampling methods and ways of determining vital parameters such as undrained shear strength.

2.3 APPLICATIONS FOR NORTHERN NORTH SEA PLATFORMS

2.3.1 The piles on which North Sea platforms can be safely founded are generally far larger than those used onshore (St John (1980)). With lengths of up to 140 metres, and capacities in the range of thousands of tonnes, the costs of offshore load tests are prohibitive. Indeed, only the pair of tests described by Price (1971) and Fox et al. (1970) have been attempted. The experiments involved the loading to failure of two 760mm diameter piles. Both were driven in stages to a final penetration of 18.5 metres through a Lias and Boulder clay sequence beneath British Petroleum's West Sole platform, 42 miles east of the Humber Estuary.. The West Sole conditions are not typical of Northern North Sea platform sites.

2.3.2 Hight (1983) gives a review of the geological aspects of foundation engineering for the Central and Northern North Sea, he concludes that the foundation soils for most offshore fields are Holocene or Quaternary in age. An isopachyte map for the Quaternary soils is given in Figure 2.3 and shows thicknesses between 100 and 1,000 metres over the main part of the North Sea. These soils vary from loose to dense sands and soft to hard clays of mainly low to medium plasticity. The clays range from poorly sorted tills or moraines to more uniform glacio-marine deposits. Figure 2.4 shows typical ranges for index properties, and Figure 2.5 gives some grading curves.

2.3.3 Complex sequences of deposition erosion and redeposition have been interpreted for the Quaternary layers and Hight draws attention to the special effects of wave loading on strength and insitu horizontal stress. Readers are refered to Holmes (1977), Milling (1975), Caston (1977) or McCave et.al (1977) for details of the Quaternary Geology of the North Sea.

2.3.4 The current design code in use for North Sea piled foundations is the American Petroleum Institute's RP2A document, which was developed from an inspection of a number of driven pile load tests, most of which were carried out in the United States. The following sections discuss the relevence of this data base to North Sea foundation conditions.

2.4 THE API PILE TEST DATA BASE

2.4.1 The RP2A design code was drawn up from an interpretation of the 42 load tests collated by Vijayvergia and Focht (1972). Only the briefest mention of the site and test details are given by the authors and many of the original references are obscure. However, McClelland Engineers have recently researched the source data and, in some cases, made fresh interpretations of the soil conditions and shaft capacities. Their collection of records was used as the basis of this review.

2.4.2 The API data base is summarized in Table 2.1 where the pile details and soil parameters are indicated, as are the overall values of the shaft friction coefficient, α, and the set up times between driving and load testing. It can be seen that wide ranges of soil types and pile sizes were considered. The mean shear strength was 120kpa, and the average pile length 24 metres.

2.4.3 Inspection of the table shows the values of α to vary between 1.1 and 0.35. Such a margin of uncertainty is unacceptible in economic design and there have been a number of attempts to find ways of assessing appropriate values of the average skin friction coefficient between these limits. The API code assumes α to vary only with undrained shear strength and recommends the relationship drawn through the scatter plot shown on Figure 2.6. There is, however, no standard

method for determining the shear strength parameters. Vijayvergia and Focht (1972) considered pile length to be important and showed their lambda skin friction coefficient varying with depth. More recently Semple (1984) has suggested that the combination of overconsolidation ratio and depth governs the development of capacity and has replotted the API data in the form given in Figure 2.7. These scatters imply that α reduces with both depth and OCR.

2.4.4 However, it is reasonable to suppose that pile capacity depends on a number of additional factors and, in particular, soil type and set up time could be important. In Table 2.2 the pile test results are divided according to plasticity index. At first sight it would appear that α declines with increasing PI, but on closer examination it can be seen that the 'high placticity' tests include a disproportionate number of 'normally consolidated' sites where Cu'/σ'_v falls below 0.5. Also, the average set up time for the 'low plasticity' tests was less than half that for the two other categories.

2.4.5 The influence of set up time can be seen, crudely, in the scatter of data shown in Figure 2.8. There is a trend for the value of α to rise sharply, with apparent gains of up to 100% over 5 to 6 weeks. This is highly significant as most pile tests are carried out only a few days after installation.

2.4.6 The same body of test data might be used to look for the effects of pile type. Average α values of 0.73 are found for closed and piles, and 0.53 for open pipe piles. This result would, however, contradict the trends given by Kraft et al (1981) from their examination of a similar collection of test results. Complications could also be found in differences in sample quality and in the methods of measuring undrained shear strength. The API data show considerable differences between unconfined, confined triaxial and field vane strengths at the same sites.

2.4.7 It is evidently impossible to separately evaluate the various factors influencing the coefficient α from the pile tests alone. Capacity could be considered to depend on the profiles of soil properties (Cu, OCR, Ko, Cv, PI, etc.), the pile size and type, the set up time, the sense of loading and the rate of penetration. However,

the API data base offers a total of 14 sites and 42 tests, and thus any statistical exercise intended to separate the variables could not be justified. Although the records suggest that alpha increases with set up time, but decreases with depth and OCR, no single trend is dominant. Sections of the data might be directly applicable, however, if groups of the tests could be shown to be closely comparable to the foundation conditions at sites in the North Sea.

2.4.8 In general the piles in the data base are smaller than those used offshore and developed far less capacity. The steady penetration applied in a pile test also differs from the extreme conditions experienced by offshore structures, where the loads fluctuate rapidly due to the action of waves and wind. Furthermore, few of the sites share the particular geology of the Northern North sea. The ranges of soil composition can be judged from the distribution of plasticity index for the API test sites. This is compared in Figure 2.9 with the histogram prepared by Hight (1983) for a number of North Sea platform sites. The plots show the data base to include tests on far more plastic clays, with a mean PI of approximately 35%, compared with 25% for the North Sea locations. The mechanical properties are difficult to discuss, but a comparison can be made by inspecting the average values of shear strength and apparent overconsolidation ratios.

2.4.9 Semple (1984) used the ratios of mean shear strength Cu, to mean effective overburden stress, σ'_v, to estimate the degree of overconsolidatiuon at the API test sites. Curves relating Cu/σ'_v have been derived for K_o consolidated sediments, as described by Ladd et al (1977) and Gens (1982). In his review, Hight (1983) noted that Northern North Sea sites often show a characteristic soil profile with hard, apparently overconsolidated, clay layers within 20 metres of the sea bed, with lightly overconsolidated or normally consolidated clay at greater depth. It is convenient here to define the upper layers, as Zone A, where the minimum shear strength is 250 kpa and the minimum value of Cu/σ'_v is 1.25, which excludes all but hard, overconsolidated soils. The deeper layers, Zone B, can be defined as having a minimum shear strength of 100 kpa and a maximum Cu/σ'_v of 0.5. Thus, for OCR equal to 2.0, Zone B would commence at around 20 metres depth and could only be compatible with normally consolidated conditions at a depth of

about 40 metres. The limits to the zones are plotted in Figure 2.10 as combinations of Cu/σ'_v and Cu. The mean values from each of the tests in the API data base are shown, and it is immediately clear that few of the points fall in either area. Indeed, most are confined to a narrow region corresponding to shear strengths between 10 and 150 kpa with lightly overconsolidated conditions. The tests in the London Clay at Stanmore reported by Tomlinson (1970) and the North Sea tests at West Sole, referred to in 2.3.1, stand as exceptions. As might be expected, the latter tests fall into Zone A, and therefore merit at least a brief discussion here.

2.4.10 The axial pile loading experiments at West Sole were carried out in July 1969, and involved the staged testing of two pipes driven from penetrations of 3 to 18 metres in 3 metre intervals. Both tension and compression tests were made, and one of the piles was fitted with an internal driving shoe. The loadings were typically carried out 3 to 12 hours after the end of driving, but in 3 of the 22 staged tests a longer set up time of around 5 days was allowed.

2.4.11 Interpreted using the data from the 1968 site investigation, the results appeared to show alpha values of around 0.5. However, reinvestigations using modern site investigation techniques yielded an undrained shear strength profile which was considerably stronger, and so smaller values of α would be interpreted from the same load-displacement curves, (A. Speaker (1979)). The scatter of data plotted in figure 2.8 suggests that pile capacity changes markedly with set up time, and that gains continue for several weeks after installation. It is interesting that the capacities at West Sole increased by 16.5% over a period of 5 days, as this rate exactly matches the trend line previously plotted through the results obtained from the API data base. To the writer, it would appear probable that gains in capacity continued for several weeks after testing and that the deduced shaft adhesions were not representative of the long term strengths. Reanalysis of the West Sole data continues, and one point of special interest is the modelling of one of the compression tests at 18.3 metres by Hobbs (1979), who carried out elasto-plastic finite element studies. In order to obtain reasonable agreement for displacements at working stresses the assumption $Eu = 500\ Cu$ was made in the soil mass, but this lead to overpredictions

of settlement at smaller loads. If the process of post-installation equilibration was indeed incomplete, a stiffer response would be expected after full consolidation.

2.4.12 In summary, the API code recommends that alpha be taken as 0.5 wherever Cu exceeds 75 kpa, which applies equally to the soils in the two "North Sea" zones, A and B identified in 2.4.9. The interpretation made by Semple (1984) suggests that these rules give estimates of the shaft friction mobilised in typical pile load tests which would be conservative in both zones by 10 to 30%. If allowance was made for a set up period of months, rather than days, the margins might increase considerably. However, such conclusions can only be tentative as the data base does not adequately cover the anticipated site conditions and contains many inconsistencies regarding test details and site investigation procedure. As the interpretations of the data base do not conclusively identify the factors controlling pile shaft capacity, extrapolations of the apparent trends for North Sea Platform design cannot be considered reliable.

2.5 RESEARCH FOR NORTH SEA PILED FOUNDATIONS

2.5.1 The difficulties of pile design in the North Sea have been widely recognised by the offshore industry, and development has been sponsored in three main areas of application;

(i) Improvements in site investigation procedures,
(ii) Instrumented studies of working piles on North Sea platforms, and on-shore trial piling programmes.
(iii) Improvements in analytical procedures.

2.5.2 It is self evident that good site investigations are important. Significant developments have been made in commercial offshore practice, as summarized by McClelland (1975), Andresson et al (1979), Zuidberg and Windle (1979), St. John (1980), Henderson et al (1979) and Toolan (1983). Perhaps the most important development in laboratory work has been the drive towards carrying out a high proportion of tests on board the site investigation vessel. Conventional methods for the determination of index properties, strength

and compressiblity appear to have been retained for most projects; Bjerrum and Landva (1966)1, Bjerrum (1973) and Sullivan et al (1979).

2.5.3 Naturally, the main object of the commercial investigations has been to obtain, where it is possible, the parameters required for the empirical methods of pile analysis. However, improvements in techniques can give misleading results. For example, if sampling quality improves and confined triaxial tests replace unconfined compression tests the shear strengths determined for a particular normally consolidated clay layer might be expected to increase. Without reducing the assumed value of α, this improvement might lead to an overestimate of axial capacity.

2.5.4 Three instrumented studies have been made of the behaviour of piled foundations in the North Sea. The first consisted of measurements made on a working pile from the FD structure in British Petroleum's Forties Field. Sutton et al (1979) present records of the radial total stresses and pore pressures developed at four locations on the pile shaft during, and shortly after, installation.

2.5.5 The second study was the more comprehensive Foundation Monitoring Project (FMP) for BP's Magnus Platform; Offshore Research Focus (1981). The project included the monitoring of pile shaft and group load distributions, mud-mat pressure measurements and recordings of dynamic displacements of the pile group under storm loading. The writer developed settlement gauge systems for the FMP and much of the laboratory testing described in this thesis was carried out to help understand the behaviour of the Magnus Platform's foundations.

2.5.6 The monitoring of the load-displacement characteristics of the pile groups of Conoco's Hutton Tension leg platform constitutes the third of the instrumented field studies. This work has been reported in Offshore Engineer (1984)2, and was carried out in association with Imperial College and the writer. The site details, a description of the field work and a preliminary interpretation of the data are presented in Chapter 11.

2.5.7 Field studies have also be carried out on driven piles installed at on shore test-bed sites. Gallagher and St. John (1980)

present preliminary results of cyclic and static loading tests on instrumented tension piles installed in Boulder Clay at Cowden. Peuch and Jezequel (1980) report similar studies carried out in France on two piles installed in sand-silts and soft clay respectively. Other non-instrumented onshore trials are detailed by Heerema (1979), Cox and Kraft (1979), Heins and Barends (1979) and Rigden and Thorburn (1976). Although useful, the test piles and site details for these trials differ in important ways from the foundation conditions of most North Sea Platforms. Several records of instrumented studies of pile driving and field horizontal load tests have recently been presented, e.g. Ransche (1978) and Odone et al. (1979)

2.5.8 Numerical and analytical methods have advanced over the same period. Elastic solutions have been extended to consider the response of groups of compressible piles to inclined forces and eccentric loading, Butterfield and Ghosh (1979). Simplified approximate closed form elastic solutions have been found by Randolph and Wroth (1978). Non-linear elastic properties can now be used in analysis, Desai (1974) and Lopes (1979), and allowance for pile slip after peak capacity can be combined with linear elasticity for pile group analysis, Poulos (1979). Elastoplastic soil behaviour has been considered in analyses of pile loading by Hobbs (1979) and Baguelin and Frank (1979). Toolan and Horsnell (1979) have combined the 'T-Z' types of analysis with linear elasticity in an attempt to improve pile group predictions.

2.5.9 Smith (1979) carried out a survey of numerical methods in offshore piling and concluded that analysis had advanced further than the ability to evaluate the required soil parameters. There remained, for example, no fundamental basis for the selection of parameters such as Eu, Z_c or α.

2.6 SUMMARY

2.6.1 The empirical relationships derived for calculating pile capacity and displacement at working load have been discussed in this chapter. The absence of a theoretical framework, or an understanding of the processes in terms of effective stress, leads to severe problems when extrapolati ng the results of small scale on-shore tests to

consider the design of large offshore structures. Considerable achievements have been made in the areas of field measurement, pile testing, site investigation technique and numerical methods of analysis. However, it would appear that few improvements can be made in the understanding of pile-soil problems without reference to the central principles of soil mechanics. The next chapter therefore discusses the developments towards a general effective stress theory for pile behaviour.

CHAPTER 3

DEVELOPMENTS TOWARDS A GENERAL EFFECTIVE STRESS THEORY OF PILE BEHAVIOUR

3.1 INTRODUCTION

3.1.1 This chapter considers the components required for a general effective stress theory of pile behaviour. A review is made of recent research in soil properties and pile-soil interaction, and existing theories are discussed. Published laboratory and field data are used to test the various analytical predictions and, in some cases, fresh interpretations are made of experimental results. Attention is concentrated on displacement piles installed in clay.

3.1.2 Chapter 2 discussed the difficulties of making significant improvements in the empirical methods of pile analysis without embarking on large field testing programmes designed to match prototype conditions. It is clear that the development of a fundamental, effective stress, theory for pile behaviour would be of great value. If pile-soil behaviour was properly understood existing experience could be extrapolated and allowances made for particular site conditions.

3.1.3 The starting point for any general effective stress theory of pile behaviour is the division of the problem into five areas;

(i) The General stress-strain and strength properties of soils.
(ii) The evaluation of initial ground conditions.
(iii) The process of pile installation.
(iv) The equilibration of the soil after pile installation.
(v) The process of pile loading from fixed initial conditions.

This approach has been adopted by many workers including Esrig and Kirby (1979), Randolph and Wroth (1982), Martins (1983), Francescon (1983) and Morrison (1984).

3.2 INVESTIGATIONS INTO THE GENERAL STRESS-STRAIN AND STRENGTH PROPERTIES OF SOILS

3.2.1 To carry out analyses of pile-soil behaviour it is necessary to describe, as completely as possible, the general elemental behaviour of the foundation materials. Observations of the response of soil to the load or strain are generally only feasible in laboratory tests. The constraints of the apparatus limit the possible stress or strain paths that can be followed, and the types of soil samples that can be tested. Most designs of apparatus produce stress or strain non-uniformities within test specimens; Wood (1980), Hight (1983).

3.2.2 Investigations of the general behaviour of clays are most frequently carried out using triaxial, plane strain and simple shear experiments. The state of stress in most simple shear apparatus is indeterminate and tests can be difficult to interpret in terms of effective stress, unless specially instrumented equipment is employed, such as the Cambridge cell used by Borin (1973) for his tests on kaolin.

3.2.3 Tests in the triaxial apparatus are usually limited to stress paths where the two radial stresses are equal, and the direction of major principal stress is either vertical or horizontal. Bishop and Wesley (1975) described a hydraulic conventional triaxial apparatus which is convenient for anisotropic consolidation and stress path testing. In special, true triaxial, apparatus the intermediate principal stress can be varied, see for example Wood (1975).

3.2.4 Plane strain apparatus is generally more difficult to use than triaxial equipment. Many designs, such as that used by Atkinson (1973), cannot be used for stress paths in which σ_v is less than either horizontal stress.

3.2.5 A new hollow cylinder apparatus has been developed at Imperial College which can independently vary the magnitude of all three principal stresses and also rotate the inclination of σ_1' in a vertical plane (Hight (1983)). The Directional Shear Cell designed at University College, has recently been developed to provide similar capabilities, Arthur et al. (1977). To date, however, detailed investigations with these equipment have been restricted to sands. In

summary, the types of loading that can be considered in laboratory tests on elements of clay are somewhat restricted, and are unlikely to give exact models for pile-soil problems.

3.2.6 Gens (1982) presents the results of a literature search for general studies of the monotonic behaviour of clays in the form shown in Table 3.1. The Atterberg limits, testing apparatus, type of consolidation, drainage conditions and stress orientation are tabulated[1]. Inspecting the columns, it can be seen that there are few entries where the behaviour of an anisotropically consolidated clay at various overconsolidation ratios has been assessed, and fewer still that have employed more than one stress path. The investigations with Lower Cromer Till (LCT), Spestone Kaolin, Weald Clay and Boston Blue Clay (BBC) provide by far the most comprehensive studies. As a consequence of the difficulties of obtaining large quantities of uniform intact material, all four groups of tests were carried out with reconstituted soil.

3.2.7 The results of some of the earlier studies on isotropically consolidated soils e.g. Henkel (1960) and Loudon (1967), were central in the development of Critical State Soil Mechanics (CSSM), Schofield and Wroth (1968). These ideas were developed into complete mathematical formulations such as the Modified Cam Clay model, Roscoe and Burland (1968). When combined with numerical solution routines CSSM models can provide powerful tools for the investigation of boundary value problems, Wroth (1976).

3.2.8 Whilst later experimental studies, including those employing anisotropic consolidation, reinforced many of the ideas put forward in critical state soil mechanics, a number of contradictions have become clear. The study of LCT by Gens (1982), provides the most comprehensive body of data with which to review the application of CCSM to low plasticity clays. Accordingly, his main conclusions regarding stress-strain and strength properties are summarized in the following sections.

3.2.9 Gens found a family of straight and parallel w/c - log p' lines by consolidating samples from slurry at various constant stress ratios ($\sigma'_3/\sigma'_1 = K$). For each value of K a constant ratio of axial to volume strain was found. Plots of w/c against log p' during swelling were found to be non-linear.

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[1] Some of the references given in Table 3.1 are not fully detailed in this Thesis. For further details, see Gens (1982).

3.2.10 Comparisons between samples monotonically consolidated from slurry, and samples tested after sampling from a prepared block, showed that reconsolidation to approximately 1.8 times the previous maximum pressure was required before the effects of sampling and preparation method were eliminated. It was also demonstrated that the behaviour of the clay in similar tests at equal OCR's, but different maximum consolidation pressures, can be normalised over a wide range of stress.

3.2.11 Well defined ultimate conditions were found which primarily depended on water content. Similarly well defined peak deviator conditions, which did not always coincide with the ultimate conditions, were found in compression tests on anisotropically consolidated samples. For normally and lightly over-consolidated samples very pronounced undrained brittleness was observed in anisotropically consolidated triaxial tests.

3.2.12 The angle of friction at the ultimate condition was found to be close to 30^0 in all tests. The ultimate states were generally developed at large strains and it was difficult to assess whether these conditions exactly corresponded to critical states as defined by Scholfield and Wroth (1968). The effect of σ_2' was investigated with triaxial compression and extension tests, and plane strain compression tests, all on isotropically consolidated samples. The stress paths for normally consolidated soil gave very similar shapes in J - p' space, with the common path being truncated by the different values of J/p' which correspond to $\phi = 30^0$ for the three values of b, where $b = (\sigma_2 - \sigma_3)/(\sigma_1 + \sigma_3)$. (Failure in plane strain generally gave values of b less than 0.5). The three stress strain curves were also initially similar but diverged as their respective ultimate values of $(\sigma_1 - \sigma_3)$ were approached. For each type of test a distinct w/c - log p' line was found for the ultimate conditions, and for triaxial compression tests this line was independant of the initial consolidation stress ratio, K.

3.2.13 Anisotropically consolidated samples tested in triaxial compression and extension showed greater divergences in behaviour than isotropically consolidated samples. A division of the strength differences into a peak effect, a b-effect and a critical state anisotropy was proposed. The pre-yield stress strain characteristics were non-linear, and the values of the stiffness parameters were found to depend

on the size of the stress increment $\Delta\tau/Cu$, OCR, consolidation type and test stress path. Particularly high values of the stiffness parameter Eu/Cu were found from the initial portions of triaxial compression tests on lightly over-consolidated samples. The ratio tended to rapidly reduce with increasing OCR in a similar way to the general trends summarized by Ladd et al. (1977).

3.2.14 The tests showed that, on the wet side, there were difficulties with the definition of the State Boundary Surface (SBS). Gens (1985) notes that as usually defined in CSSM the surface has two distinct roles;

a) to provide a boundary between possible and impossible states,

b) to be the locus for all possible void ratio-effective stress states for soil undergoing yield.

As shown in Figure 3.1, Gens' experiments with drained stress probes at constant values of K gave yield points which defined the most extensive area of possible normalised stress space. However, the normalised undrained stress paths for isotropically and anisotropically normally consolidated samples fell well inside this space as shown in Figure 3.1, a and b. If the first part of the SBS description is retained, and the surface is defined by the yield points of the constant K tests, it can be seen that zones exist within the SBS in which the soil behaves plastically. The undrained stress paths were shown to provide boundaries where yield occurred if a stress path traversed from an elastic region to a plastic zone. On the 'dry' side the SBS could be well represented by a single Hvorslev type surface.

3.2.15 The observed differences between the shape of undrained stress paths for isotropically and anisotropically normally consolidated samples of LCT, and the stiff response of lightly over consolidated clay to shear are striking. However, similar results can be seen in most clays where sufficient tests are carried out to make the comparison; Ladd et al. (1971), Parry and Nadarajah (1974) and Koutsoftas (1981). As noted by Gens (1982) the Lower Cromer Till is a clay of particularly low plasticity with LL = 25% and P.I. = 12%. In comparison with more plastic soils it would be expected to show low compressibility, high angle of friction, strong anisotropy, different peak and critical state angles of friction and higher shear stiffness, Ladd et al. (1977).

3.2.16 The studies reviewed here have considered only materials with the simple stress history of consolidation from slurry, possibly followed by monotonic overconsolidation. This stress history may be appropriate for clays sedimented through water, but does not model the genesis of many glacial soils, Hight (1983). Lodgement tills for example, can be deposited with moisture content near the plastic limit. Hight and Gens (1979) showed that such materials can show stress strain behaviour which is comparable with reconstituted soil when either remoulded or overconsolidated to appropriate inital conditions.

3.3 RESIDUAL FABRIC INVESTIGATIONS

3.3.1 The formation and properties of residual shear surfaces are areas that are generally not considered within critical state soil mechanics, and yet have importance in the understanding of pile-soil behaviour. Lupini (1981) and Lupini et al. (1981) discuss how polished surfaces with reduced angles of friction can form after large shear strains have developed in thin zones between displacing soil masses. They consider three types of shear under such conditions; turbulent shear in predominantly granular clays, sliding shear with reoriented surfaces in more plastic soils and transitional behaviour between these limits. Lupini's ring shear tests on LCT showed turbulent behaviour without reorientation, even after large displacements.

3.3.2 Martins (1983) noted that the ultimate skin friction of piles could be mobilised with failure developing at the pile-soil boundary, and gives a review of previous studies of shearing at interfaces between soil and structural materials. He also described experiments in a direct shear box where kaolin was displaced past a polished epoxy resin interface. Residual conditions were reached after far smaller movements than are normally required in soil-soil tests. He also postulated that low residual angles might be found when shearing against smooth interfaces with soils that exhibit turbulent shear in soil-soil tests.

3.3.3 Lemos (1985) has recently carried out a comprehensive programme of soil-soil and soil-interface tests in the ring shear apparatus. He varied soil type, interface roughnesses, rate of shearing and extent of displacement in order to study aspects of shaft friction.

Figure 3.2 shows the development of stress ratio τ/σ'_n with displacement, for LCT shearing against a polished steel interface. The peak angle of friction (14.6^0), mobilised at 0.3mm displacement, is far lower than the usual soil to soil angle of 30^0. These 'peak' conditions are followed by a slight reduction in strength, giving a minimum of 13.5^0 at 6.3mm displacment. With continued shearing the limiting friction angle climbed steadily until ϕ' stabilised at about 31^0, when the displacements had reached 12 metres. Electron scanning micrographs have been used to examine the way in which a thin sorted layer initially forms at the surface of the steel, but is disrupted as mobile sand grains continually bombard the interface. The limiting angle of friction can be seen to depend at each stage on complex interactions between soil and interface.

3.3.4 Martins (1983) demonstrated that the behaviour of soil-interface residual fabric is important in understanding shaft friction, particularly for driven piles. Lemos (1985) shows that there is a rapid transition in interface behaviour between low plasticity clays such as LCT, and more plastic soils. Lemos carried out ring shear soil-soil and soil-interface on the two soils considered in detail for this thesis, and the results are discussed in Chapters 6 and 7.

3.4 THE EVALUATION OF INITIAL CONDITIONS

3.4.1 When considering the initial foundation conditions at a site the two most important features are;

(i) The sequence of the soil layers and their geological history,

(ii) The insitu stresses and pore pressures.

3.4.2 For sedimented clays the geological history can often be evaluated by studying the soil fabric and the profiles of maximum effective overburden stress determined from oedometer yield points. Variations in the ratio C_u/σ_v' with depth can be found from the undrained shear strength profile, and this ratio is sometimes used to calculate OCR - depth relationships by using published curves. For simple stress histories, values of K_o might be evaluated from the graphs of Brooker and Ireland (1965) which plot K_o in terms of PI and OCR, the formulae proposed by Wroth (1972,1975), or the expression given by Mayne and Kulhawy (1982):

$$K_o = (1 - \sin \phi') \, OCR^{\sin \phi'} \qquad \text{Eq. 3.1}$$

Gens (1982) found Eq. 3.1 to give close agreement with his data for LCT.

3.4.3 The vertical stresses are, of course, derived from careful measurements of bulk density. Hydrostatic pore water pressures are usually assumed, although in under-consolidated offshore conditions or onshore sites with underdrainage, non-hydrostatic distribution may prevail.

3.4.4 It is more difficult to evaluate the initial conditions for soils which were not sedimented through water or have complex stress histories. Oedometer tests and C_u/σ_v' ratios can be used to calculate apparent degrees of overconsolidation which reflect the position of the current w/c - log p' state in relation to a virgin consolidation or critical state line. The analogy with literal overconsolidation is useful for certain types of behaviour, but does not justify the use of apparent OCR to calculate K_o values from the sedimented clay test curves.

3.4.5 In situ test techniques have been developed to measure σ_h, Wroth and Hughes (1973), Baguelin et al. (1978), Tedd and Charles (1983), but as yet these tools are not available for use offshore. Investigatons in lodgement and other tills at inland sites indicates K_o values which are lower than those calculated using the apparent degree of overconsolidation. Figure 3.3 shows an interpretation of the data presented by Powell et al. (1983) for the Cowden site. Lines corresponding to values of K_o between 3 and 0.5 are indicated on the figure; the trend of the results is for K_o to initially rapidly reduce with depth, until a steady value of 1.0 to 1.5 is obtained between 5 and 12 metres. These observations are supported by other tests carried out by Clarke (1984) and are in general agreement with the arguements of Hight (1983).

3.5 THE PROCESS OF INSTALLATION FOR DRIVEN PILES

3.5.1 For simplicity the installation of a driven pile will be first considered as a pseudo-static process which is equivalent to steady jacking.

3.5.2 When a solid pile is jacked into soil the advancing tip produces a zone of intensely sheared soil close to the shaft, an intermediate

volume where plastic strains are induced and an outer zone where the soil does not reach yield. This sequence is sketched in Figure 3.4. With saturated clays, the installation process is usually assumed to be undrained and the pile volume is accommodated by displacements at the boundaries. If the clay layer is deep and wide, the main constraint on the patterns of straining will be the distance to the free upper surface. Cooke, Price and Tarr (1979) made measurements of ground heave around jacked piles, from which they were able to calculate the ratio of heave volume to inserted volume as a function of pile penetration, see Figure 3.5. As the limit of integration for surface heave volume was taken as a radius of three metres, their data can be seen as a lower bound to a possibly larger total.

3.5.3 Measurements of the patterns of soil displacement around jacked piles have been made with laboratory models and field tests. Figures 3.6 and 3.7 give summaries of the displacements observed at approximately mid pile depth by Cooke and Price (1973), Randolph et al. (1979) and Francescon (1983). Figure 3.8 shows the soil trajectories given by Randolph et al. (1979) for a spread of radiating points on an initially horizontal reference line undergoing displacement caused by pile installation. It can be seen from Figure 3.6 to 3.8 that only material within 2 radii of the pile centreline experiences permanent vertical movements. In contrast, Figure 3.7 shows radial displacements which persisted up to 10 radii from the centre of the pile.

3.6 CYLINDRICAL CAVITY EXPANSION AS A MODEL FOR PILE INSTALLATION

3.6.1 Randolph et al. (1979) considered the experimental data to justify the assumption of plane-strain, cylindrical cavity-expansion as a model for the installation of displacement piles. Soderberg (1982) and Butterfield and Bannerjee (1970) had already carried out detailed analyses with this theory, and solutions for cavity expansion in elastic-plastic solids had been obtained by a number of workers including Bishop et al. (1945) and Gibson and Anderson (1961). The analyses predict that the creation of a cavity of internal radius r_o results in the formation of an annular zone of plastically deforming soil to an outer radius R within the otherwise elastic continuum, see Figure 3.9. A number of alternative expressions exist for the distributions of total stresses and pore pressure changes in the surrounding

soil. Chodorowski (1982) made a useful review of the assumptions of each analysis and gives an evaluation of the solutions for a range of soil conditions.

3.6.2 The assumption of simple elastic-plastic soil properties infers that the ground stresses are only influenced by radius from the cavity centre and the ratio of elastic stiffness to shear strength. Thus, for example, the simplified solution of Randolph and Wroth (1979) predicts;

$$\Delta\sigma r/Cu = 2 \ell n \frac{R}{r} + 1 \qquad \text{in the plastic zone} \qquad \text{Eq. 3.2}$$

$$\text{with } \Delta u/Cu = 2 \ell n \frac{R}{r} \qquad \text{in the same region} \qquad \text{Eq. 3.3}$$

$$\text{and } \Delta\sigma r/Cu = \left(\frac{R}{r}\right)^2 \text{ with } \Delta u = 0 \qquad \text{in the elastic zone} \qquad \text{Eq. 3.4}$$

$$\text{in each case } \frac{R}{r_o} = \sqrt{\frac{G}{Cu}}$$

3.6.3 It can be seen that the expressions can take no account of contractant or dilatant soil behaviour. Some of the formulations make provision for more realistic pore water pressure responses by including an empirical deviatoric pore pressure parameter such as Henkel's α_f.[1] For example, the expressions of Gibson and Anderson (1961) can be written as;

$$\Delta u/Cu = 2 \ell n \frac{R}{r} + 0.81 \alpha_f \qquad \text{in the plastic zone} \qquad \text{Eq. 3.6}$$

$$\text{and } \Delta\sigma r/Cu = 0.81 \alpha_f \left(\frac{R}{r}\right)^2 \qquad \text{in the elastic zone} \qquad \text{Eq. 3.7}$$

$$\text{where } \frac{R}{r_o} = \sqrt{\frac{E}{2 (1 + \nu) Cu}} \approx \sqrt{\frac{G}{Cu}} \qquad \text{Eq. 3.8}$$

Considering α_f values of -1 for a heavily overconsolidated clay and 3 for a normally consolidated condition, it can be seen that the expressions predict a marked sensitivity of pore pressure to OCR.

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[1]For triaxial compression $\alpha_f = 3Af - 1$

3.6.4 Randolph et al. (1979) investigated the same problem using Finite element analysis and a variant of the modified Cam clay model. Amongst other assumptions the authors considered;

(i) There is a single value[2] of q/p' for critical state conditions, which is not compatible with a unique Mhor-Coulomb angle ϕ'. For triaxial compression,

$$q/p' = M = 6 \sin\phi' / 3 - \sin\phi' \qquad \text{Eq. 3.9}$$

Whilst for plane strain conditions, it is assumed that $\sigma_2' = \frac{1}{2}(\sigma_1' + \sigma_3')$, and q/p' is taken as:

$$q/p' = M = \sqrt{3} \sin\phi' \qquad \text{Eq. 3.10}$$

(ii) No Hvorslev type surface is required as a yield function on the 'dry' side.

(iii) That the soil has a linear pre-yield of shear modulus, G. However, G was varied in calculations starting from different initial stresses and G/Cu was assumed to steadily increase with OCR.

Reference to Section 3.2 will show that in these three areas the assumptions contradict the results of recent fundamental research.

3.6.5 The numerical solution involved enlarging a small existing cavity using finite strain routines. The stresses near the pile face were obtained by extrapolating the results from radial distances greater than 1.15 r_o. Parameters were assumed for Boston Blue Clay and predictions made over a range of overconsolidation ratios for;

(i) The stress paths of elements close to the 'pile' face, expressed in terms of σ_r', σ_θ' and σ_z'.

(ii) Radial distributions of the changes in the stresses and pore pressures at the end of cavity expansion.

3.6.6 Wroth et al. (1979) and Francescon (1983) supplement these predictions of BBC with studies of London Clay and kaolin, and conclude that the stresses adjacent to the 'pile' can be computed from the simplified expressions given by Randolph et al. (1979);

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[2]Alternative assumptions can be made with Modified Cam Clay, See Appendix 2.

$$\sigma_r'/Cu = \sqrt{3}\,M + 1 \qquad \text{Eq. 3.11}$$

$$\sigma_\theta'/Cu = \sqrt{3}\,M - 1 \qquad \text{Eq. 3.12}$$

$$\sigma_z'/Cu = \sqrt{3}\,M \qquad \text{Eq. 3.13}$$

Table 3.2 evaluates these formulae for Boston Blue Clay, London Clay and kaolin. Randolph et al. (1979) and Wroth et al. (1979) alternatively define M from triaxial and 'plane strain' conditions, and it is not clear which should be substituted into equations 3.11 to 3.13. Values are therefore given for both cases, assuming an invariant ϕ'. The 'plane strain' values correspond most closely with the plots presented by the authors, but is can be seen that the stresses are sensitive to the choice of M. This is particularly so for σ_θ' and p', the mean effective stress.

3.6.7 Whilst the analysis shows the effective principal stresses to depend only on M, the pore water pressures are also effected by G/Cu and (p'f - p'o). Both parameters are assumed to vary with OCR and Figure 3.10 plots the results found with the considered soils.

3.6.8 Figure 3.11 compares the initial K_o relationships between σ_r'/Cu and OCR with those predicted after pile installation. In all cases except normally consolidated kaolin, increases in σ_r' are calculated. With Boston Blue clay at overconsolidation ratio 8, σ_r' is predicted to rise by around 400%.

3.6.9 Figure 3.12 illustrates the radial variations in stresses predicted by the Cambridge cavity expansion solution for normally consolidated Boston Blue Clay. The extent of the boundary to the critical state region, and the stresses in the elastic zone depend on the assumed G/Cu ratio. The normalised stresses in the elastic region are sensitive to OCR and generally reduce with increasing overconsolidation, so that in some cases σ_θ' becomes tensile.

3.6.10 To save repetition in later sections, the predictions of the cavity expansion theory developed by Randolph, Wroth and Carter will be referred to as the 'Cambridge theory'.

3.7 STRAIN PATH SOLUTIONS FOR PILE INSTALLATION

3.7.1 An alternative approach to cavity expansion theory is the strain path method which has been developed at MIT. The essence of the solution is to independently obtain the complete velocity field around a cone or pile as it is installed in a soil layer. The solutions are for deep penetration only, and are used to find the strain paths experienced by points within the soil mass. The method is fully described by Levadoux and Baligh (1980), and the uncoupling of strain and stress fields considerably reduces the problems of the two dimensional numerical analysis.

3.7.2 Baligh (1975) showed the displacement fields for problems such as cavity expansion to be independent of the properties of the surrounding material. Neglecting inertial and viscoelastic effects, steady penetration is then reduced to a flow problem where soil particles move along streamlines around a fixed rigid body. By combining the hydrodynamic solution for a single source with that for uniform flow he was able to simulate the penetration of a smooth 'simple pile' through a deep incompressible homogenous, isotropic, soil layer. Baligh (1984) attempts to show that this problem is also independent of material properties.

3.7.3 The velocity fields obtained from the streamlines can be integrated to calculate strains and movements. The predictions for displacements caused by an advancing pile tip are shown in Figure 3.13 and the strain paths experienced by three points are given in Figure 3.14. Baligh uses three strain axes; E_1, E_2 and E_3. These are defined on Figure 3.15 and represent vertical strain, cylindrical expansion strain and simple shear strain respectively. In cylindrical cavity expansion theory E_1 and E_3 are neglected, and only a monotonically increasing E_2 strain is considered. The strain path solution shows soil near the centre line of the pile (element G) to be experiencing axial strains sufficient to induce failure in compression before being pushed aside. Points A, F and G all show considerable reversals of axial and simple shear strain. Baligh and his co-workers maintain that such features are important in understanding the state of stress surrounding a pile after installation.

3.7.4 The variations of octahedral shear strain given in Figure 3.15 show a similar distribution with radius to those derived from cavity expansion theory. Shear strains which are greater than those required to cause yield in normally consolidated LCT persist to the limits of the contour diagram.

3.7.5 Levadoux and Baligh (1980) derived the strain path method to calculate the variations of soil stresses and pore pressure around a penetrating smooth cone or pile. Their solution is summarized in the flow chart given in Figure 3.16. Two soil models are required in the calculation routine, the first for deviatoric stress-strain relations, the second for shear induced pore pressure changes as functions of strain.

3.7.6 The penetration is assumed to occur without drainage and Levadoux and Baligh (1980) chose to retain a total deviatoric stress-strain model. The main features of the model centre on attempts to reproduce features of non-linearity, anisotropy and strain softening found in laboratory tests on Boston Blue Clay (BBC). Anistropically consolidated BBC has many common features of stress-strain and strength behaviour with Lower Cromer Till. In particular, K_o normally consolidated samples show initially stiff, then highly brittle behaviour in compression, with quite different behaviour in extension. The deviatoric stress-strain model assumes that plastic strains can develop from even small stress changes and considers a nested set of yield surfaces bounded by a failure surface. The surfaces are assumed to be of the Von Mises kind and are governed by hardening rules which depend on the plastic flow undergone. The anisotropy is modelled by centering the yield surfaces at a position off the stress space diagonal.

3.7.7 The model uses curve fitting techniques to find strain softening rules for post-peak behaviour in compression, but assumes perfect plasticity for extension modes. The formulations were tested by comparing predictions of Cu/σ'_{vo} with laboratory test data for triaxial plane strain and simple shear tests. A summary of this comparison is given in Table 3.3 which also shows the ratios predicted by Levadoux and Baligh (1980) using a variant of Modified Cam-Clay (MCC), similar to that employed by Randolph et al. (1979). For each entry, an error from the corresponding laboratory result is quoted.

3.7.8 It will be seen that neither soil model is completely successful. The assumption of an off-centre Von Mises criterion produces difficulties when σ_2 varies between σ_1 and σ_3, in a similar way to the MCC variant used in the Cambridge studies. As the MIT model is based on curve fitting and employs more parameters, the finding that its predictions give slightly less error is not surprising.

3.8.9 The pore pressure-strain model was similarly derived from laboratory data and relates shear induced pore pressure to octahedral shear strain. When combined with the stress-strain formulations good agreement is found over the first few phases of cyclic triaxial tests. This is important as the MIT solutions indicate strain reversals during penetration.

3.7.10 Levadoux and Baligh (1980) suggest two means of estimating the accuracy of their solutions. The first is to calculate the total stresses by integrating the solutions using different routes. If the method is perfect no differences should be seen. The second way is to compare predictions with field or laboratory measurements. In fact, significant differences were found between solutions integrated along stream lines and those integrated along isochrones. The authors argue that the latter method should be favoured.

3.7.11 To further evaluate their work Levadoux and Baligh (1980) computed solutions for cavity expansion in normally consolidated BBC. They contrasted their results with those published by Randolph et al. (1979), and found;

(i) Steeper curves of pore pressure and radial stress with cavity volume strain,

(ii) Similar values of limiting total radial stress at the cavity boundary, but different distributions of effective stresses.

are summarized in Table 3.4.

3.7.12 Comparisons were then made between both sets of predictions and the results of a field self boring pressuremeter test carried out in lightly overconsolidated BBC. Here the differences were considerable, with the field data giving a normalised limiting pressure σ_r/σ_{vo}', approximately twice the result calculated from the two models. Various explanations are suggested including the low ratio of length to diameter

of the device employed, strain rates, overconsolidation and partial drainage. The soil at the test site was assumed to be normally consolidated, although oedometer data suggest an OCR of 1.35. If the clay was indeed overconsolidated to this degree, the theoretical results would have underpredicted the pressure displacement curves by about 30%.

3.7.13 Levadoux and Baligh (1980) present solutions for the state of stress and pore pressure around long smooth penetrometers passing through a deep layer of normally consolidated BBC. The variations of pore pressure and radial effective stress for 18^0 and 60^0 cones are reproduced in Figures 3.17 and 3.18. The stress and pore pressure conditions are further summarized in Tables 3.5, and can be seen to vary considerably with distance from the tip. However, for points further than 10 r_o above the apex the stresses tended to assymptotic values which are independent of cone angle.

3.7.14 The strain path method has been revised by Kavvadas (1982) who substituted an anisotropic, critical state, effective stress model for the total stress and pore pressure-strain models described by Baligh and Levadoux (1980). The model was found to give improved predictions of the triaxial, plane strain and simple shear test data. Figure 3.19 a) and b) shows how the adoption of the effective stress model influences the calculation of σ_r' and Δu around a pile shaft, at a point far above the tip. The same plot shows the stresses calculated from the combination of the MIT strain field and the Modified Cam clay variant used by Randolph et al. (1979). It is apparent that the effect of the soil model is significant but not dominant at this stage. Further emphasis is given in Figure 3.19 c) and d), where a similar comparision is made for the case of cylindrical cavity expansion, Kavvadas (1982).

3.7.15 The results given in Figures 3.10, 3.19 and in Tables 3.2 and 3.4 permit a comparison of the two analytical approaches for points far behind the tip of a pile installed in normally consolidated BBC. The main differences are clear;

(i) The Cambridge cavity expansion theory predicts very much larger values of σ_r' and p' at the pile face, but smaller values of σ_z'

(ii) The strain path solutions predict smaller increases in pore water pressure.

As detailed in Appendix 2, alternative versions of the MCC model are available, and it is possible that different results could be obtained if, for example, the yield function was chosen so as to make ϕ' independent of σ_2'.

3.7.16 The strain path calculations show steep gradients in stresses and pore pressures near the tip, with the values of Δu and σ_r' exceeding those predicted on the shaft.

3.7.17 The Cambridge results have been expressed as ratios of stresses to Cu, but as strength can vary with testing method these ratios can be ambiguous. The MIT workers have preferred to use σ_{vo}', which is more easily obtained. However, in the following sections comparisions will be made with experimental data, and it will be convenient to temporarily retain the use of Cu.

3.8 TWO DIMENSIONAL FINITE ELEMENT SIMULATIONS OF PILE INSTALLATION

3.8.1 Fathallah (1978) and Bannerjee (1979) present accounts of two dimensional analyses of pile installation, which used the finite element method with both elastic-plastic and critical state soil models. The process of pile 'driving' was simulated with the finite strain procedure illustrated in Figure 3.20. The process starts with the mesh being pierced, as if by an infinitely sharp needle, and the sides of the resulting small cavity being pushed back in stages to form a conical void. The sequence is repeated for the next row of elements and the upper space is enlarged until it matches the diameter of the imaginary pile. These cycles are continued until the full penetration has been obtained.

3.8.2 The simulation thus amounts to a progressive cavity expansion in which the conditions vary with depth. Near the surface, heave can occur as only the boundary stresses are controlled. At greater depth, the installation will take place under approximately plane strain conditions. However, the simulation cannot be considered realistic as it does not consider the punching shear failure that takes place ahead of the tip as the pile advances. The strain paths are practically equivalent to cylindrical cavity expansion.

3.8.3 Fathallah (1978) carried out two studies of pile 'driving' using a Cam-Clay model (see Appendix 2). In the first he assumed a set of soil parameters similar to those found from isotropically consolidated triaxial tests on kaolin, with an invariant value of M. Installation into a bed of isotropically consolidated soil was modelled and predictions made for the variation of induced stresses and pore pressures. In the second study he attempted to simulate the procedure for his model pile test programme with;

(i) K_o consolidation from slurry to $\sigma_v' \approx 150$ kpa in one stage,
(ii) Unloading to $\sigma_v = o$, with partial swelling from the boundaries,
(iii) Pile installation over a relatively long period of time,
(iv) Reimposition of $\sigma_v = 150$ kpa
(v) Consolidation.

3.8.4 The swelling and consolidation portions of the sequence were considered using a coupled Biot analysis. However, neither of his two studies correspond to the case of instantaneous penetration into a K_o consolidated soil. Although Fathallah's calculations cannot be directly compared with the preceeding theoretical results, it is worth noting his prediction that installation into isotropically consolidated soil would increase σ'_r/Cu by between 1.2 and 1.4, and $\Delta u/Cu$ by between 4.5 and 5.4.

3.9 EXPERIMENTAL STUDIES OF THE EFFECTS OF PILE INSTALLATION

3.9.1 Laboratory and field investigation with jacked or driven piles can be considered according to to the test boundary conditions and the scope of the measurements made. Thus it is possible to list four main areas of investigation;

(i) Soil displacements (see Section 3.5).
(ii) Pore pressures in the surrounding soil.
(iii) Stresses and pore pressures acting on the pile face.
(iv) Changes in soil properties around the pile.

As soil displacements have already been discussed, attention will be concentrated on the remaining features. A review of the literature shows field and laboratory studies to have the contrasting advantages and disadvantages summarized in Table 3.6 and, to a certain extent, the two approaches are complimentary.

3.9.2 Many references can be found to studies of the response of soil layers to pile driving or jacking. Table 3.7 presents a selection of case records where pore water pressures were monitored, and Table 3.8 summarises references where both total radial stresses and pore pressures were measured. (Reference 4 does not strictly belong in this group as pore pressures were assumed rather than measured.)

3.9.3 Studies with normally consolidated, or lightly overconsolidated soils provide most of the case histories. Typical behaviour during and, just after, installation can be described by reference to Figures 3.21 to 3.23. Figure 3.21 shows measurements reported by Blanchet et al. (1980) who installed piezometers near the centre lines of two piles. As each pile approached a particular piezometer, pore pressures rose rapidly until the tip had passed and the soil had been displaced to the side. With futher penetration the pressures fell to lower values. Figure 3.22 indicates similar features recorded at the end of driving by Seed and Reese (1955). In this remarkably early experiment both radial total pressure cells and pore water pressure cells were installed on the side of a pile. The values of σ_r and u were observed to increase with depth giving an almost constant value of σ_r' immediately following installation. An impression of the radial distributions of pore water pressure change at the level of the tip, and at a higher level on the shaft, is given by Figure 3.23 which reproduces data from Blanchet et al. (1980).

3.9.4 Inspection of the records summarized in Tables 3.7 and 3.8 suggests that typical values for σ_r' and Δu acting against the shafts of piles just after installation in normally or lightly overconsolidated soil might be;[3]

$1.8 > \sigma'_r/Cu > o$ with a mean value of 0.6

and $9.5 > \Delta u/Cu > 3.5$ with a mean value of 6.3

It is not reasonable to give equal weighting to each of the entries in the tables, as the numbers of tests and the quality of the measurements vary widely. The model scale programmes of Francescon (1983) and Morrison (1984) present by far the most comprehensive investigations, and should be considered more reliable than, for example, the ambiguous data presented by Fathallah (1978).

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[3]The data of Fathallah (1978) have been excluded

3.9.5 The experimental trends can be used to assess the accuracy of the various analyses of displacement pile installation, through comparisons with the predictions for pore water pressures and radial effective stresses. Considering normally and lightly-overconsolidated clays first, it will be recalled that the cavity expansion theories gave the following values for kaolin and Boston Blue clay respectively;

σ_r'/Cu = 2.2 and 2.5

$\Delta u/Cu$ = 5.5 and 4.1

Francescon (1983) and Morrison (1984) obtained consistent results in tests with these two soils. From comparison with Table 3.8, it is apparent that the Cambridge cavity expansion results greatly overestimate σ_r' after installation. In both cases the experiments showed radial effective stresses which only achieved 10 to 25% of the theoretical predictions. In addition, the field measurements of pore pressure changes in Boston Blue clay were approximately double those calculated.

3.9.6 At present, strain path solutions have been published for Boston Blue clay alone. The predictions made with the effective and total stress MIT models for the stresses on the pile shaft in normally consolidated BBC can be summarised as;

$0.4 > \sigma_r'/Cu > 0$

and $3.7 > \Delta u/Cu > 3.3$

The estimates of σ_r' fall below the mean of the experimental values by around 0.3 Cu, but far larger differences are seen between predicted and measured pore water pressures acting on the shaft. However, the data of Baligh and Levadoux (1980) show that the shape of the pore pressure distributions acting over the pile can be predicted with reasonable accuracy. This is demonstrated in Figure 3.24.

3.9.7 Whilst the strain path method has yet to be applied to soils of OCR greater than 1.35, cavity expansion solutions have been given for overconsolidated soils. The Cambridge analyses showed σ_r'/Cu, σ_θ'/Cu and σ_z'/Cu at the pile face to be invariant, but for $\Delta u/Cu$ to vary gently with OCR. The limited experimental data show no tendency for these predictions to improve with increasing OCR, but indicate a noticable dependence of $\Delta u/Cu$ on overconsolidation ratio. These features are illustrated using Francescon's (1983) data in Figure 3.25 and 3.26. It is interesting that the single reference to field

experients with overconsolidated clays suggests that negative pore water pressures were developed at the pile faced during installation, O'Neil et al. (1982).

3.10 EQUILIBRATION OF THE SOIL MASS AFTER PILE INSTALLATION

3.10.1 After pile installation there follows a period during which pore pressures and effective stresses adjust to equilibrium values. The boundary conditions for this process are mixed; the soil is free to move at the unstressed ground surface and can strain radially outwards, but there are severe kinematic restraints near to the pile. For steel piles the interface is also a zero flow boundary. With these conditions the total stresses in the soil cannot be considered to be held constant during consolidation. Simple linear analyses of the problem must start with an assumption for the excess pore water pressure distribution, and for coupled analyses it is also necessary to know the initial effective stresses and the soil constitutive law.

3.10.2 Soderberg (1962) proposed the first analysis of the problem, assuming an initial pore pressure profile similar to that from cavity expansion theory and elastic behaviour during consolidation. Table 3.9 summarizes subsequent analyses of equilibration, which have also started from cavity expansion assumptions.

3.10.3 The first three references employed finite difference solutions, whilst Randolph et al. (1979) used finite elements. The non dimensional isochrones for the solutions are dependent on R/r_o, and hence G/Cu, but tend to be of similar shape after the initial stages of consolidation. As an example, Figure 3.26 compares the isochrone given by Hagerty and Garlanger (1972) at T = 1.0 with one interpolated from Randolph et al. The latter authors show that pore pressure dissipation is not greatly influenced by the assumption of elasto-plastic soil behaviour. This is equivalent to finding that the process is governed by the elastic swelling and recompression rather than virgin consolidation. A similar conclusion was reached by Nicholson and Jardine (1981) who contrasted field rates of consolidation for the same clay when undergoing embankment loading and equilibrating after the installation of driven vertical drains.

3.10.4 A major difficulty remains in the choices of the deformation parameters and the permeability k. As discussed in Section 3.2, the values of the elastic moduli G, K' and the permeability, k, are likely to vary considerably with the size of strain or stress increment. It is also well known that permeability can be sensitive to changes in effective stress, particularly for soft clays.

3.10.5 Although it is difficult to estimate overall Cv values for piling equilibration from simple laboratory tests, the curves given in Figure 3.27 provide a consistent basis for back calculating approximate linear Cv figures from field and laboratory measurements. It will be seen that time factors as large as 300 can be required for 90% consolidation of the pile face.

3.10.6 Randolph et al. (1979) used their finite element studies to investigate the dependence of the final effective stress-state on the assumed constitutive law. It was found that different stress paths were predicted by elastic, elastoplastic and modified Cam clay models. The final stresses were dependent on the assumed properties, and in particular on the ratio G/Cu. The authors considered that the critical state model gave the best solution and that their study justified the following conclusions for a range of initial conditions;

(i) That the radial effective stress at the pile face increases during consolidation by between 50 and 60% of the maximum excess pore pressure developed during installation.

(ii) That at the end of consolidation σ_r' is the major principal effective stress with σ_θ' and σ_z' equal minor principal effective stresses. In all cases $\sigma_\theta' = \sigma_z' \approx K_o \sigma_r'$, with K_o equal to the normally consolidated value.

(iii) That the water content changes near the pile imply an increase in Cu of 30 to 100%.

3.10.7 Table 3.10 summarises these conclusions for the same three soils considered in Section 3.6. As for cavity expansion, the stresses normalised by Cu turned out to be almost independent of overconsolidation ratio. The analyses predict large increases in radial effective stresses at the end of consolidation, particularly for overconsolidated clay. For such soils the calculated values of σ_z' are also likely to exceed the effective overburden pressures, but tensile circumferential effective stresses are predicted for $r \approx 10r_o$.

3.10.8 The coupled analyses presented by Fathallah (1978) also predicted large increases in σ_r' as a result of pile installation and equilibration. For the case of isotropic initial stress conditions the overall increase in σ_r'/Cu ranged between 5.1 near the surface and 6.4 at the tip. Only slightly smaller changes were predicted for σ_z'.

3.10.9 Kavvadas (1982) further developed the strain path analysis of pile installation to include the consideration of the consolidation phase, and devised the effective stress model referred to in Section 3.7.14 for this purpose. The model represents an extension of the critical state ideas to include anisotropic strain hardening and softening. The yield surface is initially centred on the K_o line, but its axes can be altered by deformation. The model also allows isotropic hardening with volume strain in a similar way to MCC. The plastic potential is not associated with the yield surface and separate peak and ultimate (i.e. critical state) strengths can be obtained. Failure conditions are represented in effective stress space by two, nested conical surfaces whose shapes can be altered to vary ϕ' with the type of loading. Kavvadas and Baligh (1982) used the new model to simulate Boston Blue Clay and found good agreement between their predictions and measurements for a range of laboratory tests. The model is, however, complex and requires a total of 12 material parameters.

3.10.10 Kavvadas (1982) considered the consolidation process in a similar way to Randolph et al. (1979). A disc of soil was taken as representing the conditions in a layer which is both far above the tip, and deep below the surface and only radial water flow was permitted. Plane strain conditions were assumed and a non-linear form of the coupled Biot analyses employed. The soil stress-strain properties were governed by the MIT effective stress model, a constant permeability was assumed and the initial conditions were taken from the strain path installation results. As with the MCC model, the time factor definition no longer included a stiffness term and was given by;

$$T = ct/r_o^2 \qquad \text{where } c = \left(\frac{1 + 2\,K_o}{3}\right)\left(\frac{\sigma'_{vo}\,k}{\gamma}\right)$$

3.10.11 Kavvadas (1982) was able to consider normally or lightly overconsolidated soils and Figure 3.28 shows his predictions for the

variations of σ_r' with T for Boston Blue Clay at three overconsolidation ratios. Figure 3.29 shows the radial distributions of σ_r', σ_θ' and σ_z' after full consolidation. Contrasting these plots with the results of the Cambridge analysis given in Table 3.10 identifies three main differences;

(i) Kavvadas (1982) predicts far lower effective stresses at the pile face. For an overconsolidation ratio of 1.35 a slight decrement in p' from the pre-installation condition is shown, and for OCR = 1.0 the final p' is less than half the inital value. The analysis does not therefore predict significant increases in strength or reductions in moisture content around the pile shaft.

(ii) The final major principal stress direction acting near the shaft is vertical, not radial.

(iii) The normalised values of σ_r' at the pile face are found to be highly sensitive to OCR, and with σ_r'/σ_{vo}' doubling between OCR = 1.0 and 1.35.

3.10.12 Kavvadas and Baligh (1982) present similar analyses with the MCC variant substituted for their effective stress model. The radial effective stress was found to be sensitive to the constitutive law, and for OCR = 1.0, the final value of σ_r'/σ_{vo}' was 3.25 times the value predicted using the MIT effective stress model.

3.10.13 The studies of pore pressure dissipation did, however, show an important point of agreement with the cavity expansion analyses. It was found that the rates of pore pressure decay at the pile face were relatively insensitive to the choice of constitutive law, and that the process was largely governed by the elastic properties of the soil.

3.11 MEASUREMENTS OF CONSOLIDATION EFFECTS AFTER PILE INSTALLATION

3.11.1 Observations of the excess pore water pressures generated by the installation of displacement piles were reported in Section 3.9 and Section 3.10 discussed the analysis of the subsequent equilibration processes. As it will be necessary to evaluate times for various degrees of consolidation in later sections, coefficients of consolidation have been estimated from six of the case records. The elastic

solutions found by Randolph et al. (1979) were employed, via the curves given in Figures 3.26, to calculate the C_v values given in Table 3.11.

3.11.2 The soils considered cover a wide range of types, but the C_v values lie within the relatively small range of 2.5 to 46 m^2/yr. With the exception of two of the kaolin tests reported by Francescon (1983), all the cases considered normally consolidated or lightly overconsolidated initial conditions. Oedometer tests on such soils might be expected to show virgin compression Cv values in the range 0.2 to 15 m^2/yr, with recompression or swelling coefficients between 1 and 100 m^2/yr. (Lambe and Whitman (1969). The two references that report oedometer data reinforce this expectation. The values calculated from the test piles fall between the limits of the virgin compression and swelling. In the absence of other data a median value might be taken.

3.11.3 Interpreted in this approximate way the data shows a trend for C_v to fall with increasing OCR. As the dissipation is controlled by elastic parameters, this might be explained by the upward curvature of swelling W/C - log p' lines.

3.11.4 The most important feature of the consolidation process is the tendency for σ_r' to increase with time. A total of eight references have been found in the literature in which the variation of σ_r and u after installation have been reported. The experiments conducted by Fathallah (1978) are not considered in detail here. The remaining data are summarised in Table 3.12 using non dimensional forms, and again Francescon (1983) and Morrison (1984) provide the most comprehensive information. It is worth noting, however, that Francescon's model tests show larger final values of σ_r'/Cuo than all of the field studies at low OCR.

3.11.5 If slight extrapolations are allowed for the cases with incomplete consolidation, the trends in Table 3.12 can be summarised as follows;

(i) For the cases where OCR < 3 the ratios of $\sigma_{r\infty}'/\sigma_r'_I$ fall between 0.5 and 1.5 with a mean of 1.0. Similarly the values of $\sigma_{r\infty}'/Cu$ ranged between 0.7 and 4.0 with a mean of 2.2.

(ii) For the few cases where OCR > 3, the mean $\sigma_{r\infty}'/Cuo$ is calculated as 2.0 and $\sigma_{r\infty}'/\sigma_r'_I$ as 1.6.

(iii) The model test series of Francescon (1983) indicate a significant dependence of $\sigma_{r\infty}'$ on the test apparatus with his A series tests giving results 15 to 30% larger than the B series.

3.11.6 The cavity expansion predictions for σ_r' given in Table 3.10 therefore appear unrealistic. The theory overestimates the final stresses in the Boston Blue Clay by a factor of 3, and those in the Kaolin by factors between 1.5 and 2.5. Kavvadas' solutions shown on Figures 3.29 and 3.30 give better agreement for the lightly overconsolidated soil, with predictions of $\sigma_{r\infty}'/\sigma_{rI}'$ between 0.7 and 1.3, and $\sigma_{r\infty}'/Cu$ between 1.5 and 2.0. There is however, little evidence of the expected sensitivity of $\sigma_{r\infty}'$ to overconsolidation ratio.

3.11.7 The measurement of $\sigma_{r\infty}'$ gives an incomplete description of the state of stress near the pile shaft after equilibration, and it is difficult to devise means of obtaining σ_θ' and σ_z'. However, clues can be found from the post-consolidation measurements of undrained shear strength and moisture content for points close to displacement piles. Such data have been reported by a number of workers including Reese and Seed (1955), Bannerjee (1979), Martins (1982) and Francescon (1983). The calculated or measured changes in shear strength generally show $2.6 > \frac{Cu_\infty}{Cuo} > 1.0$. Francescon found the factor to reduce with OCR as shown in Table 3.13, and for normally consolidated soil the strength gains varied logarithmically with distance from the shaft, with no appreciable changes beyond 9 pile radii.

3.11.8 Within classical critical state soil mechanics gains in shear strength or reductions in moisture content are usually associated with increasing mean effective stress. If such changes take place without σ_r' significantly rising above its initial value, then $(\sigma_\theta' + \sigma_z')$ increases are expected. Thus for normally to lightly overconsolidated soils, σ_r' is unlikely to become the major principal stress as a result of consolidation. This argument contradicts the analyses of both Randolph et al. (1979), who predict overall increases in p' but give $\sigma_r' = \sigma_1'$ for all OCR's and Kavvadas (1982), who finds $\sigma_z' = \sigma_1'$, but calculates an overall reduction in p' for normally consolidated BBC.

3.11.9 For more overconsolidated conditions σ_1' before installation is more likely to act in the radial direction. The balance between increases in σ_r' and mean effective stress will dictate the direction of the final major principal stress. There is very limited data with which to further investigate the stress changes, but the process can be illustrated by reference to critical state theory and Francescon's test data. Figure 3.30 gives the e - log p' diagram for the kaolin employed; the isotropic and 1 dimensional virgin consolidation lines (VCL's) are shown, as is the critical state line. The initial conditions for OCR's 1, 2, 4 and 8 in the A series experiments are also indicated. Let us assume that undrained pile installation takes the soil close to the shaft to the critical state point, C, which is common as a result of the four essentially equal initial water contents. If the stress paths during equilibration are directed inwards from the state boundary surface then they would remain on the elastic wall, and follow the swelling line C E until the path intercepted the state boundary surface; further consolidation would follow steeper e - log p' lines. For an outwardly directed stress path, a curve such as CP would be followed but this could be no steeper than the critical state line.

3.11.10 Figure 3.30 also shows the void ratios at the preconsolidation conditions for each sample (indicated M) and the final equilibrium values measured close to the pile shaft (indicated ∞). Inspection of the plot shows three points;

(i) For OCR's 4 and 8 the void ratio changes are insufficient to exceed their K_o preconsolidation values.

(ii) It is possible for the soil near the shaft to finally become normally consolidated on the K_o VCL after full equalisation, for initial OCR's 1 and 2.

(iii) Substantial gains in p' are probable for all cases during equilibration from point C, although the magnitudes of $\Delta p'$ are far from certain.

3.11.11 Francescon's experiments showed overall increases in σ_r' of 58% and 81% for OCR's 4 and 8 respectively. Scaling from Figure 3.30 shows that, even if path CP followed the critical state line, the observed changes in water content would predict respective gains in p' of 100 and 150%. These minimum estimates of the final p' values show σ_r'/p'

remaining constant at 1.05, for OCR 4, and falling from 1.18 to 0.87 for OCR 8; larger reductions are more probable. The simple examination suggests that, even for heavily overconsolidated clays, the direction of σ_1' after installation and equilibration is not certain to be radial.

3.11.12 The measurement of soil stresses acting the pile shaft which have been reported in Chapter 3 have been made using a wide range of piles, soils and boundary conditions. These data have been discussed in relation to idealised analyses of the installation process, all of which considered only the simplest case of steadily jacked closed-end piles. For example, the strain path method gives datails of the events occurring at depth, but ignores the ground surface. In contrast, the solutions of Fathallah (1978) allow for the presence of a free surface but do not consider pile penetration realistically.

3.11.13 The details of pile geometry and method of installation could also be important. Carter et al. (1978) considered differences between closed and open ended piles, through an extension of the cavity expansion analysis. Assuming no soil plug to develop, it was predicted that open ended piles would induce smaller pore pressure changes, but that the values of $\sigma_{r\infty}'$/Cuo would not be greatly affected. Baligh (1984) found strain path solutions for the penetration of a cylindrical sample tube into clay. Although no details of the stress state acting on the outside of the tube are given, his observation that soil below the cutting edge is first failed in triaxial compression suggests that the strain paths are more complex than simple cavity expansion. It will be recalled that the tip of an advancing closed end pile also caused compressive failure in the soil immediately beneath it.

3.11.14 Installation by driving, rather than jacking, is thought to produce particular effects through pile whip and the cyclic nature of the loading, Kraft (1982). An analogy with horizontal pile loading suggests that the effect of pile whip would be confined to the soil within 5 diameters depth of the ground surface. As will be discussed in section 3.12, differences in soil fabric may also be expected.

3.11.15 The data presented in Tables 3.8 and 3.12 are insufficient to evaluate the differences between soils, pile type and test boundary conditions. However, the measurements with driven open-ended pipe piles

show similar results to the tests on jacked, or driven closed ended piles, and the overall trends suggest that none of these factors dominates the processes.

3.12 CHANGES IN SOIL FABRIC

3.12.1 Reviews were given of soil displacement observations during pile installation and fluid velocity field analogies in Sections 3.5 and 3.7. It was evident that even a smooth displacement pile develops an annulus of soil which has undergone considerable shear straining. The effects on soil fabric can be seen in samples taken near to displacment piles and may be studied in greater detail using thin section or electron microscope techniques.

3.12.2 Martins (1983) presented a review of published fabric studies, and also made considerable use of thin sections to interpret his model pile tests. Martins' results, and those of previous workers underline the tendency of vertical soil movements caused by installation to be confined to a zone within approximately one radius of the pile shaft. Within this region soil bedding fabric becomes inclined in the axis of displacement, outside it remaines horizontal. Martins noted that the large vertical displacements between pile and soil near the shaft were accommodated by shear surfaces within the soil, which were generally found no further than 0.05 radii from the shaft. Similar observations were reported by Koizumi and Ito (1967), Tomlinson (1970) and Grosch and Reese (1980). In addition to the principal displacement surfaces, Riedel shear structures inclined at 10 to 20^0 to the pile were found within a zone around the interface which was about one radius thick. Martins' results compliment the fabric studies of Tomlinson (1970) and showed slight differences between the effects of jacking and driving for piles. The latter method appears to give a skin of numerous shear surfaces which are less oriented than the single principal displacement surface which was formed during jacking.

3.12.3 Kitching (1983) reported two studies of soil fabric changes around jacked piles. The first concerned installation in a boulder clay. A trial pit was sunk and samples taken from the soil near to the pile. A clear boundary at the pile-soil interface was found which was

highly polished and striated. Thin sections of the adjacent material showed a high degree of soil orientation and particle sorting. This study was, however, complicated by the presence of a thick oil based paint on the pile.

3.12.4 The second study was carried out in the London Clay at Cannons Park. A device developed by the Building Research Station was employed to take samples of the clay near the interface. Metal tubes were jacked out from port holes formed in the pile and the thin section technique used to examine fabric changes. The experiment was not entirely succesful, but many of the features observed in model tests were confirmed. In particular, a polished shear surface was proved to exist a few millimetres from the pile-soil interface.

3.12.5 The soil fabric studies underline the importance of the vertical,normal and shear strains induced by pile installation. They also reinforce the importance of considering residual fabric conditions in the soil near to the pile or at the interface. The zone within one radius of the pile can be seen to be distinctly reoriented, and in some cases changed in grading. These effects combine with the changes in states of stress discussed in Section 3.11 to produce the inital conditions for pile loading.

3.13 THE MONOTONIC VERTICAL LOADING OF PILES

3.13.1 The discussion of pile-soil action under vertical load will be limited to the load displacement behaviour of a short segment from the central part of a long pile. As the shaft displaces downwards the soil must adhere to the pile until rupture occurs. Thus, adjacent to the pile the circumferential and vertical soil strains will equal zero for undrained conditions, as will ε_r. The near pile behaviour is thus controlled by the kinematic response of the soil under these constraints, Martins (1983). The pile displacements can give rise to changes in σ_r', σ_θ', σ_z', τ_{rz} although radial symmetry gives $\tau_{r\theta}$ and $\tau_{\theta z} = 0$. If the loading is undrained the pore pressure can also change. The stresses at failure near the interface are therefore difficult to analyse and are likely to be sensitive to the constitutive law assumed for the soil.

3.13.2 As the pile segment is displaced shear strains develop in the surrounding soil, with any plastic strains being concentrated near to

the shaft. Further away from the pile the loading gives rise to a rapidly varying distribution of τ_{rz} which largely governs the elastic load-displacement behaviour, Cooke (1973), Randolph and Wroth (1978). Attention will first be concentrated on the response of the soil near the shaft.

3.13.3 The earlier analyses of the soil-pile action assumed elastic soil behaviour. The uncoupling of volumetric and shear straining implicit in this theory results in the prediction that near the shaft σ_θ', σ_z', σ_r' and u do not vary as the pile is loaded. This is shown in Figure 3.31. The elastic analyses of Parry and Swain (1979) and Lopes (1980) therefore predict that if the soil fails as a Mohr Coulomb material, the rupture planes can form at various angles to the pile and that δ' will vary with the initial stress ratio K. Martins (1983) plotted the variation of δ' with K in the form shown in Figure 3.32. He also indicated the constant ratio predicted from the assumption of perfectly plastic behaviour at failure, and the results of the careful model pile tests reported by Chandler and Martins (1982). The tests were carried out with kaolin samples normally consolidated to various stress ratios before the installation of grouted piles into holes bored through the clay with minimum disturbance. For these general conditions the elastic assumption is clearly invalid, even when combined with a realistic failure criterion.

3.13.4 Potts and Martins (1982) considered pile loading using two variants of modified Cam Clay. In addition to the type of model, (A), employed by Randolph et al. (1979) they used a second version, (B). For the second version the yield surface gave a constant ϕ' for all critical state conditions in the deviatoric plane, and the plastic potential gave realistic predictions of σ_2' for plane straining. These models are discussed in more detail in Appendix 2. Both variants include the assumption that all volume straining ceases at the ultimate, critical state conditions. Axes of principal stresses and strain increments will also coincide at failure. These assumptions give the principal stress directions at failure as $\pm 45^0$ from the vertical and

$$\delta' = \tan^{-1} (\sin \phi') \qquad \text{Eq. 3.14}$$

3.13.5 Potts and Martins used finite element simulations of the model pile tests to check their choice of constitutive law. Good agreement

was found between the laboratory results and the predictions of model B up to peak capacity. The curve of τ_{rz} against pile displacement and the observed slight fall in σ_r' were reproduced as shown in Figure 3.33. The pile tests, however, showed a marked fall in τ_{rz} with increasing displacement which was not found in the computer simulations. Martins (1983) used a series of thin section studies to show this reduction to be related to fabric changes in the soil close to the pile. A residual surface was seen to develop rapidly after peak conditions.

3.13.6 Potts and Martins (1982) and Martins (1983) report wide ranging numerical studies of pile loading from a variety of initial conditions. Economical finite element solutions were achieved using model variants A and B, with a disc like finite element mesh, to simulate the behaviour of a short section from a long pile. Their main results are summarised below;

(i) Model A tended to show far larger values of τ_{rz} at failure than model B.

(ii) The capacity of a pile installed with minimal disturbance into a normally consolidated soil would be expected to be only slightly dependent on drainage conditions. Model B would predict values of α, the skin friction coefficient, of around 1.0 for a drained loading and around 0.9 for an undrained test. In the latter case negative pore water pressure changes might be expected initially, and in both cases σ_r' would not vary greatly from its original value. Figure 3.34 shows the variations of stresses and strains caused by drained loading.

(iii) When the state of stress predicted by Randolph et al. (1979) for full consolidation around a driven pile was considered different results were found. The assumption of normally consolidated soil with $\sigma_z' = \sigma_\theta' = \sigma_r'$. (K_o)n.c. gave rise to reductions in σ_r' of around 30% and 40% for drained and undrained loading respectively. Large pore water pressure increases were predicted in the latter case. The calculations also showed increased differences between drained and undrained capacities. In relation to the pre-installation undrained shear strengths, the α values amounted to 1.9 and 1.6 respectively.

(iv) Parametric studies were presented to show the effects of

varying G, λ, K and ϕ'. It was found that none of these variables gave differences in capacity as significant as those resulting from the choice of initial stresses and undrained shear strength. The effect of overconsolidation ratio was investigated for the range 1.0 > OCR > 2.0, assuming the conditions of $\sigma_\theta' = \sigma_z' = (K_o)\ \sigma_r'$. With increasing OCR the reduction in σ_r' caused by loading became less marked, but there was little effect on the ratio of τ_{rz} to preloading undrained shear strength.

(v) The presence of a shear surface formed during pile installation was assumed to give no effect until the limiting ratio $\tau_{rz}/\sigma_r' = \tan \delta'$ is reached. At that point the various stress-displacement curves were simply truncated as illustrated in Figure 3.35. If loading started from the conditions discussed above in (iii), then the shear surface would suppress the large reductions in σ_r', but the overall α values at peak capacity would be smaller. However, Potts and Martins concluded that even with such preformed residual surfaces the α values calculated from the 'Cambridge' post-consolidation stresses would be unrealistically large.

3.13.7 Randolph and Wroth (1981) considered undrained pile loading through an analogy with simple shear tests. They noted that, if the values of p'f and ϕ' found in triaxial compression held for plane strain, the plane strain undrained shear strength would be smaller than the triaxial Cu. (For convenience they assumed the σ_2 was the mean of σ_1 and σ_3 in plane strain). The authors reviewed the uncertainties regarding the failure conditions in simple shear tests and concluded that the hypothesis of Josselin de Jong (1971), that failure might first develop on vertical surfaces, gave the best description. This supposition could explain the brittleness observed in simple shear tests and the tendency for the ratio of τ_{zh}/σ_z' at peak strength to fall below tan ϕ'. Combining these assumptions, a formula was produced for the ratio τ_f^{ss}/Cu^{TC} which related simple shear strength to the triaxial compression Cu. This ratio was seen to fall slightly below the scatter found experimentally.

3.13.8 Randolph and Wroth (1981) combined the expression for τfss/CuTC with an equation for triaxial shear strength derived from

Modified Cam clay. This gave a formula for the ratio of simple shear strength to vertical consolidation pressure. Noting that the cavity expansion results of Randolph et al. (1979) predicted $\sigma_\theta' = \sigma_z' = K_o \sigma_r'$, the authors assumed that the simple shear results (where $\sigma_r' = \sigma_\theta' = K_o \sigma_z'$) could be transposed by cycling σ_r' for σ_z'. The cavity expansion finding that $\sigma_{r\infty}'/Cu^{PS}_o$ varied between 6 and 5 could then be used to calculate peak shaft capacity factors, α and β, for a variety of initial conditions. These values were seen to fall within the range given by pile load tests.

3.13.9 With regard to this important argument, it is useful to note the following points;

(i) The analysis starts from the assumption that after consolidation, σ_r' is the major principal stress near to the pile shaft. The data discussed in section 3.11 do not support this. The simple shear tests show reductions of up to 80% in σ_v' during undrained loading so, for the analogy to be appropriate, similar reductions in σ_r' should be occurring on the shaft of piles loaded to failure.

(ii) The ratio of simple shear strength to triaxial compression is usually explained as a result of anisotropic soil structure and stress history. Pile installation may not simply produce a 'sideways' version of these effects.

(iii) The Josselin de Jong (1971) failure hypothesis would result in rupture first occuring on horizontal surfaces at the pile edge. No evidence for these features was noted in the fabric studies carried out by Martins (1983). In contrast, Randolph and Wroth take no account of the possibility that residual fabric might be induced by pile installation.

(iv) As described in Section 3.2, undrained brittleness and the achievement of peak strength conditions before peak obliquity are common in tests on K_o normally consolidated clays performed in the triaxial and plane strain apparatus', where the failure mechanisms are not ambiguous. Finding similar features in simple shear tests need not imply the development of vertical rupture surfaces at peak shear strength.

3.13.10 The discussions concerning pile-soil failure conditions can only be satisfactorily resolved through field trials with instrumented

piles. However, the presently available data are limited. Martins (1983) reviewed case histories of measurements of pore pressures generated along the shafts of displacements piles during loading. Table 3.15 reproduces his summary which indicates changes of between 0 and 1.5 Cuo being found at the pile face. Peuch and Jezequel (1980) report simultaneous measurements of pore pressures and radial stress. They showed that both σ_r and u increased during pile loading with a resultant change in σ_r' of around 0.5 Cuo.

3.13.11 O'Neil et al. (1982) present measurements of shear stress, pore water pressure and radial stress for field scale piles, both prior to loading and at failure. The authors report considerable scatter in the measurements of σ_r and u, but for each of the five instrumented piles, the changes in σ_r' during loading fell within ± 8% of the post consolidation values, and typically amounted to ± 0.1 Cuo. These driven piles showed peak capacities which exceeded the residual values by around 10%, and peak angles of δ' may be calculated from the reported data. Using only the instruments considered reliable by O'Neil et al., a δ' value of $\approx 14^0$ can be calculated in the upper, high plasticity, clay layer with $\delta' \approx 13^0$ in the underlying low PI strata. These values of δ' are far below the quoted laboratory peak strengths and suggest that residual conditions prevailed at failure.

3.13.12 Francescon (1983) reports measurements of radial effective stresses in his model pile tests for three conditions; peak capacity, the residual states after monotonic loading and after cycles to failure in tension and compression. Although there were differences between his two test series the results can be summarized as;

(i) For normally consolidated soil, peak conditions were achieved with reductions in radial effective stress of between 3 and 17%. Similar reductions were noted for samples with initial OCR's less than 8. For more heavily over consolidated soil reductions of 25 to 50% were found.

(ii) At 'residual' conditions the σ_r' values were further reduced by around 10 to 15% for the tests with OCR's of 1 and 2, but the more overconsolidated samples showed no clear trend for σ_r' to fall with strain.

(iii) Cycling to failure in tension and compression caused further

decreases in σ_r' for the series A tests alone.

(iv) Francescon's measurements allow average values of δ' to be calculated at peak and post peak conditions. Table 3.14 lists these data for test series A. The mean peak value of δ' is found to be 14.8^0, with 13.5^0 for the average angle post-peak. angles. Tomlinson (1970) also noted small peak effects with piles driven into London Clay that were not apparent after slow jacking. Martins (1983) found similar features when comparing model piles installed by jacking and driving. The values of δ' measured by Francescon give good agreement with the residual angles determined by ring shear tests on kaolin, as reported by Lupini (1981).

3.13.13 Although Morrison's (1984) model pile experiments were principally concerned with steady penetration and consolidation phenomena, he reports some preliminary data for loading after full equilibration. The equipment was not designed for this purpose and the results may be questioned as;

(i) The shear stresses were not measured near the PLS cell, but calculated from the total pile load minus an assumed tip resistance.

(ii) The loading system was crudely controlled, and the peak load was obtained in only a few seconds, thus severely testing the response time of instruments and monitoring system.

(iii) Laboratory tests carried out with the model pile showed that during consolidation a 'cake' of clay adheres to the rough porous stone of the PLS cell. On loading, local shearing of this 'cake' might be expected to produce higher pore pressures than those acting on the smooth lateral stress sensing membrane, Hight (1985).

3.13.14 Bearing in mind these difficulties, Morrison's data showed sharp increases in pore water pressure as the pile was loaded, with smaller gains in total lateral stress. Ignoring the possible errors in pore pressure measurement, large reductions in σ_r' were found at peak capacity. The average values of δ' calculated at peak conditions, and the percentage reductions in σ_r' are given in Table 3.16. The data for two tests in the lower clay are summarised in Figure 3.36, and it can be

seen that further reductions were recorded in σ_r' post peak. The data also appears to show δ' rising post peak with tensile final values of σ_r' in many cases. The peak values of δ' given in Table 3.16 are, however, similar to those found in soil-interface shear tests when allowance is made for the different normal stress levels and fast rates of displacement, Morrison (1984).

3.13.15 In summary, observations of pile loading effects are too limited to allow many firm conclusions to be drawn. The predictions of Martins (1983) and Potts and Martins (1982) that jacked piles might fail at residual conditons, with only slight reductions in σ_r', are in general agreement with the field data presented by O'Neil et al. (1982) and the model pile test results provided by Francescon (1983). Morrison's (1984) data only fits this pattern if it is assumed that the PLS cell over-reads porewater pressure on loading. The latter two sets of tests indicate further reductions in σ_r' if the pile is displaced beyond its peak capacity, a feature which was not predicted theoretically. The analysis of Randolph and Wroth (1981) is difficult to apply as it assumes initial states of stress rather different from those measured. The hypothesis that rupture starts on horizontal planes is contradicted by the absence of any post-peak increase in δ' in either Francescon's tests or those reported by O'Neil et al. The apparent tensile values of σ_r', and increase in δ' after peak capacity, calculated from Morrison's experiments may simply have resulted from the different roughnesses of the pore pressure and lateral stress sensing components of the PLS cell.

3.14 EFFECTIVE STRESS METHODS FOR DETERMINING SHAFT CAPACITY

3.14.1 Randolph and Wroth (1981) proposed design charts for the shaft capacity coefficients α and β by combining the Cambridge cavity expansion results and their simple shear analogy. The problems of the two component parts of the argument have already been discussed.

3.14.2 Cavity expansion solutions have given rise to the series of effective stress design methods ESM1, ESM2, ESM3 and ESM4, Kraft (1982). The latest of these variants introduces empirical factors in order to obtain more satisfactory agreement with field data. The mean effective

stress at the pile shaft after installation is assumed to be some proportion of the critical state value for a remoulded triaxial test at the same moisture content so;

$$p' = \eta\ p'_{cs} \qquad \text{Eq. 3.15}$$

The term η is less than unity and accounts for the effects of cyclic soil loading and pile whip during driving. The excess pore pressure acting at the pile face, Δu, is taken from the expression derived by Randolph et al. (1979) for cavity expansion;

$$\Delta u = (p_o' - pcs') + Cu\ \ell n\ (G/Cu) \qquad \text{Eq. 3.16}$$

The change in p' during consolidation is assumed to be given by $\xi\ \Delta u$. This second empirical factor accounts for reductions in total stress during equilibration. Failure conditions are found by assuming that loading produces no change in mean stress up to peak conditions and that;

$$\tau_{rz} = p_\infty' \sin \phi' \qquad \text{Eq. 3.17}$$

Kraft deduces values of η and ξ by considering load tests before and after consolidation and assuming values for p_o', p_{cs}', G/Cu and ϕ'.

3.14.3 Effective stress method 4 is empirical and therefore does not represent a great advance on the α and β methods. The assumptions that p' at failure equals p_∞', and that Equation 3.17 holds for an intact value of ϕ' contradict the findings of Martins (1983) and finds little support in the data of O'Neil et al. (1982), Francescon (1983) or Morrison (1984).

3.14.4 Martins (1983) proposed a lower bound formula for the capacity of long displacement piles. He argued that σ_r' at failure would not fall below $\sigma_{vo}'(K_o)n.c.$, and that the angle δ' could not fall below the value found in a direct shear test between soil and a polished interface. The minimum shaft capacity would then be;

$$\tau_{rz} = (1 - \sin\phi)\ \sigma_{vo}'\ \tan\delta' \qquad \text{Eq. 3.18}$$

Evaluations of this expression gave good lower bounds to the field test data collected by Kraft, Focht and Amerasinge (1981). Although the review of experimental data given in Section 3.11 indicated some case histories where $\sigma_r'^\infty$ fell slightly below $(1 - \sin\phi)\ \sigma_{vo}'$, Martins's values of δ' are possibly conservative for driven piles, and the expression remains tenable.

3.15 PREDICTION OF LOAD-SETTLEMENT BEHAVIOUR

3.15.1 The main emphasis of the work reviewed in this chapter has been the consideration of the stresses acting against the pile face during phases of installation, equilibration and loading. This is natural, as the provision of adequate capacity is the primary aim in the design of the piles. However, the estimation of the load-deflection behaviour is also important, and a brief section is included here to discuss the relationship between the described effective stress process, and the settlement characteristics of piles.

3.15.2 Pile head settlement at any given load derives from the sum of displacements on residual discontinuities, plastic strains in material near the pile and elastic strains in the surrounding soil. A two dimensional finite element analysis could be used to derive load-displacement behaviour providing that all three components were correctly considered. Assuming that the pre-loading stress distributions can be found, and that suitable formulations are made for the constitutive behaviour near to the pile, the problem would rest with the treatment of the shear stiffness of the soil mass.

3.15.3 As discussed in Section 3.2 soil stress-strain behaviour before yield is unlikely to be isotropic or linearly elastic. Furthermore, the process of installation could produce radial inhomogenieties around the pile. The shear strains developed during driving will have initially reduced stiffnesses within an annulus extending to perhaps $10r_o$, but increases in mean stress during equilibration would cause soil within this zone to become stiffer with time.

3.15.4 The presently available analyses of pile settlement under load are based on either the theory of elasticity, or total stress non-linear approaches. These methods were discussed in Chapter 2, and the review will not be resummarized here. Nonetheless, it is useful to note that Cooke, Price and Tarr (1979) were obliged to assume steep radial variations in elastic properties, in order to match their field observations of ground strains around jacked piles loaded after equilibration. The critical state modelling of Potts and Martins (1982) used an effective stress criterion to separate zones where the soil was

alternatively yielding or straining elastically. However, the assumption of a constant and invariant elastic shear modulus is unlikely to provide a realistic description of the stress-strain characteristics in the continuum surrounding the pile.

3.16 SUMMARY

3.16.1 Recent advances in the understanding of soil properties have shown important features that are not fully considered by many existing soil models. These include the brittle behaviour of K_o normally consolidated clays in compression, the effects of σ_2' on failure, anisotropy of undrained shear strength, non-linear stress-strain behaviour before yield and the development of residual fabric in soil-soil or soil-interface shear.

3.16.2 Analyses of pile installation effects have been found to be extremely sensitive to the initial conditions in the soil layers, the choice of soil model and the assumption of soil strain paths. Field and laboratory measurements suggest that modified Cam clay (model type A) coupled with the assumption of cylindrical cavity expansion gives approximate estimates of pore pressure changes but greatly over estimates the radial stresses acting on a pile shaft after driving.

3.16.3 The assumption that soil strains can be found by analogy with fluid velocity fields gives an improved description of the soil deformation during pile installation. Use of the deduced strain paths and non-linear anisotropic soil models gives more realistic predictions of σ_r' for lightly overconsolidated clays. However, the strain path methods underestimate the pore pressures and total stresses acting at the side of the pile.

3.16.4 Coupled consolidation analyses of the equilibration phase that follows pile installation are also sensitive to the initial conditions and assumed soil constitutive law. Widely different estimates of the states of stress pertaining to full consolidation have been obtained.

3.16.5 Field and laboratory data indicate that the predicitions of the Cambridge cavity expansion solutions seriously overestimate the final values of σ_r' and possibly underestimate σ_θ' and σ_z'. Closer agreement for the radial effective stress is found with the results of a

coupled consolidation analysis that starts from the initial conditions derived from the strain path method, and employs an isotropic critical state model.

3.16.6 The consideration of pile loading after full consolidation depends on the assumptions concerning the equilibrium soil stresses, the undrained shear strength, and the possible development of residual fabric. The field and laboratory data indicate only slight changes in σ_r' on undrained loading, and suggest that near residual conditions apply to displacement piles. The use of a constitutive law such as modified Cam clay, variant B, with allowance for sliding on residual surfaces, appears to give a satisfactory model for pile loading, but more reliable field data are required to verify this observation.

3.16.7 Pile load displacement characteristics will be controlled by the combination of the behaviour near the interface, where plastic straining and possibly sliding can occur, and the stress-strain properties of the rest of the soil mass. Laboratory data suggest that the pre-yield characteristics are unlikely to be linear, and may be altered by the processes of installation and consolidation.

CHAPTER 4

PARAMETRIC STUDIES WITH THE FINITE ELEMENT METHOD

4.1 INTRODUCTION

4.1.1 Chapter 3 considered the components of an effective stress theory for pile behaviour. Theoretical treatments were explored, and data from the field and laboratory were used to give general assessments of the effective stress conditions of soil elements close to the shaft of driven piles.

4.1.2 It was shown that analysis of the sequence of installation, equilibration and loading is difficult, even for elemental sections of ideal piles. The need to study these processes with the appropriate soils was evident, and the use of laboratory model piles appeared to offer an expediant approach. There was also a clear requirement to develop ways of generalising the elemental behaviour to allow predictions of the full scale response.

4.1.3 In the following pages two sets of parametric studies are reported which were carried out using elasto-plastic finite element analysis. The first study was implimented to assess the value of a laboratory model designed to simulate pile behaviour under controlled stress conditions. The second analysis attempted to identify any special features of full scale pile-soil behaviour that might influence the course of the investigations, and provided a preliminary impression of full scale foundation characteristics.

4.2 A STUDY OF THE MODEL PILE APPARATUS DEVELOPED BY MARTINS

4.2.1 Martins (1983) developed a model test apparatus in order to study the behaviour of ideally installed grouted piles in normally consolidated kaolin. His experimental programme considered drained pile loading with the boundary conditions of his cylindrical kaolin samples being carefully controlled.

4.2.2 Towards the end of his work, Martins extended his study to consider, in a qualitative way, the behaviour of jacked and driven

piles. At first sight, it appeared that his test equipment could be usefully applied to study the development of stresses, strength and response to load of steel model piles driven into prepared samples of North Sea Clay. A test programme was envisaged with reconstituted clay from the Magnus site being Ko consolidated to provide a representative range of overconsolidation ratios and initial stresses.

4.2.3 Pilot tests in a small oedometer indicated that the times required to consolidate suitable test specimens from slurry were considerable. A non-linear finite difference consolidation program was written to help determine appropriate strain rates for a displacement controlled consolidation cell but, even with optimum conditions, a 10 week compression period was required to produce 200mm long samples of firm clay.

4.2.4 Despite the long consolidation times it was calculated that it would not be practicable to carry out pile loading tests without partial drainage occuring, and that undrained cyclic testing would not be feasible.

4.2.5 Martins (1983) had carried out numerical simulations of his experiments and had concluded that tests on overconsolidated kaolin samples would involve unacceptable stress inhomogeneities. It was decided to determine the probable response of over-consolidated samples of soil from the Magnus site in the same equipment. The Magnus soils were thought to consist of mainly low plasticity clays which would follow a Hvorslev type state boundary surface on the dry side after yield, and would show no tendency for residual fabric to develop between displacing masses of soil.

4.2.6 The calculations were intended to consider an ideal pile which had been installed without disturbance into a bed of clay, that had been Ko consolidated to an overconsolidation ratio of 4.0 with $\sigma'_v = \sigma'_r = \sigma'_\theta = 200$ kpa. The soil was assumed to behave as a modified - Cam Clay material, and variant B was chosen for the calculations, values were assigned to the parameters X, Y and Z as recommended by Potts and Gens (1983)[1]. The more familiar constants were estimated from the Fugro Site Investigation report (81/0261-2, of July 1981) with;

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[1] See Appendix 2.

v_1 = 1.10
λ = 0.085
k = 0.015
ϕ' = 30°
G = 40,000 kpa. (equivalent to Eu/p'_o = 600)

A Hvorslev surface of slope angle equivalent to $\phi'/2$ was specified on the dry side.

4.2.7 The axially symmetric finite element mesh shown in Figure 4.1 was used to represent Martin's test equipment, and that in Figure 4.2 to model an element from a long ideal pile. The numerical simulations were carried out using program ICFEP, which is briefly described in Appendix 1. Loading was modelled by imposing increments of vertical displacement on the pile elements. For each increment, the distributions of the stresses acting over the pile shaft were recorded, and the results are summarized in Figures 4.3, 4.5 and 4.6. A similar procedure was followed to simulate the loading of the pile element, and this gave the load-displacement curve shown on Figure 4.4.

4.2.8 The results obtained from the analysis show a number of interesting features: (i) The shear stress-deflection curves for the experiment and the ideal pile differ considerably. (ii) The 'elastic' response of the 'experiment' is around 40% too stiff, and (iii) The 'experimental' stress-displacement curve shows neither the sharp yield and peak effect or the subsequent strain-softening characteristic predicted by the pile element study.

4.2.9 The reasons for such discrepancies can be seen in Figures 4.5 and 4.6 which give the predicted variations on shear stress, τ_{rz}, the stress measure[1], S, and the radial effective stress σ'_r, over the length of the model pile. There are striking non-uniformities in S and τ_{rz} with failure developing near the ends of the pile before the average load reaches half its ultimate value. Progressive failure could propagate rapidly under these conditions, and the stress state on the pile shaft only becomes uniform as the pile reaches its ultimate capacity.

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[1] S is defined as J/J_{max} and approximately equals $\frac{\tan\phi \text{ mob}}{\tan\phi^1}$

4.2.10 At the outset, the Martins apparatus appeared an effective model with which to study the loading of a pile element. The analysis showed that this is not the case, and that serious non-uniformites of stress and strain are to be expected. The relatively narrow soil sample, the lack of complimentary shear over the vertical boundary, and the restraint of vertical displacements all contribute to the difficulties. By analogy, it would appear that even greater nonuniformities might be expected when considering displacement piles with this apparatus, as the installation strain paths and boundary conditions would be severely constrained. In Chapter 3 it was noted that consistent differences were found by Francescon (1983) between identical tests carried out in his A and B model pile test cells, and this reinforces the conclusion that the results of such experiments are sensitive to the details of the experimental apparatus. For the same reasons load-displacement curves from model tests could give poor indications of field shear stress-displacement characteristics, particularly with soils that are non-linear before yield or exhibit pronounced brittleness.

4.3 STUDIES OF THE BEHAVIOUR OF LARGE DRIVEN PILES

4.3.1 The second analysis considered in this chapter was undertaken to obtain a preliminary assessment of the response to load of a group of large driven piles. Elasto-plastic finite element simulatious were used to calculate the complete load-displacement curves of single piles, and also to produce estimates of group behaviour.

4.3.2 The parametric study was not performed as a purely theoretical exercise. In fact, the work related to a specific site and was carried out by the author in conjunction with his colleagues Dr D. Hight and Dr D. Potts. The original aim of the project was to develop a strategy for interpreting load-settlement data from Conoco's Hutton Tension Leg Platform (TLP). The monitoring of the TLP foundations, and the interpretation of the results, will be described in Chapter 11.

4.4 OUTLINE OF THE TLP PARAMETRIC STUDY

4.4.1 The foundation conditions at the Hutton TLP site were investigated for Conoco by Fugro Ltd. Numerous factual and design reports have been compiled and the simplified interpretation given in Appendix 5 of the Fugro report No. 00638 was used as the basis for the parametric study. This particular profile was thought to represent a conservative assessment of the soil conditions, it also provided compatability with the designers' assumption.

4.4.2 The profile could be divided into nine soil layers, three of which were considered to be granular. Undrained shear strengths were assigned from the results of triaxial tests, and values of Ko had been derived using procedures similar to those described in Section 3.4. The clay layers were classified as typically being of medium plasticity, and the sandy strata were assumed to be frictional materials with ϕ' between 30° and 35°.

4.4.3 Fugro's consultants had carried out finite element studies with the simplified profile as a means of estimating the effects of combined vertical and horizontal loads on individual piles with linear elastic-plastic properties being assumed for the soil. In the clay layers the Youngs' modulus was taken as E_u = 400 Cu, and Tresca yield assumed. For the sands E' was calculated for $E' = 4{,}500 \sqrt{\sigma_v}$ (Kpa), Poissons ratio was taken as 0.35 and yielding assumed to follow a Mohr-Coulomb criterion with $\phi/2$ as the angle of dilation. Limiting skin friction capacities were assigned to interface elements adjacent to the pile shaft.

4.4.4 The work carried out by the author and his colleages concerned axial loading alone, the finite element method was used to assess the effects on load-displacement characteristics with undrained conditions in the clay layers, but drained in the sand. To achieve this the calculations included;

(i) Simple elasto-plastic modelling in which the assumed values of the shaft capacity coefficient α and the Youngs moduli were varied,

(ii) Analyses of the pile loadings in which critical state soil models were used to calculate the response of the clay layers. For

these more sophisticated computations, the assumed initial stress distributions and critical state soil parameters were also varied.

4.4.5 It will be recalled from the last chapter that critical state models could give reasonable descriptions of the stress conditions close to a pile shaft as it is loaded to failure. To do this it was necessary to select a yield function that gave consistent values of ϕ' in the deviatoric plane, and to make allowance for the possible development of residual fabric close to the pile shaft.

4.4.6 With the available models the elastic response of the soil to shear could only be described using a linear shear modulus. It was noted in Chapter 3 that this last assumption is unlikely to closely represent real soil behaviour.

4.4.7 None of the soil models employed were able to represent the true characteristics of residual fabric at the pile-soil interface. In the simpler elasto-plastic calculations soil strain softening could be allowed in the elements of soil which were in contact with the pile, and this facility was used to approximately simulate the effects of progressive failure from peak to residual capacity.

4.4.8 The study considered the foundations of the Hutton TLP, and tension pile loading was simulated by imposing upward increments of pile displacement. The size of the increments was varied to ensure convergent solutions and minimal residual stresses between the steps.

4.5 DESCRIPTION OF INDIVIDUAL ANALYSES

4.5.1 Seven of the analyses carried out will be discussed in this section. The initial conditions and assumed soil models are summarized in Figures 4.7 to 4.10 and Table 4.1. The single pile analyses are referred to by the identification letters A to H, (run E is not considered here). The pile itself was represented as a solid cylinder 1.83 metres in diameter with an equivalent Young's modulus, E pile, of 27.7×10^6 KN/m^2 and Poissons ratio of 0.25. This corresponds to the actual dimensions of the pipe piles and a steel modulus of 20.7×10^7 KN/m^2. The finite element mesh for the single pile is given in Figure 4.11 and ICFEP was used to find complete load-displacement curves for each case.

4.5.2 Analyses A,B,C and D were intended to show the effects of soil stiffness and shaft friction characteristics using the simpler elasto plastic models. The strength of the thin elements close to the pile were reduced for analyses A and D to give shaft friction capacity of Cu/2, or $\alpha = 0.5$. In all the computations, the limiting skin frictions for the granular soils were calculated assuming the limiting skin friction angle, ϕ', to equal $\phi'-5^{o}$. In runs B and C peak α values of 1.0 were specified, but strain softening was permitted so that the ratio τ_{rz}/Cu ultimately fell to 0.7. Figure 4.12 illustrates the way in which α was assumed to reduce with E_p for these two cases. (The strain invariant E_p is a measure of octahedral plastic strain and is defined on Figure 4.12). The stiffnesses for runs A, B and C were not varied from those assessed by Fugro. For run D the soil layers were assumed to be five times stiffer.

4.5.3 The load displacement curves for the four analyses are shown on Figures 4.13 and 4.14, and the following observations are of interest;

(i) The plots show linear portions up to a yield point. The slopes before yield were identical for A, B and C but the initial gradient for analysis D was steeper by a factor of approximately 2.3.

(ii) The yield load for analysis D fell far below that for run A. Run A, in turn, showed yield at a smaller load than analyses B and C.

(iii) The peak strengths permitted for runs C and D had little effect on the load-displacement curves, and the ultimate capacities both corresponded to overall α values of around 0.7. No intermediate 'peak' effects were apparent.

4.5.4 The profiles of surface settlement indicated important features of the pile-soil behaviour. Figure 4.15 shows the calculated radial variations for run A at three loads. Yield first occured with a load factor of around 0.5, and at higher factors an increasingly large proportion of the pile head displacement was accommodated by plastic strains very close to the pile.

4.5.5 Analyses F, G and H employed critical state soil models. In the first two cases normally consolidated initial conditions were

assumed for the clay layers and, following the results of Randolph et al (1979), the stresses were taken as;

$$\sigma'_v = \sigma'_\theta = (1 - \sin\phi')\sigma'_r \qquad (4.1)$$

It will be recalled that the Cambridge cavity expansion analyses described in Section 3.10 predict that at the end of the consolidation period there will have been considerable increases in σ'_v near the shaft. However, Randolph and Wroth (1982) suggest that σ'_v should be taken as the initial effective overburden pressure as this result is more consistent with field measurements.Their rule implies only small increases in p' at the shaft for piles installed in normally consolidated soils, and reductions in p' for overconsolidated layers.

4.5.6 For the critical state calculations it is only necessary to specify the listed soil parameters and insitu stresses; the undrained shear strengths in the clay layers are calculated by the program using the equations for Modified Cam Clay. The profiles of Cu therefore differ from the initial distribution assumed by Fugro, with lower strengths near the surface and higher values at depth.

4.5.7 Identical shear stiffnesses were adopted for runs, F, G and H. Plots given by Gens (1982) and Ladd et al. (1977) show the variations of Eu/Cu with OCR for several soils, and these were used to estimate stiffness profiles for the undisturbed soil conditions. The selected values are equivalent to assuming Eu/Cu to equal 150 near the surface and 750 at depth. The parameters λ, k, v_l and ϕ' for analysis F correspond to those determined for the medium plasticity soil, Drammen Clay. For run G parameters appropriate to the Hutton soils were substituted. The last analysis,H,retained the Hutton parameters, but the initial stresses and overconsolidation ratios were taken from Fugro's estimates of the conditions before pile installation. As a consequence, the shear strength profile calculated in the program for run H differs from those determined for runs G and F, and is approximately equivalent to the initial shear strength-depth variation estimated by Fugro.

4.5.8 The load-displacement curves for the three critical state analyses are shown on Figure 4.14. The agreement between the curves is

remarkable, with yield starting at an early stage in each case. The similarity in the capacities is at first surprising. It will be recalled that the initial radial effective stresses in the clay layers for analyses F and G generally exceeded those for run H : with runs F and G, σ'_r/σ'_{vo} equaled 2.0, but varied between 2.08 and 0.7 for run H. However, Potts and Martins (1982) showed this effect to result from the dependence of the loading stress paths on the direction of the initial principal stresses.

4.5.9 The ultimate capacities for analyses F, G and H were larger than those calculated for runs A to D. Noting that, at failure, the α value in the clay layers was 0.5 for runs A and D, with 0.7 for runs B and C, it can be seen that the 'critical state' runs all indicated α values around 1.0 with respect to the Fugro Cu profile.

4.5.10 The surface displacement data from analysis F are plotted in Figure 4.16 for three load factors. It can be seen that the critical state model predicts the appearance of a discontinuity in the heave distribution at a far smaller load factor than was noted for run A.

4.6 DISCUSSION OF THE FULL PILE ANALYSIS

4.6.1 The parametric study used relatively simple soil models to represent the loading of a single pile in tension, and the results point to several important observations;

(i) For large piles, such as those installed at the Hutton site, plastic straining can develop near to the pile at relatively small load factors.

(ii) The load displacement curves depend on soil stiffness in a complex way. Where a high Young's modulus is assumed, the result is both a steeper loading curve, and the start of progressive failure at a relatively small load factor. The latter feature is illustrated in Figure 4.17 where the distributions of the stress measure S are contrasted for runs A and D at a load factor of 0.5. The higher soil stiffnesses assumed for run D attract a greater proportion of load to the upper layers and failure occurs at a relatively early stage near to the ground surface.

(iii) If the shear stress-displacement characteristics at the pile-soil interface exhibit strain softening, the peak shaft resistance will depend on the pile overall compressibility. In the case considered, only the ultimate shaft adhesion governed peak capacity. However, had the softening rate been less steep, or the pile stiffer, peak adhesion would have been more important.

(iv) In a practical example, there was little difference between assuming that pile installation left the soil conditions unchanged, and adopting the stress state which Randolph and Wroth (1981) considered appropriate to the conditions after driving and full consolidation. Different results might be expected if more realistic pre-loading stress conditions were assumed, and allowance was made for residual soil fabric near the pile shaft.

(v) The surface displacement profiles at working loads show that large piles installed in elasto-plastic soils exhibit radial distributions of heave, or settlement, which are radically different from those predicted with the theory of elasticity. The concentration of straining close to the pile suggests that elastic pile group calculations will overestimate the interaction factors at working loads.

4.7 PILE GROUP INTERACTION AND SETTLEMENT MONITORING

4.7.1 Randolph and Wroth (1979) proposed an approximate elastic pile group analysis which assumed that settlements could be calculated by superimposing the displacement fields of neighbouring piles. This ignores the rigid inclusion effect, but comparisons with group elastic analyses made with integral equation techniques indicated that the errors were not large. The careful pile group field tests of Cooke et al (1979) further support the use of superposition, and it appears reasonable to use the calculated surface displacement profiles of single piles to determine group interaction.

4.7.2 The method allows the non-linear single pile analyses to be extended to consider group behaviour. Such an exercise was carried out for the Hutton TLP foundations with the case of even loading in each of

the 8 piles in the four circular groups. The configuration and relevant dimensions of a pile group template are shown on Figure 4.18. The calculations are facilitated by drawing an axis of symmetry through opposing pile centres, as shown, and then finding the radial distances r_1 to r_5 from the eight pile origins to points on the line such as R. From Figures 4.15 and 4.16 the surface displacements around a single pile are functions of both load and radius, so for a fixed force T per pile the displacements are functions of radius alone. To find the combined movement at point R due to loading all 8 piles, the sum is simply evaluated by adding the components interpolated from the surface profiles;

$$\delta_{R\ group} = \delta(r_1) + \delta(r_5) + 2\left[\delta(r_2) + \delta(r_3) + \delta(r_4)\right] \qquad \text{Eq. 4.2}$$

If the displacements are to be referred to a datum position within the influence of the group, the movements are 'corrected' by subtracting the value of $\delta_{R\ group}$ for that point. Using this technique, group load-displacement curves can be calculated and profiles of surface settlement due to group action drawn up.

4.7.3 Two such profiles have been assessed from runs A and F, and are shown on Figures 4.15 and 4.16 for the piles equally loaded to $T/T_{ult} = 0.125$. Figure 4.19 further illustrates the group response with two overall load-displacement curves calculated from run A. The flatter, remote datum, plot predicts the displacements that would be monitored from a datum position 50 metres from the nearest pile, and the steeper, local, curve indicates the movements as observed from the nearby location L identified on Figure 4.18.

4.7.4 The plots derived from the elasto-plastic analyses suggest that group load-deflection behaviour is relatively insensitive to the mobilised factor of safety, with almost straight line behaviour up to failure. The pile interactions apparently mask the development of plastic strains near the piles. However, the introduction of non-linear behaviour before yield might be expected to further concentrate straining around the piles, and to give more curved group load-displacement characteristics.

4.7.5 The tendency for plastic strains to develop near to the piles before failure suggests a simple way of monitoring the ability of pile groups to sustain the environmental forces imposed by severe storms. An instrumentation system which recorded any changes in level between the pile group and a distant, or local, datum position could be used to measure the magnitude of any permanent displacements caused by storm loading. Provided that permanent displacements could be related to the stability of the piles, such an instrumentation system could be used to verify foundation design. Chapters10 and 11 describe the development of a suitable monitoring system and report its application for two North Sea Platforms.

4.8 SUMMARY

4.8.1 The parametric studies reported in this chapter illustrate the power and flexibility of non-linear finite element programs such as ICFEP. The ability to study continuum behaviour, including yield and strain softening, was invaluable when considering a single pile loaded to failure. The analyses could also be extended to estimate group action by considering of the ground displacement profiles calculated for single piles.

4.8.2 The results of the TLP pile analysis emphasise the difficulties of attempting to separate the calculation of shaft capacity and axial displacement at working load. With large driven piles, yield can be expected near the ground surface at a relatively early stage and this influences progressive failure, pile head settlement and pile group interaction.

4.8.3 The studies were carried out with relatively simple soil models and, for improved predictions to be made, it will be necessary to develop analyses based on more realistic descriptions of; (i) the shear response of the soil away from the shaft, (ii) the yielding of soil near the pile. (iii) residual fabric effects at the interface and (iv) the effective stress state before loading.

4.8.4 It has been expected that useful data regarding points (i) to (iv) could be obtained from instrumented model pile tests performed with a modified version of Martins (1983) test cell. However, a finite

element analysis of the test conditions warned of severe stress inhomogeneities which might obscure the elemental behaviour of the soil under test. The programme of model pile tests was therefore not pursued.

4.8.5 The observation that plastic yield occurs near the surface of the pile, and that permanent displacements become progressively larger as the pile is loaded to failure, suggested a way of using underwater settlement gauges to monitor the safety of offshore pile groups.

CHAPTER 5

LABORATORY TEST PROGRAMME; AIMS, DEVELOPMENT OF EQUIPMENT AND PROCEDURES EMPLOYED

5.1 INTRODUCTION

5.1.1 In order to extend the analyses discussed in chapters 3 and 4 to make improved predictions of full scale behaviour, detailed experimental studies were required in a number of areas. In Chapter 1 it was explained that the laboratory work would be focussed on investigating soils from the Magnus field and the Canons Park test bed site. The first part of Chapter 5 explains the aims of these two testing programmes and reviews the limitations to the experimental work. The second section describes the equipment and reports the development of new internal strain measuring devices. Finally, the third part of the chapter summarizes the testing procedures and discusses the particular problems of saturating intact samples from the Magnus site.

5.2 AIMS OF THE INVESTIGATIONS

5.2.1 The vital first step at any location is to carry out a general site investigation. This had already been implemented at the Magnus site, as reported by McLelland Engineers (1977) and Fugro (1981), but no borehole or laboratory data were available for Canons Park. The work at the latter site therefore had to include a series of routine tests and a full description of the geological profile.

5.2.2 Aspects of the effective stress analysis of driven piles were discussed in Chapter 3, and it was shown that the calculation of the states of stress close to the shaft is strongly dependent on the constitutive characteristics of the soil. It was also shown in Chapters 3 and 4 that the load-displacement relationships of a single pile or pile group would be governed by the combination of yielding or slip near to the pile and the shear response of the surrounding soil.

5.2.3 The programmes of testing therefore needed to include assessments of the general stress-strain and strength characteristics of the Magnus and London clays. As a minimum, tests were required to investigate the properties of Ko consolidated material in the oedometer, and its response to shear in triaxial extension and compression for a range of overconsolidation ratios.

5.2.4 Gens (1982) showed that undrained triaxial tests could give a clear picture of the general properties of a sedimented clay, and Wroth (1971) demonstrated that elastic shear moduli should be independent of drainage. A decision was therefore made to concentrate on undrained shearing from pre-determined initial conditions. The relevance of consolidated undrained tests is further emphasized when it is recalled that during installation and extreme loading of offshore piles in clays, undrained conditions are likely to prevail. The aim of predicting pile load-displacement relationships required that special care should be taken in determining the stress-strain characteristics at the early stages of shearing. Recent work at Imperial College had shown that it is essential to use local instrumentation to observe such features, particularly for overconsolidated soils, Daramola (1978), Costa Filho (1980).

5.2.5 In addition to the minimal programme of work it was desirable to estimate the effects of the shearing and reconsolidation cycles which accompany, for example, pile installation or 'undisturbed' sampling. The strain path solutions for continuous penetration were reviewed in Chapter 3, and Figure 3.15 shows that material near to a displacement pile is subjected to intense shear straining during installation. Studies of the sampling process have also been made by Baligh (1984), and his strain path solution is illustrated in Figure 5.1. The plots indicate that soil on the sampler's centre line is first subjected to failure in triaxial compression, as the cutter advances, and then failure in extension when the sample is retained within the tube. Apted (1977) had already shown that shearing on the inner walls of the sample leads to the development of a remoulded outer annulus of soil, overall changes in the mean effective stress of the sample and redistributions of moisture content.

5.2.6 Section 3.2 discussed the difficulties of implementing detailed investigations of general stress-strain and strength properties using intact samples, and it was noted that the most comprehensive studies invariably employed reconstituted soil. However, it was important that the work described in this thesis should also include tests on intact material. The reconstituted soil enables general patterns of behaviour to be established, and comparisons with the response of intact samples may be used to identify any special features associated with fabric, stress history or bonding. The use of reconstituted clay also allows assessments of the effects of shearing rate and load cycling to be made using identical samples.

5.2.7 Some features of soil behaviour cannot be explored in simple 'element' tests where the stress and strain conditions are more or less uniform. It had been intended to carry out 'pile' model tests but, as described in Chapter 4, finite element analyses warned of unacceptable interference from the test boundary conditions for other than unsampled, unaged, soils at OCR 1. Discontinuous shear, either soil-soil or soil-interface, also required investigation; for this the ring shear apparatus is essential. Use could also be made of the resonant column test to give clues regarding the soil behaviour at small shear strains.

5.3 LIMITATIONS TO THE LABORATORY TEST PROGRAMME: SAMPLES FROM MAGNUS SITE

5.3.1 The piles on which the Magnus structure is founded were driven through an 85 metre thick succession of soils. When analysing the complete pile groups, it is necessary to consider the variations in properties from the sea bed to a depth of more than 120 metres. In fact, the commercial site investigations had assessed conditions to around 200 metres.

5.3.2 Fugro Ltd had taken a full profile of soil samples in their site investigation of April 1981. Thin walled 76 mm diameter, 900 mm long, pushed in sampler tubes had been used, and the retrieved cores had invariably been extruded shortly after recovery from the sea bed. After trimming away disturbed portions, the samples had either been

tested immediately or stored for work onshore. Each sample for storage was first wrapped in polythene film and aluminium foil, and then enveloped in a 10 mm thick wax coating which was cast in a cylindrical cardboard case.

5.3.3 After carrying out their onshore laboratory programme, British Petroleum made the remaining material available to Imperial College. Many of the sealed samples had been opened, divided for testing, and the remnants resealed. Table 5.1 summarizes the complete stock of samples which was transferred to Imperial College in December 1981. It was clear that the testing programme would be limited by the volume and quality of the samples, and that the Fugro data would have to be used to establish the trends of soil properties with depth. Work at Imperial College could only concentrate on reconstituted material with a limited number of comparative tests on intact soil.

5.3.4 The allocation of laboratory equipment, time and technical support defined a second limit on the scope of the programme. For the Magnus work, 3 Bishop and Wesley (1975) stress path cells were available for 38 mm diameter samples, as were 3 conventional '38 mm' triaxial cells. Oedometer equipment could be used, as could the Martins pile test cell, and it was also feasible to carry out ring shear tests. The triaxial cells were to be employed for a period of 18 to 24 months.

5.3.5 The testing of intact samples implied trimming cylindrical specimens, 38 mm in diameter and 76 mm high, from the 76 mm diameter samples. The presence of fissures, laminations or other inclusions such as rock fragments, shells or sand pockets would limit the ability to prepare satisfactory samples. The large relief of total stress on sampling, and the possibility of gas exsolution could also make saturation difficult.

5.3.6 The Fugro investigations showed significant variations in soil composition with depth at the Magnus site. Changes in grading, fabric and index properties suggested that it was not feasible to prepare a constituted soil mix to model all the layers present, and it was decided to only use material from depths between 30 and 70 metres. The poorer quality samples were dried for the first batch and, as the

triaxial tests on samples of good quality were completed, extra material became available. Some 50 mm diameter cores from the Magnus site had been held in storage by McClelland Engineers since sampling in 1977, and soil from the appropriate depths was broken down to supplement the material available for the reconstituted test programme. Complete consistency in the reconstituted soil mixes was in fact difficult to achieve.

5.4 LIMITATIONS TO THE LABOARATORY TEST PROGRAMME: SAMPLES FROM CANONS PARK

5.4.1 In contrast to the Magnus site, Canons park was readily accessible and only the top 10 metres of soil were of interest. The absence of previous borehole investigations led to the requirement for a detailed inspection of the soil layers. A drilling team was provided by the Building Research Establishment (BRE) and, under the author's direction, two 10 metre deep bore holes were formed with continuous sampling. Thin steel walled sample tubes, 102 mm in diameter and 700 mm long, were taken using a push in action. After retrieval, any soft material was removed from the top of the tube and both ends waxed before transport to Imperial College.

5.4.2 Observing the sampling operation, it could be seen that the conditions at Canons Park were far from uniform. Below a gravel layer, the London clay was often fissured and frequently contained horizontal laminae of sand. Claystone bands were also encountered, and it was apparent that trimming 38 mm diameter intact samples would be extremely difficult.

5.4.3 It is well known that the undrained shear strength of London clay depends on sample size, rate of testing and method of sampling Bishop et al. (1965), Marsland (1971), Sandroni (1977). In order to obtain consistent profiles of intact properties with depth it was decided to test the specimens at their sampled diameter. A suitable conventional triaxial cell was at hand, but anisotropic consolidation or stress path testing was not feasible with the 102 mm samples.

5.4.4 Reconstituted London clay could, however, be investigated under more general conditions using '38 mm' triaxial apparatus. The equipment employed for the Magnus test programme was available for a short period, as were two further Bishop and Wesley cells, one being operated by Fourie (1984), the other by Maswoswe (1985).

5.4.5 At the time of the Canons Park site investigations Fourie was carrying out analyses of the Bell Common tunnel, which was being cut at another London clay site. The volume of samples available from Bell Common was limited, and Fourie considered it advantageous to participate in the Canons Park studies. The batch of material used for the reconstituted test programme was mixed with samples taken at both sites, and the merging of the laboratory programmes gave considerable savings in time and resources.

5.5 DESCRIPTION OF TESTING APPARATUS

5.5.1 The following pages describe the equipment employed for the two test programmes. To save space, no mention will be made of the standard apparatus used for gradings and Atterberg limits, or the standard oedometers. The conventional triaxial equipment will be discussed first, then the hydraulic stress-path cells, and finally the large oedometer used for the preparation of reconstituted soil.

5.5.2 The conventional triaxial apparatus for testing 102 mm diameter samples was similar to that described by Bishop and Henkel (1962) and Figure 5.2 gives the general arrangement of the equipment. The perspex cell was banded to withstand pressures above 1,000 kpa, and there was sufficient annular space to allow the attachment to the sample of pore pressure probes or displacement transducers. The deviator stresses were determined internally with an Imperial College 4,500 N load cell. Provision was made for both base and mid height pore pressure measurements, with 1,000 kpa transducers mounted in deairable brass blocks connected to the base plate. The base pressure was monitored through a fine ceramic disc let into the pedestal and the mid height pressure through a ceramic probe similar to that used by Maguire (1975). Fine bore stainless steel tubing was used to connect the probe to its monitoring and deairing circuit. A 1,000 Kpa transducer for cell pressure measurement was located in a brass block

adjacent to the cell inlet valves. The sample displacements were observed externally by means of a dial gauge, and electrolevel transducers were used to directly monitor the axial straining of the soil. As the tests in the 102 mm cell were all carried out without drainage or the application of back pressure, a volume measuring device was not required.

5.5.3 The cell pressure and deaired water supply were controlled through a christmas tree valve network. The standard self-compensating mercury pressure system was employed and loading was applied through a 5 tonne loading frame. The servo-controlled electric motor was reduced through a gearbox to allow rates of displacement between 0.0015 and 75 mm per hour.

5.5.4 Three conventional triaxial sets were available for the studies, but as the narrowness of the cells would not permit the attachment instruments to the samples, two of these apparatus' were used for pilot tests only. The third set was converted to take a larger cell, which was able to withstand pressures up to 1,000 kpa. The systems for cell pressure control measurement and water supply were similar to those used for the 100 mm apparatus. Top and base drainage was provided, with the lines meeting at a pore pressure transducer block connected to the base plate. A 1,000 kpa transducer was used, and the drainage leads could be isolated for undrained test stages, or connected to a back pressure system through a paraffin-water burette volume-gauge. All the exposed drainage lines were made from Saran tubing, which is particularly stiff and yeilds less that 10^{-5} cm^3/metre for a 1 kpa change in pressure.

5.5.5. In addition to the pressure transducers, an internal load cell was fitted, as were electrolevel strain measuring transducers, an external dial gauge and an external displacement transducer. A pore pressure probe was not employed, and loading was applied by a 1 tonne frame with an electrical motor acting through a gear box. The equipment was capable of applying displacement rates between 91.2 and 0.0073 mm per hour.

5.5.6 A total of five sets of hydraulic triaxial apparatus were used for the work described in this thesis. All were coverted to have

'enlarged tops' which allowed local instrumentation to be mounted, and the cells differed only in their control systems. Of the 35 stress-path tests discussed in Chapters 6 and 7, all but three were carried out in the apparatus operated by the author, and only these will be described here. Readers are refered to Maswoswe (1985) and Fourie (1984) for details of the other control systems.

5.5.7 The design and operation of the stress path testing apparatus is fully described by Bishop and Wesley (1975) and permits any desired stress paths to be conveniently followed in triaxial space. Both the axial and radial stresses are hydraulically controlled, and the cells are self contained without the need for a loading frame[1].

5.5.8 Control of the stress paths followed by samples requires the simultaneous manipulation of both cell and ram pressures. For nearly all the tests, this was achieved through a mercury self compensating system adopted from Bishop and Henkel's original design. The ratchets were disconnected and the cable winding drums of the conventional system were linked to electric motors through a two stage, high reduction, gear box system as shown in Figure 5.4. The equipment is described in detail by Gens (1982), who developed procedures for using calibrated D.C. servo-amplifiers to accurately follow general stress paths. However, the mechanical arrangements were cumbersome, and the way in which the cable wound onto the drum effected the pressure variations. By 1982 the gear boxes and motors were reaching the end of their service life and were scrapped, after many breakdowns, at the end of the work described here. Control of the motor speeds and gears allowed pressure rates between 10^{-5} and 10^{2} kpa per minute to be selected.

5.5.9 It was anticipated that higher pressures than usual would be required for the tests on intact Magnus soil. For this reason two of the three sets were converted to give controllable pressures in the extended range of 500 to 1,500 kpa. This involved adding extra limbs to the mercury control lines and purchasing new transducers to monitor the ram and cell pressures.

5.5.10 The shear phases of almost all the tests were carried out

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[1] The general arrangements for these cells is shown in Figure 5.3.

under conditions of displacement control. This was done by connecting the chamber drive unit to one of two screw ram pumps. The pumps were of different design, but both were driven by regulated electric motors acting through gear boxes. For most of the shearing tests the motors were set to give an axial strain rate equivalent to 4.5% per day.

5.5.11 In some of the later tests on the Magnus and London clays a digitally governed pressure source was used to provide the axial stress control. A stepper motor was connected to a manostat air valve through a gear reduction, in such a way that a single pulse gave a pressure increment of ±0.07 kpa. Oscillator circuits provided pulses at a controllable rate of between 50 pulses per second and 1 pulse per 1,000 seconds. The regulated air pressure was applied to the chamber through an air-water interface and gave greatly improved control for stress path tests. Whilst the old mechanical winding systems were noisy, cumbersome, inaccurate and could not be reversed without lengthy 'backlash' intervals, the new controllers responded precisely and instantly. These systems are now standard equipment at Imperial College.

5.5.12 In addition to the conventional rear loading oedometers, a large 229 mm diameter cell was employed to anisotropically consolidate samples from a 'slurry' condition to produce blocks of a firm consistency. The quality of instrumentation was thus much lower than described above. The load was applied to a rigid top plate by a second-order lever system and a loading yoke. The knife edges of the levers ensured vertical loading even if the loading beam was not horizontal. Top and bottom drainage was provided through coarse stones, and it was feasible to produce blocks of both Magnus and London clay with mean undrained shear strengths of approximately 50 kpa using the two week staged loading programme given in Table 5.2.

5.5.13 De Campos (1984) carried out an investigation of the permeability of the triaxial testing membranes offered by a number of suppliers. He concluded that those offered by WYKEHAM FARRANCE (WF 10500) provided the lowest permeability, and these were used throughout the described testing programmes. De Campos also recommended soaking the membranes before use, but as this suggestion was made at a late stage the work, it was not adopted by the author.

5.6 INSTRUMENTATION

5.6.1 In the descriptions of the triaxial equipment reference has been made to load cells, pressure transducers, displacement transducers and electrolevel gauges. The electrolevel devices were developed for the Magnus test programme, and are discussed in detail in section 5.7. The characteristics of the other transducers, and their calibrations, are briefly discussed in the following paragraphs.

5.6.2 Throughout the testing programme Imperial College load cells were employed. The principle of operation has been described by El-Ruwayih (1975). The deflections of a strain gauged triangle, housed in an oil filled cavity, is directly related to the deviator force in a triaxial test. The strain gauges are connected to form a full bridge circuit which is highly sensitive to deviator force but unaffected by horizontal forces, eccentricty of loading or temperature changes.

5.6.3 Load cells of 4,500 N capacity were used in all the triaxial sets and, for this range, the strain gauged 'stars' are made of maraging steel. During the earlier stages of the investigations there were considerable difficulties with drifting load cell outputs, which were eventually related to a chemical reaction between Manganese traces in the steel and a phosphoric acid strain gauge preparation reagent. Changes in gauging technique overcame this fault and the modified cells performed satisfactorily throughout the remainder of the programme.

5.6.4 Two kinds of electrical transducer were employed to measure the ram, cell and pore water pressures. The more common was the Bell and Howell fluid pressure gauge, which uses unbonded strain gauges connected to a pressure sensing diaphragm to give an accurate and stable device. Most of the gauges used in this study had a maximum range of 1,000 kpa, but semiconductor diaphram transducers of 2,000 kpa capacity were substituted to cope with the higher pressure systems employed with two of the stress path cells. The devices were made by Senso-metrics Inc. of California, and gave excellent performance throughout the test series.

5.6.5 The transducers employed to measure external displacements were supplied by Minster Precision Engineering. They consist of a stainless steel shell which houses a pair of strain-gauged cantilever arms, the deflections of which are dependant on the linear movement of a tapered wedge. The devices had a range of 25 mm and gave a completely satisfactory performance.

5.6.6 Piezometer probes of the type described by Hight (1983) were mounted in single tests on the reconstituted London and Magnus clays. Their use helped the selection of rates of stress change during Ko consolidation, and axial strain during shear. The Druck piezometer probe incorporates a miniature silicon diaphragm into which is diffused a full semi conductor strain gauge bridge. The devices offer excellent linearity in calibration, but need careful deairing. In the case of the probe employed for the Magnus test, a cross sensitivity to radial effective stress was noted and, after contact with the manufacturers, the devices design was slightly amended. An increased gap between the porous stone and the diaphragm, and a thickening of the metal casing helped to eliminate the cross sensitivity.

5.6.7 All the electrical transducers were connected to one of the two Solartron Data Transfer Units (DTU's). These stations provided a constant current power supply (10 mA) and performed the data digitisation and recording. Readings could be taken at rates between 1 per second and 1 per 4 hours, and the data punched on tape or continuously printed out. Except for the printers, the DTU's were reliable, but are now being replaced by more versatile 'Orion' systems.

5.6.8 The dial gauges employed were of the standard kind, with full scale ranges of 12 mm and resolutions around 0.005 mm. The paraffin-water volume gauges were as described by Bishop and Henkel (1962). The devices employed had a total capacity of 5 cm^3 and changes of ± 0.01 cm^3 could be resolved, providing the meniscus remained stable. The laboratory in which the tests were conducted was temperature controlled to give ± 1°C.

5.6.9 The electrical instruments were calibrated at the start of the investigations and were regularly checked for response and 'zero'

drift. For the pressure and force tranducers, a Budenberg dead weight tester was used which was fitted with electric motors to slowly spin the calibrated piston and ram cylinders. The system was thus free from friction and stick-slip problems. The displacment transducers were bench-tested using a high accuracy vernier micrometer.

5.6.10 For each device, the calibration cycle typically consisted of a load,un-load loop with 20 points being recorded. With the pressure and displacement gauges this was simply done by covering the range expected in the experiments. With the load cells, both negative and positive deviator forces had to be considered. To cope with 'tension' tests the transducer was suspended from a frame and weights were loaded onto a hanger connected to the load cell base. A further complication with these instruments was the need to assess their compliance so that external axial strain measurements could be corrected for the deflection of the central star under load. This was carried out my mounting a steel dummy sample in the triaxial cell and observing the outputs from the load cell and displacement transducers as cycles of deviator force were applied.

5.6.11 The calibration data were analysed using an HP 41C calculator and a statistics package. Linear regression routines were run to find best fitting straight line relationships between voltages and force, pressure or displacement. For each of the recorded points the deviation from the determined relationship was calculated and the mean errors assessed. Table 5.3 summarizes typical values of sensitivity, resolution and accuracy[1] for the transducer types employed. Figure 5.5 shows a typical load-deflection characteristic for a 4,500N load cell, and it can be seen that the instrument's compliance cannot be neglected.

5.6.12 Calibrations were also required for the strain and pressure control systems of the hydraulic triaxial apparatus. The transducers mounted on the stress-path cells were used to relate the motor control settings to rates of strain and stress change. These relationships were checked continuously during the course of each test.

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[1] defined on Table 5.3

5.7 THE MEASUREMENT OF SOIL STIFFNESS IN THE TRIAXIAL APPARATUS

5.7.1 Accurate determinations of soil stiffness are difficult to achieve in routine laboratory testing. Conventionally, the determination of a triaxial sample's axial stiffness is based on external measurements of displacement which include a number of extraneous movements; the true soil can be masked by deflections which originate in the compliance of the loading system and load measuring system. Such errors add to a variety of sample bedding effects to give a poor definition of the stress-strain behaviour of the material under tests, particularly over the small strain range. Most triaxial tests therefore tend to give apparent soil stiffness far lower than those inferred from field behaviour.

5.7.2 The importance of such errors has long been recognised and many diverse techniques have been employed in attempts to improve strain measurements. One solution has been to measure relative displacements between two reference footings over a central length of a sample using displacement transducers e.g. (Yuen et al 1978, Daramola, 1978, Brown et al, 1980, Costa Filho, 1980). Strictly these techniques are only suitable for very small strain levels, as bulging of the sample will cause the footings to rotate in the later stages of each test. Although important results have been obtained with such techniques, they are cumbersome and can suffer from jamming and damage at large strains.

5.7.3 X-ray and Optical methods have also been used to follow reference points within the sample or on its membrane, Roscoe et al. 1963, Arthur and Phillips (1975). However the accuracy of these mothods is limited, with resolutions of around ± 0.1%.

5.7.4 The resonant column apparatus offers a different approach for the determination of the dymanic stiffness of soils. The technique involves the application of periodic small strain perturbations to a sample, as described by Richart et al (1970). However the technique does not provide direct measurements of the elemental behaviour of the soil under test, as the states of stress and strain vary continuously both with time, and in their distributions within the sample.

5.7.5 In summary, the available methods of soil strain measurement suffered a number of serious limitations. There was an urgent need for a simple but precise method for the routine measurement of the stress-strain behaviour of soil specimens under controlled stress or strain paths, particularly as the soils were expected to exhibit high stiffness at small strains.

5.7.6 The following sections discuss the sources of error in axial strain measurements, and then describe the development of the electolevel strain measuring system. As the laboratory programme was to be concentrated on undrained tests, the no volume change condition obviated the need to measure radial strains. However, in tests designed to investigate more general efective stress behaviour, the local measurement of radial and axial strains are equally important. Symes (1983) describes the use of proximity transducers for radial strain determination and Maswoswe (1985) describes the use of a high accuracy LVDT for the same purpose with 38 mm triaxial samples.

5.7.7 In a conventional triaxial test there are several sources of movement that develop during shear testing and which may cause the overestimation of axial strains. One such source is the compliance of the loading system itself, and for present purposes this will be termed Δram. As discussed in Section 5.6.11 an internal load cell wil also produce a significant deflection, which can be termed Δ L.

5.7.8 The more important sources of error are illustrated in Figure 5.6, some of the deflections shown in this Figure may be quantified by careful calibration, but large unaccountable errors remain due to:

1) the inability to trim a sample whose end faces are perpendicular to the vertical axis of symmetry;
2) the inability to rigidily connect the load cell to the sample top cap, and;
3) the inability to avoid bedding at the ends of the sample, due to local surface irregularities or voids.

5.7.9 To circumvent these difficulties in the testing of rock samples it is common to specify the careful grinding of sample ends combined

with the use of ground cylindrical seated plattens (e.g. Vogler and Kovari, 1978). The observation that many rocks fail in a brittle fashion at axial strains of 0.1% or less has led to the specification of flatness limits of ± 0.01 mm and parallelism requirements of around 3 minutes of arc for high quality sample preparation. However, The preparation techniques employed for rock testing are unsuitable for most soils, and it is probably not possible to approach the same standards of sample regularity.

5.7.10 Measures can be taken to reduce the errors implicit in external strain measurement. The results obtained from tests carried out on soil which has been anisotropically consolidated to a high level of mean efective stress suggests that these procedures considerably reduce sample bedding and tilting errors, see Gens (1982). The SHANSEP methods of testing soft clays can also be expected to lead to significant improvements in strain measurement. However, where swelling stages are included in such tests, tilting and other errors may redevelop, Daramola (1978) and Costa Filho (1980). Moreover, it is often desirable to obtain accurate strain measurements in tests which have not involved such anisotropic consolidation stages.

5.8 THE DEVELOPMENT OF THE ELECTROLEVEL TRANSDUCERS

5.8.1 A more satisfactory approach is to make use of local instrumentation which can be attached to a central gauge length of the sample. Symes and Burland (1985) have given a description of the design of instruments which employ electrolytic levels to measure combined horizontal shear strain and axial strain in a hollow cylinder apparatus. The same principles were adapted to develop vertical displacement measuring systems for use in a large diameter triaxial apparatus (see Burland and Symes, 1982). A further development and improvement of the earlier devices was made for the work described in this thesis which enabled mean axial strains to be determined, with a resolution of ± 0.001%, in triaxial stress path cells designed for the testing of 38 mm diameter samples.

5.8.2 Cooke and Price (1974) describe the use of electrolytic liquid levels for the local measurements of ground strains around test piles.

Their reliability, simplicity and accuracy make these transducers attractive in a wide variety of applications, and by mounting the capsules in simple mechanisms it is possible to develop a range of devices to measure axial, radial and shear strains in laboratory tests.

5.8.3 The liquid level transducers consist of an electrolyte sealed in a glass capsule. In the simplest devices three coplanar electrodes protrude into the capsule and are partially immersed in the electolyte. The impedance between the central electrode and the outer ones varies as the capsule is tilted. A variety of levels with different sensitivities are commercially available.

5.8.4 The electrolevel transducers employed were supplied by IFO International Ltd and have a working range of ±10°. The system was excited using a 5V AC power supply of 4KHZ frequency. The gains were adjusted to give a ± 2 V full scale output which was monitored to ± 0.1 mV with typical scatters of ± 0.1 mV. The levels are sensitive to temperature and vibration and should be operated in still environments which are temperature controlled to within ± 3°C. Under such conditions the gauges can be stable over periods of weeks.

5.8.5 The principle of the strain measuring device is essentially that a hinged arrangement converts displacements between two footings mounted on the sample into a rotation of the capsule, as shown in Figure 5.7.

5.8.6 The major difference between the instruments described here and the earlier designs lies in the geometrical configuration which permits their use on 38 mm diameter samples and Figure 5.8 shows the detailed construction. In addition to the geometrical changes, the hinge mechanisms have been improved by replacing the original brass pivots with polyfluorotetraethylene (PFTE) and by simplifying the construction of the hinges themselves. The capsule which protects the electolytic level from the action of pressure and water is constructed from stainless steel, as are the tubular arms BC and AC. The gauges are fully submersible and have been tested at pressures of up to 1500 kPa. The electolevels are mounted in diametrically opposite pairs on a sample using a rapidly curing contact adhesive which bonds the brass footings to the membrane. The gauges rely on the

radial effective stresses to anchor the footings to the sample under test.

5.8.7 It should be noted that if the sample tilts when loaded the output from each gauge is made up of a strain component and a tilt component as shown in Figure 5.9. Provided the sample is homogenous, the mean axial strain is given by half the sum of the outputs of a pair of diametrically opposed gauges, and the tilt is given by half the difference of the outputs. The ability to detect sample tilt is valuable feature of the gauge.

5.8.8 The gauges were designed to permit the resolution of around 1 μm and a working range equivalent to the maximum travel of the stress path cells. Having produced prototypes, it was necessary to examine their resolution and accuracy in calibration.

5.8.9 The resolution and range of the gauge was determined by a two part procedure. Firstly, routine calibrations were performed over a displacement range of 15mm by mounting two opposing gauges on a micrometer winding frame graduated to 0.1mm. A typical gauge-length voltage characteristic is presented in Figure 5.10. A third order polynomial regression analysis can then be used to model the characteristic, with a typical correlation coefficient of 0.99999 within the limits shown. To determine the resolution a second stage of calibration was carried out by mounting a high precision, small travel, LVDT on the central axis of the winding frame so that the changes in output could be determined for small displacements which were known to within ±0.1μm. Using such techniques the gauges were seen to be capable of resolving to less than 1μm as shown in Figure 5.11. The scatter visible in the very small deflection range mostly reflects fluctuations in the signal conditioning unit, and definition could be improved by averaging a larger number of data sets.

5.8.10 Following the first calibration, checks were made to assess the repeatability and degree of scatter found over a smaller range of travel. The instruments were cycled over a range of ±0.1 mm, and it should be noted that these deflections corresponded to ±0.2% axial strain over the 50 mm typical gauge lengths. The displacements relative to the initial position were calculated from the response of the

electrolevels and the LVDT, and Figure 5.12 shows the scatter of data recorded over the top 0.025 mm of three cycles. The mean trend line lies almost exactly on the line of equality with 80% of the data points falling within limits if ±1μm. The results point to the importance of obtaining as many points as possible in the early stages of tests, and show that trend lines could be drawn through scatters of data to provide accuracies of between 0.001 and 0.002% axial strain.

5.8.11 The original purpose of developing the electrolevel devices was to produce precise equipment which is easy to use. The LVDT systems of Daramola (1978) and Costa Filho (1980) were connected to the soil samples by pins passing through the membranes. It was hoped that this difficult procedure could be avoided by simply bonding the footings to the outside of the membrane. Gens (1982) had carried out careful tests in which he followed markers on the membrane with a travelling telescope, and had concluded for Lower Cromer Till that the membrane follows the soil until large strains develop. It was decided to check this finding by carrying out a programme of preliminary tests on samples of chalk.

5.8.12 Brooks (1983) obtained blocks of intact chalk from a quarry at Northfleet, Kent and had prepared a series of 38 mm diameter triaxial samples. The test programme described by Jardine, Brooks and Smith (1985) was intended to investigate the chalk properties and evaluate the new gauges. Three tests will be described here, A, B, and C. Tests A and C employed samples which had been saturated, and were compressed to failure without drainage from an initial isotropic effective stress of 350 kpa. Sample B was compressed in a drained condition from the same initial streses, but the specimen was partly dried so that metal-foil electrical resistance strain-gauges (ers) could be bonded axially to opposing faces of the chalk cylinder. A membrane was fitted over the instrumented sample and the leads let out through a rubber grommet which was sealed with small 'O' rings and three coats of latex. The ers gauges were wired into $^1/_4$ bridge circuits made from precision resistors, which were connected to the Solartron DTU system. Electrolevel transducers were mounted on the membranes of all three samples, and for test B the instruments were alligned with the ers gauges. Tests A, B, and C were carried out in the enlarged conventional '38 mm' triaxial cell.

5.8.13 The compression stages were performed with a nominal external rate of strain of 4.5% per day. However, the chalk was far stiffer than the load cell and the true rates of strain before failure were typically 0.2% per day. Tests A and C suffered brittle failures at strains of 0.07%, sample B was taken to the same strain but did not fail.

5.8.14 Samples A and C had been ground to give ends which were flat and parallel to within ±0.1mm. These limits fall far below the recommendation of Vogler and Kovari (1978) for rock testing, but are typical of the standard obtained by careful trimming of soil samples. A far rougher specimen was selected for test B, with variations in length of up to 0.4 mm, and it was expected that the strain response under compression would be particularly non uniform.

5.8.15 Pre-failure stress-strain curves for the samples are shown in Figure 5.13. The strains determined with the electrolevels for tests A and C show similar smooth lines that pass through the origin and indicate scatters that are generally within ± 0.001%. The straining within sample B is illustrated in Figure 5.14,where the traces of individual foil gauges and electrolevels are plotted. There is no sign of a strain lag near the origin, but loading evidently causes tilt and strain inhomogeneity in the sample. Tensile strains persisted until $(\sigma_1 - \sigma_3)$ reached 1,400 kpa, when the four strain indicators gave essentially parallel lines. The mean plots for the two instrument systems are given in Figure 5.15 and show identical trends, but the electrolevel trace initially wandered to either side of the strain gauge curve by up to 0.002% strain.

5.8.16 On unloading the data were far more consistent, and there was no measurable strain lag when the stress increments were reversed. Figure 5.16 illustrates the trends and shows the electrolevel gauges to be consistent to within ± 0.0005% strain. It is reasonable to conclude that the deviations observed during loading were related to irreversible processes that occured near to the sample ends as loading wrenched the sample into a more parallel alignment.

5.8.17 In summary, the calibrations and pilot test programme proved the electrolevel gauges to be capable of providing axial strain

measurements with an accuracy of ± 0.001% over the initial stages of a test. It was seen that data quality is improved by taking as many readings as possible and by careful sample preparation. The main advantage of this type of instrumentation is that the small strain behaviour of a material can be accurately investigated and yet the gauges are simple to mount and remain undamamged at strains of up to 35%

5.8.18 Finally, it is worth noting that there is scope for further development with the electrolevel systems. The experiments described were conducted on the 5th floor of a relatively flexible building, and vibrations from the laboratory, the ventilation system and the swaying of the structure would all have contributed to the scatter of the inclinometers. If the tests had been located in the basement, and the cells had been mounted on vibration reducing blocks, the accuracy could have been improved. Similarly, a reduction in the angular range of the capsules and the substitution of a tandem transducer arrangement for each gauge could improve the electrical stability by a factor of a least four. The use of higher quality signal conditioning and an increase in the rate of data logging could also upgrade the performance of the instruments.

5.9 TESTING PROCEDURES: TRIAXIAL TESTS ON INTACT LONDON CLAY

5.9.1 The following pages deal with the procedures adopted in the laboratory test programme. The most convenient method of presentation is to consider each of the main types of test in turn, starting with the conventional triaxial experiments on intact London clay.

5.9.2 Before extruding a 102 mm sample for testing a number of preparatory procedures were carried out.

(i) The rubber membrane was tested, with checks for leakage being made and a rubber grommet incorporated to retain the mid height probe. The latter action involved punching a 6.5 mm hole in the membrane, inserting the grommet and locally painting on three coats of liquid latex to ensure impermeablity.

(ii) The transducers and porous stones for the base and probe were deaired thoroughly and kept saturated prior to testing. A soaked filter paper disc was placed over the base stone.

(iii) Lubricated ends were prepared to prevent sample bulging during shear. Four discs of soaked membrane rubber, 9.5 cm in diameter, were formed and a number of radial cuts made to reduce their circumferential stiffness. Two sets of discs, with a layer of silicone grease between them, were assembled and the sides of the pedestal and the top cap were smeared with the same grease.

(iv) The electrolevel gauges were cleaned and a light oil sprayed on all three hinges. The surfaces of the gauge footings were de-greased and roughened with a semi-circular file.

5.9.3 A sample tube was brought from the store and placed in a vertical frame for extrusion using a hydraulic jack. After examining the first two samples it was clear that the top 20 cm of the core was generally softer than the remainder, presumably due to sampling effects. Care was therefore required to obtain a sample of 20 to 25 cm length from the lower two thirds of each tube. The test specimen was taken by cutting the core horizontally at the sampler edge, using a sharp fine tooth saw. This operation was made difficult on occasions by the presence of laminae and partings of sand. The remaining soil was either cut into sections and waxed for oedometer testing, or split to allow a full description of the geological profile.

5.9.4 The specimen was placed in a split mould and the ends trimmed flat and parallel to the fixed length of 198mm. Moisture content samples were rapidly taken from the trimmings, and the specimen was weighed and measured before being placed on the lubricated ends of the triaxial cell pedestal. The sample was jacketed with a rubber membrane, using a stretcher and suction tube, so that the rubber grommet was suitably alligned for the mid height probe. The upper set of lubricated ends were positioned and the perspex top cap installed. Any trapped air was smoothed from under the membrane and six 'o' rings were used to secure the assembly at the pedestal and top cap.

5.9.5 A 7.3 mm diameter hand drill was inserted through the rubber grommet and a hole approximately 15 mm deep was formed in the sample. The damp pore pressure probe was then pushed into the cavity until refusal, and small 'o' rings were used to compress the grommet on to

the brass casing of the probe. Three further coats of latex were locally applied to guard against leakage.

5.9.6 This done, the electolevel gauges were mounted. Diametrically opposed lines were drawn on the membrane, and positions marked for the footings at the sample mid-height, such that the instruments would have adequate travel for the shear test. The electrolevel footings were cleaned again with Butanone, as were the target areas of the membrane. The gauges were then attached using the two part instant adhesive M-BOND 200, with catalyst being applied to the membrane and a single drop of glue being smeared on each footing. Once contact had been made, the gauges were supported by the membrane. After allowing 1 hour for the glue and latex to cure, the top of the cell was lowered over the sample and the cell filled with deaired water. A cell pressure of approximately 650 kpa was immediately applied and the sample left for 24 hours to allow stabilisation.

5.9.7 The equilibration of the sample was monitored using the mid height pore pressure probe and, when the pore pressure was stable to within a drift rate of ± 1kpa per hour, a 100 kpa increment in cell pressure was applied undrained. The response was used to calculate the pore pressure coefficient B. If this did not quickly reach a minimum of 0.98, the sample was left for a further 24 hours and the saturation check repeated. This procedure was successful in every case and gave equilibrium pore pressures of around 600 kpa before shearing.

5.9.8 With the sample in a state of undrained equilibrium, the load cell was slowly brought into contact with the top cap and the loading frame set in motion. The externally applied rate of displacement was equivalent to 4.8% strain per day, and this velocity allowed the complete effective stress path to be reliably plotted using the pore pressures monitored with the mid height probe. However, the base measurements could not be considered reliable. At the start of each test the Solartron DTU recorded data sets at 1 minute intervals. This frequency was gradually reduced as the test progressed. When the shearing was complete, after three to four days, the readings were being taken once per hour.

5.9.9 At the end of the test the cell was quickly emptied and the

sample stripped out. The clay was weighed, and moisture content samples taken. These checks indicated no measureable changes caused by membrane leakage, or setting up procedure. The failed specimen was sketched and the broken material used to complete the description of the soil profiles in the boreholes.

5.10 TESTING PROCEDURES: TRIAXIAL TESTS WITH RECONSITUTED LONDON CLAY.

5.10.1 The first stage for the reconstituted soil was the reduction of intact samples to dry powder. For each mix, approximately 10 kg of soil was air dried, ground and passed through a 420 μm sieve. Atterberg tests were then carried out to determine the volume of water required to mix a slurry at a liquidity index 1.25. The selected water content, 83%, gave the wettest possible initial state from which block samples could be produced with sufficient length for '38 mm' triaxial testing.

5.10.2 The slurry was stirred in a large electrical mixer for around 45 minutes and then carefully spooned into the 229 mm oedometer. Before this operation the porous stones had been deaired and the side walls lubricated with silicone grease. The oedometer cell was mounted in its loading frame, the top cap gently placed over the slurry and a small weight placed at the end of the lever arm. The consolidation programme given in Table 5.2 was then followed, and after 15 days a cake of firm clay with OCR 2.0 had been produced. This was taken from the oedometer by rapidly unloading the cell and disconnecting the base plate from the side walls. The cake was then gently extruded and cut into blocks, which were wrapped in cling film and coated in wax. Approximately fifteen 38 mm diameter samples could be obtained from each cake.

5.10.3 The procedures carried out before testing the reconstituted clay were similar to those used for the intact samples. The loose porous stones used in the 38 mm apparatus were boiled, the transducers and their lines deaired, and a membrane prepared. In only one case was a pore pressure probe employed, and this required a small rubber grommet to be incorporated into the membrane.

5.10.4 The samples were trimmed from the consolidated blocks by cutting an oversize prism of soil with a thin wire, and then progressively reducing the soil to give a cylindrical specimen, 38 mm in diameter. This operation would typically take 5 minutes and, when finished the cylinder would be placed in a split mould and trimmed to a length of 78 mm with a thin wire. Moisture contents were taken, the sample weighed and its dimensions measured with a vernier calliper.

5.10.5 A saturated porous stone was slid through the water meniscus on the base pedestal and excess fluid removed with a paper tissue. The specimen was mounted on the stone, and, after greasing the pedestal, the membrane was placed over the sample. With the top drainage system saturated, the upper porous stone and perspex cap were connected to the sample and 'o' rings used to secure the membrane at the top and bottom. Six of the Ko consolidated samples were required to sustain greater radial stesses than axial, and for these tests a rubber suction cap was placed over the perspex top cap as shown in Figure 5.17.

5.10.6 In all the tests, a pair of diametrically opposite electrolevel gauges were attached as described in section 5.9.6, and Figure 5.18 shows a photograph of one of the gauges at this stage. The top of the stress path cell was lowered into place and the cell filled with deaired water. A pressure of around 500 kpa was immediately applied and the sample left to stand undrained for at least 24 hours. Figure 5.19 shows a 38 mm diameter sample installed in a stress-path cell, before any deviator stress had been applied.

5.10.7 Similar saturation checks were made to those described for the '102 mm' samples and adequate response was invariably proved at the end of the 24 hr test period, with the mean effective stress typically reaching 90 kpa. From this stage the specimens were either brought to a isotropic initial effective stress condition for unconsolidated, undrained (UU) tests[1], or anistropically consolidated for undrained testing from nominally K_o conditions. The latter category will be termed K_oCU tests.

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[1]As many of the UU tests included slight adjustments in their initial effective stresses they are not strictly unconsolidated,

5.10.8 Before commencing a UU experiment the initial effective stress in the sample was brought to 90 kpa. The cell or back pressure was adjusted to provide this condition and the sample allowed to drain or swell, with observations being made with the volume gauge and the inclinometers. When the primary swelling or consolidation was complete and the axial creep rate less than 0.005% per hour, the drainage valves were switched off and the sample left for 2 hours to equalise. Over this period the pore water pressures drifted by no more than 2 kpa.

5.10.9 The ram position was then adjusted to raise the sample until it touched the load cell and the shearing phase commenced. A constant rate of strain could be applied by either the mechanical drive of the 38 mm conventional apparatus or the ram pumps connected to the stress path equipment. In two of the tests, the digitally controlled pressure source was employed to provide a constant rate of axial stress change with a stress path cell. As will be discussed in the next chapters, various rates of testing were used for the UU series.

5.10.10 After shearing to a preset stress or strain level, the samples were stripped out and moisture contents taken.

5.10.11 A different procedure was followed for the K_oCU tests. After the saturation stage the hydraulic triaxial cells were controlled to follow the stress path A, B, C, D shown in Figure 5.20. Point A represents the initial condition of the sample after setting up. Prior to this the soil had been consolidated from slurry (I) to the maximum stress condition (II) and swelled back to III in the large oedometer before the transfer to the triaxial cell. The path AB represents triaxial drained compression and portion BC provides a transition to the Ko consolidation line CD. The latter line was assumed to follow $\sigma_3'/\sigma_1' = Ko$ and was calculated from the Jaky expression with $\phi' = 23.5°$;

$$K_o = 1-\sin\phi' \qquad \text{Eq. 5.1}$$

5.10.12 All eight K_oCU tests were consolidated to give $\sigma_a' = 400$ and $\sigma_R' = 240$ kpa. Such an increase above the preconsolidation stresses should produce test conditions which are free from the influence of

sampling or setting up procedure, Gens (1982). Path DE represents the assumed K_0 stress path in swelling, and was followed as far as the required final stresses for the six overconsolidated experiments. The path was defined by reference to Costa-Filho (1980) and the expression given by Mayne and Kulhawy (1982). Section 5.10.8 discusses the radial strains developed during consolidation on the selected Ko paths, and good general agreement is noted.

5.10.13 When consolidating or swelling it is important to maintain an appropriate rate of stress changes. Radial drains were not employed for these experiments, and to complete the programme within a feasible length of time, slight gradients of pore water pressure within the samples had to be accepted. However, if these gradients became steep, the non-uniform effective stress conditions could cause the samples to bulge or even fail. Although the test programmes of Costa-Filho (1980) and Gens (1982) provided guidance for the consolidation rates, it was decided to carry out a pilot test equipped with a Druck mid-hieght piezometer probe.

5.10.14 The observed dependence of excess pore pressure on the rate of axial stress change during consolidation is shown in Figure 5.21. Relatively rapid changes could be applied without difficulty up to the point B defined in Figure 5.20. Beyond this stage the rate of axial stress change had to be reduced, and it was found that the pore pressure difference could be limited to around 3% of the axial effective stress if $\dot{\sigma}'_A$ was maintained at 20 kpa/day. Following these rules, the sample could be consolidated from A to D in approximately 12 days.

5.10.15 When point D was reached the stresses were held constant, but the dissipation of the remaining excess pore pressures and the early stages of secondary consolidation combined to give high rates of axial strain. After a four day 'rest' period the settlement rate had reduced below 0.005%/hour and the samples could either be sheared undrained or swelled back to a higher overconsolidation ratio.

5.10.16 The pilot test was used to assess appropriate stress change rates for the swelling path, and the data are presented in Figure 5.22. The plot again shows that rapid changes in $\dot{\sigma}'_A$ are feasible ini-

tially. However, for overconsolidation ratios greater than 2.0, $\dot{\sigma}'_A$ should be reduced to around 20 kpa/day. In total, approximately 10 days were required to swell from the maximum stress to reach OCR 7.

5.10.17 The Ko consolidation paths could not provide true Ko conditions throughout the sample, as finite excess pore pressures developed between base and mid height. Similarly, the stress control systems were manually maintained and gave deviations from the Ko lines due to friction in the stress path cells' linear bearings, motor variations, uneven cable winding and operator error. Typically, the effective stresses at the ends of the sample were kept to within 5 kpa of the desired values.

5.10.18 Radial straining could be expected to develop over the initial stages of the consolidation path, but to reduce as the Ko line was approached. Figure 5.23 shows the typical progression[1] for ϵ_A/ϵ_V which should tend to 1.0 for conditions of zero radial strain. The ratio does indeed fall towards unity, but the plot indicates slight bulging and deviations from the desired conditions. On the swelling path, ϵ_A/ϵ_V continues to fall, and even drops slightly below 1.0. From these data it may be concluded that the assumed stress paths gave approximately Ko conditions, with no signs of instability or sharp change in radial strain.

5.10.19 Once the required stress state had been obtained, the samples were left to settle until the axial strain rate fell below 0.005%/hour. The drainage valves were shut and a further two hours allowed for equilibrium to be reached. A ram pump was then used to provide an axial strain rate of 4.5% per day and the data collected in the usual way, using the Solartron DTU. The piezometer probe had been used to assess the difference in pore pressure between base and probe for undrained tests at this rate of strain, and the results are shown in Figure 5.24. Negligible discrepancies were observed up to 0.1% axial strain and the maximum, which occured at 1.0%, amounted to 4 kpa.

5.10.20 The shearing stages of the tests were continued until the

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[1] The volume was calculated from the overall changes (monitored externally) whilst the axial strain was monitored the central two third of the sample.

full travel of the cell had been taken up, which generally allowed strains of 20% to be obtained. The samples were then stripped out, and moisture contents taken. Following the described procedures, each KoCu test took between 24 and 40 days to complete.

5.11 TESTING PROCEDURES; TRIAXIAL TESTS WITH RECONSTITUTED MAGNUS CLAY

5.11.1 The laboratory programme with the Magnus clays started 18 months before the London clay experiments, and the testing procedures for the two reconstituted soils were almost identical. For brevity, only the differences between the two programmes will be considered in this section.

5.11.2 The soils used for the Magnus programme were generally less plastic than the London clay. It was thus possible to prepare blocks of reconstituted soil with adequate final thickness starting from a slurry mixed at the slightly higher liquidity index of 1.5. Figure 5.25 shows the ingredients of the first oedometer cake in histogram form and, as explained in section 5.6.3, efforts were made to maintain this composition in subsequent mixes. Initially, there was concern that the fixed strain consolidation cell and the 'sampling' procedure might produce variations in water content within the prepared cakes. A slice from the first block was carefully divided to examine the distributions of water content, and the contour diagram given in Figure 5.26 shows only minor non-uniformities. Clay on the centre lines tends to be slightly drier, and there is evidence of water being drawn into the upper and lower boundaries during 'sampling'.

5.11.3 The reconstituted blocks were prepared using the same vertical loading sequence as the London clay (see Table 5.2). The triaxial tests could be similarly divided into those carried out in the conventional 38 mm cell, and those performed in the stress path cells. In both cases 76 by 38 mm samples were used. The Magnus programme was more extensive than the London clay series; in addition to UU and K_oCU tests[1], experiments were carried out on isotropically consolidated soil

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[1] Again the tests were not strictly unconsolidated as the initial effective stresses were contolled

and material that had been perfectly sampled from K_o conditions. Shear reversal tests were also performed with a number of samples. In each case the control systems and principles of testing were similar to those employed for the London clay experiments.

5.11.4 The UU tests differed only from those previously described in the selection of the nominal initial effective stress, 70 kpa, and the use of lubricated ends in some of the experiments. The free ends were prepared from 38 mm diameter discs of membrane rubber as detailed in section 5.9.2. Electrolevel gauges were used in all but one test.

5.11.5 The K_o consolidated samples were taken to maximum conditions of σ'_A = 400 kpa and σ'_R = 200 kpa by following the sequence given in Figure 5.27. The consolidation and swelling effective stress paths were evaluated from the expression of Mayne and Kulhawy (1982), which Gens (1982) had found to be appropriate for low plasticity clay;

$$K_o = (1-\sin\phi)\ OCR^{(\sin\phi)} \qquad \text{Eq. 5.2}$$

5.11.6 The first K_oCU test with the Magnus soil was equiped with a Druck pore pressure probe loaned by De Campos. As with the London clay tests, the differences between base pore pressure and probe measurements were used to assess rates of consolidation, and Figure 5.28 shows the correlation. It first appeared that rates above 25 kpa/day could not be supported in the later stages of the sequence. However, when the sample was allowed to rest at σ'_A = 400 kpa, it was noted that the axial strain rates diminished rapidly but the probe measurement remained almost constant. Calibrations were carried out after the test with a stone sample, in the way described by De Campos (1984). These investigations showed that for a fixed pore pressure, the probe was sensitive to σ'_r. The dashed lined in Figure 5.28 indicates the approximate development of the probe error for the given stress path, and shows the final excess pore pressures to be smaller than had been thought[1]. From these results the consolidation rates were taken as 40 kpa/day up to σ'_A = 200, 30 kpa/day to σ'_A = 300 and 25 kpa/day up to full cosdolidation. The rates of swelling were

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[1] After identifying the cross sensitivity discussions were held with the manufacturers which led to improvements in the deisgn.

chosen to reduce with the stress increment from the maximum condition in similar way. It was decided to allow the sample to drain at σ'_A = 400 kpa for 2 to 3 days before shearing or swelling back, and undrained tests were not started from any condition before the axial strain rate had reduced to 0.005% per hour.

5.11.7 The cross sensitivity of the Druck probe made it unsuitable for the verification of undrained shearing rates. Tests by Takahashi (1981) and De Campos (1984) showed that reliable base measurements could be made with Lower Cromer Till at comparatively fast rates of shearing, and it was estimated that a strain rate of 4.5% per day would give good equalisation. The single experiment with the Druck probe showed no tendency for the offset to increse by more than 2 kpa during undrained shear, and the later London clay tests also supported the choice of this particular rate.

5.11.8 As was explained in section 5.10, the stress path and strain rates chosen to represent K_o consolidation and swelling can be checked by monitoring the ratio ϵ_A/ϵ_V. The lag in pore pressure at the centre of the sample and the imperfect control of the stress path give rise to the trends shown in Figure 5.29. Slight bulging of the samples can be seen but the data again show that the selected path gave a smooth characteristic without any significant tendency for radial strains to develop.

5.11.9 The KoCU tests carried out using the above procedures took between 18 and 40 days to complete. Where 'perfect sampling' cycles or shear reversals were included, the experiments were extended by a typical period of 5 days.

5.12 TESTING PROCEDURES: TRIAXIAL TESTS ON INTACT MAGNUS SAMPLES

5.12.1 The testing procedures used for intact London Clay were not appropriate for the Magnus samples. The volume of intact material was limited, the cores could not be tested at their original diameter and the samples were generally stronger than the Canons Park material. The programme was divided into UU and anisotropicaly reconsolidated (KoCU) test series, but common methods of preparation and saturation were employed.

5.12.2 Trials with disturbed samples showed that trimming 38 mm diameter specimens from the cores could be difficult. Patterns of fissuring, shells, sand laminae, drop-stones and other obstacles were frequently encountered. In order to improve the probability of obtaining satisfactory test specimens, it was decided to X-ray the cores using the Faxitron machine located in the sedimentology laboratory of the geological department. The radiation was not able to penetrate the aluminium foil wrapping of the samples, and cores were examined with only the cling film protection in place. To obtain an even exposure the samples were packed into a long box filled with dry Ham river sand. Kodak Industrex C film was used and multiple exposures were made to obtain the best possible image. The results of the study were interesting, but gave little useful guidance for sampling trimming. In the end, cores were mostly cut up by trial and error and suprisingly few samples were unsuitable for testing. Trimming was carried out with a soil lathe, but the nature of the samples meant that a sharp hardened steel cutting blade was required to reduce the 76 mm cores to 38 mm x 76 mm cylinders.

5.12.3 Preliminary tests were also carried out in the conventional 38 mm triaxial apparatus using a similar procedure to that employed for the reconstituted soil. It was soon evident that the saturation techniques were unsatisfactory, as sample pore pressures invariably drifted downwards after each application of cell pressure and satisfacory B values were not obtained. A steel cell was prepared and a high pressure source connected to apply radial streses of up to 2,000 kpa. Even with these conditions the samples could not be saturated undrained and the initial effective stresses slowly increased by as much as 300 kpa. It was decided that back pressures could be required to saturate the soil, and that lower cell pressures should be used. The problem of saturating the intact samples was of special interest and is considered in some detail in section 5.14. The studies leave little doubt that the difficulties were caused by numerous internal gas-filled small voids. The application of back pressure allowed these spaces to be filled with deaired water, producing inert pockets in much the same way as gravel particles.

5.12.4 The procedure adopted was to mount the samples in the usual way and to rapidly apply a cell pressure of approximately 800 kpa

without drainage. The pore pressure transducer was monitored and a maximum value was usually observed after 1 to 2 hours. A back pressure equal to the maximum pore pressure was applied through a volume gauge, and both the burette and electrolevels were used to monitor the equalisation process over 24 to 48 hours. Once complete, the B value was checked and invariably found to be satisfactory. At this stage the sample was considered to be stabilised but unconsolidated and generally only very small volume changes (<0.2 cm^3) were required to obtain the stabilised condition.

5.12.5 Approximately half the intact samples were sheared in undrained compression from this stabilised state. These UU tests were carried out in the same way as the reconstituted series, with a strain rate of 4.5% per day.

5.12.6 The remaining samples were reconsolidated to their estimated insitu stress states. The vertical effective stresses were computed from the saturated density profiles given in the Fugro reports, and values of Ko derived using the procedures suggested in section 3.4. The results of the tests carried out by Gens (1982) point to a dependence of the behaviour of soil consolidated to a given condition on its previous stress path. It was therefore considered important to attempt to follow these rules:

(i) The direction of the final portions of the reconsolidation path should be similar to that estimated insitu. As most samples are at least slightly overconsolidated, the last stages usually involved swelling.

(ii) Reconsolidation should not involve any significant reductions is water content. It was not intended to take the sample up to, or beyond its preconsolidation pressures as recommended in the SHANSEP procedures. The intact series was intended to demonstrate features that might be destroyed by large volume changes.

(iii) If sampling had produced mean effective stresses which prohibited reconsolidation following rules (i) and (ii), isotropic swelling should be used to obtain suitable initial conditions.

5.12.7 Details of the individual reconsolidation paths are given in

Chapter 7 and the control systems were used to follow these paths in a smooth, continuous, fashion. The rates of axial stress change were gauged from the reconstituted pilot test, and varied between 40 and 20 kpa/day. At the end of the reconsolidation phase, the sample was allowed to rest until the axial strain rate fell below 0.005%/hour. Undrained shear was then applied in the usual way with a strain rate of 4.5% per day. The tests were continued to give axial strains of approximately 20%, before stripping down and taking water contents.

5.13 FURTHER NOTES ON LABORATORY PROCEDURE

5.13.1 No correction was made to the triaxial deviator stresses to account for the effects of membrane restraint in the interpretation of the two test programmes. Wesley (1975) carried out comparative tests between jacketed samples, and samples tested without membranes in a paraffin filled cell, and no difference was discernible. Costa Filho (1980) and Gens (1982) also carried out investigations of membrane restraint and similarly concluded that the effects were negligible for firm to stiff samples tested in compression. In extension, the membrane does influence the ultimate strength slightly, but no corrections were made to the reported data.

5.13.2 The recorded transducer voltages were printed onto paper records by the Solartron DTU system. These raw data were transferred to a Hewlett Packard 41-CV calculator, which retained programs to convert the voltages to engineering units and calculate stresses, pressures and strains for all the transducers. A small line printer produced summary record for each time increment from which stress-strain and stress-path plots could be constructed.

5.13.3 The descriptions of laboratory procedure given so far have concentrated on the performance of triaxial tests, although the programmes included index tests, gradings, oedometer tests, resonant column experiments and ring shear tests. The routine experiments were all performed in accordance with BS 1377 or the recommendations of Akroyd (1964). The other special tests were not carried out by the author and the essential references to procedures will be given in the appropriate sections.

5.14 ANALYSIS OF SATURATION PROCESSES WITH INTACT MAGNUS SAMPLES

5.14.1 The difficulties of saturating intact samples of Magnus clay in an undrained condition were reported in the preceding section. The following paragraphs summarize an analysis of the observed phenomena.

5.14.2 The most simple explanation for the pore pressure behaviour was the hypothesis that the sample contained a number of discrete gas filled cavities. As the pore pressures build up around such voids internal seepage would occur and the pore pressure would show a downward drift for undrained conditions. Hight (1983) had remarked that many North Sea clays were sedimented with a high organic content, and gas pockets could develop within a soft deposit. On consolidation arching would have taken place around the voids and many would survive the slow transformation from slurry into a hard clay. At the time of sampling, these cavities might be expected to be full of gas diffused from the surrounding soil, and the internal pressure would approximately equal the ambient pore water pressure.

5.14.3 The Magnus samples were taken in 186 metres of water and from depths below the sea bed of up to to 200 metres. Extracting material from these conditions involves a considerable release of total stress, and these changes would cause the samples to slightly expand and to develop large suctions in the soil matrix. However, the enclosed gas pressure would only fall significantly if the internal voids were able to expand without restriction. For a spherical cavity this can occur if the pressure difference exceeds the limit pressure:

$$Pc = \frac{4}{3} Cu \left[1 + \log_e \frac{Eu}{3Cu}\right] \simeq 10\ Cu \qquad \text{Eq. 5.3}$$

Considering a sample taken from 100 metres below the sea bed, the internal pressure would approximately equal 2,800 kpa. Thus if the shear strength was less than 280 kpa expansion could occur, and would continue until the internal pressure reduced to 10Cu.

5.14.4 Inspection of the Cu profiles obtained by Fugro indicated that the gas pockets would be near to the limit pressures over a wide range of depth. Furthermore, if cavity expansion did not occur, the high

gas pressure gradients would probably cause outward diffusion of the gas over the storage period. In summary, when the sample was placed in the triaxial apparatus the void spaces could be expected to be enlarged and to contain only low pressure gas. At the same time, gas dissolved within the sample's pore water could be coming out of solution. Any free vapour would migrate towards the cavities and fissures, and might be able to find a path to the outside of the sample.

5.14.5 The application of the cell pressures would cause only small contractions of the void spaces, unless an increment greater than 10Cu was to be rapidly applied so as to collapse the cavities. If similar holes had existed in the intact London clay, or the reconstituted soil samples, the applied initial cell pressures would have been sufficiently large to eliminate the voids by internal collapse. With the stronger Magnus clays this was not possible, and each increment of σ_3 would have been resisted by arching around the cavities, with the local shear strengths increasing gradually with time. After the first application, the collapse pressure would thus increase from 10 Cu to some higher value.

5.14.6 The extent of undrained pore pressure drift would depend on the stiffness of the soil and the density of the voids. It is simple to show that for undrained conditions:

$$\Delta \sigma' = K' V_{cav}/V_{total} \qquad \text{Eq. 5.3}$$

(where K') is the effective bulk modulus)

The ratio K'/Cu can be estimated from the results of Gens (1982), and for lightly overconsolidated soil at small strains a value of 1,000 might be expected. Thus for Cu = 300 kpa, $\Delta \sigma' = 3 \times 10^5$ Vcav/Vtotal. If the cavities were to occupy 0.1% of the total volume, pore pressure drifts of 300 kpa would be expected.

5.14.7 The time taken to establish equilibrium after each cell pressure increment is more difficult to assess. If the problem is idealised as flow into a single spherical evacuated cavity within an incompressible infinite medium, it can be shown that the rate of flow is:

$$q = H\,4\pi r\,k \qquad \text{Eq. 5.4}$$

(where H is the head difference between the cavity and the boundary of the infinite medium, r is the cavity radius and k is the permeability). From this equation the cavity will be filled when:

$$t_{100} = r_o^2 / H3k \qquad \text{Eq. 5.5}$$

considering a 10 kpa pressure change, and taking $k = 10^{-12}$ m/s from the Fugro oedometer tests, t_{100} is calculated as 0.9 hrs for $r_o = 0.1$ mm, and 8 hours for $r_o = 0.3$ mm.

5.14.8 The problem can be further complicated by considering the void to be filled with gas which is compresed by the incoming flow from its initial volume Vo, to produce an internal back pressure. Assuming Boyle's Law, the previous expressions can be rederived in terms of the altered gas pressure Pg, and volume V;

$$t = \frac{V/Vo}{F.P_{ext.}} + \frac{Pg^o}{F.(P_{ext.})^2} \log_e \left[\frac{P_{ext.} + Pg - P_{ext.}\, V/Vo}{P_{ext.} + Pg^o} \right] \qquad \text{Eq. 5.6}$$

$$\text{where} \quad F = \frac{3k}{\gamma_w r_o^2} \qquad (5.7)$$

Full equilibration occurs when the internal gas pressure and the external absolute pore pressures, P_{ext}, equalise; ie when $V/Vo = Pg^o/P_{ext.}$ Evaluating the expression shows that the 'back pressure' effect does not have a major influence. The effects of local consolidation have been estimated using solutions given by Gibson (1963) and these also do not greatly effect t_{100}.

5.14.9 To be of more quantitative use, the analysis should be extended from the modelling of a single cavity to consider an array of bubbles within a finite volume of soil. This has not yet been attempted. Nevertheless, the simple theory gives reasonable predictions for the observed resistance of the hard Magnus samples to undrained saturation. To test the central hyphothesis that holes exist within the soil mass, thin petrological sections of one of the pilot test samples were prepared using the Carbowax technique of Tchalenko (1967). Under polarised light any holes in the sample would appear as back shapes which would not change in shade as the polarising angle is rotated. Figure 5.30 shows two photographs of a

typical section with a 45 degree rotation between the exposures. Numerous small holes can in fact be seen at low magnifications, and 20 cavities of diameter between 0.2 and 0.7 mm were counted in a slice 35 mm by 37 mm. At higher magnification, around 80 holes with diameter between 0.02 and 0.2 mm could be observed per square centimetre. From these densities the ratio of cavity volume to total volume was calculated as approximately 0.15%, and substitution in Eq 5.3 predicts the observed total drifts in pore water pressure with surprising accuracy. Regarding the rate of change of porewater pressure with time, the simplified treatments show that the equilbration of each undrained application of cell pressure could require a period of hours or days to become complete.

CHAPTER 6

INVESTIGATIONS OF THE SOIL CONDITIONS AT THE MAGNUS SITE

6.1 INTRODUCTION

6.1.1 It is the purpose of this chapter to summarize the investigations carried out at the Magnus site and to report the laboratory testing programme undertaken at Imperial College. The work divides into five main parts; (i) a description of the general conditions identified from the commercial site investigations contracts, (ii) a report of the I.C. programme of tests on reconstituted soil, (iii) an interpretation and discussion of these results and (iv) a discussion of the I.C. tests on intact samples. Part (iv) gives details of some special experiments, and the chapter concludes with a summary of the investigations on the Magnus soils.

6.2 PART 1, GENERAL CONDITIONS

6.2.1 Rigden and Semple (1983) provide a summary and interpretation of the commercial site investigation work at the Magnus site. They note that six geotechnical and three geophysical surveys were carried out between 1975 and 1980, to which the field work by Fugro was added in 1981. During this period, offshore practice improved considerably and the investigations included the first use of several of the innovative techniques discussed in Section 2.5, including sea floor hydraulic drilling manipulators, large-scale vibrocoring, seafloor plate loading tests, the downhole push in pressuremeter (PIP) and offshore effective stress testing.

6.2.2 The surveys showed the foundation conditions to be uniform and consistent with existing knowledge of North Sea shallow geology. Beneath a surface veneer of sand, the uppermost layer in the geotechnical profile is a massive glacial till. The layer consists of generally hard dark grey silty clay with gravel, sand and shell fragments, and extends to around 13 to 14 metres depth. This stratum is softer near the sea bed and contains occasional boulders, up to 1 metre in diameter. The layer was thought by Rigden and Semple to

represent the last glacial advance 10,000 to 25,000 before present.

6.2.3 The massive till sequence rests on a layer of sand which is approximately 3 metres thick, and displays a coarse consistency with gravel, shell fragments and some clayey intrusions. In borehole 11 a second sand statum of similar thickness and nature was logged between 15 and 19 metres, under a dividing layer of hard clay till. The sand layers were assumed by Rigden and Semple to have been deposited on an erosion surface which represented the top of a series of older marine or glacio-marine clays. These clay layers were considered to extend to the full depth of interest and were thought to have been deposited during an interglacial period 25,000 to 50,000 years before present. A reflector was noted in the multi-electrode sparker records at around 50 metres depth. This appeared to separate the clays into an upper sequence, which was often laminated and contained numerous inclusions of silt and sand, and a lower series which was thought to display a more foliated and occasionally platey structure.

6.2.4 The positions of the three boreholes discussed in detail by Rigden and Semple (1983) are shown in Figure 6.1. Borehole 11 was formed by Fugro in 1981 was the nearest to the instrumented pile group of Leg A4, and provided most of the samples that were tested at Imperial College. At boreholes 1 and 11, sampling was carried out by controlled hydraulic pushing, whilst BH 4 employed a percussive method. The latter technique is generally thought to produce greater disturbance.

6.2.5 Figure 6.2 shows the interpreted shear strength profiles from the three boreholes. The plots are based on unconsolidated triaxial tests on 72 x 154 mm samples for boreholes 1 and 4, and 38 x 76 mm samples for borehole 11; Rigden and Semple argue that shear strength should be expected to reduce with increasing sample size. As the boreholes showed similar distributions of index properties, attention will be focussed on borehole 11 for a more detailed discussion of the variations in soil properties with depth. A series of push in pressuremeter tests were also carried out in BH 11, and the results are summarized in Appendix 3.

6.3 GEOTECHNICAL PROFILE FOR BOREHOLE 11

6.3.1 An interpreted geotechnical profile for borehole 11 has been assessed from three main sources; (i) the data in the Fugro report, (ii) simple tests and sample inspection at Imperial College, and (iii) the information in the previous site investigation reports.

6.3.2 The profile is summarized in Figure 6.3 and the fabric types have been identified following the recommendations of Hight (1983, 1985). Photographs taken by McClelland Engineers are presented in Figure 6.4 to illustrate the main changes in soil fabric and composition. The X-ray images taken at I.C. are shown in Figure 6.5 to supplement this overall impression. The high density of shell and gravel fragments in the near surface sample is clear, as are the sub-horizontal bedding and frequent erratics found in the deeper layers. The oblique fissure patterns in the sample taken from 63 metres below sea bed are interesting, and faint traces of the same macrofabric can be seen in the sample from 68 metres depth.

6.3.3 The interpretation of the soil layering given in Figure 6.3 shows rapid changes of soil type over the uppermost 20 metres. Only a thin band of structureless clay was found that could have been deposited as a lodgement till, and the remaining clay and sand layers show strong evidence of sedimentary deposition. Below 20 metres, the dominant soil fabric is the pseudo-platey 'fishscale' pattern that is typical of glaciomarine deposits. Subtle changes with depth point to variations in the rates of deposition, the energy of the sedimentary environment and the effects of freeze-thaw cycles. The tendency towards a horizontal bedding structure, the contortions of clay around erratics and the changes in strength point to large vertical consolidation pressures in the layers below 70 metres. The density of samples was insufficient to log the borehole below 100 metres, but cores from 200 metres depth consisted of hard laminated clays of higher plasticity; these frequently included slickensided shear surfaces.

6.3.4 The variations in soil composition over the first 100 metres depth are shown in the grading plots presented in Figure 6.6 and the

summary profiles of Figure 6.7. There is a clear change at 30 metres, and below this level the gravel fraction is negligible. The percentage of sand remains constant at 20% until a depth of 80 metres is reached, where the proportion starts to slowly increase. The grading curve found for the reconstituted soil mix is shown for comparison on Figure 6.6, and there is good agreement with the distributions proved for the soils taken from below 30 metres. Figure 6.8 a) gives the scatter of water contents measured by Fugro, which were rounded to the nearest whole percent[1]. Figure 6.8 b) plots bulk density profile and the trend line of the water content data is compared on Figure 6.8 c) with the measurements made in the intact testing programme conducted at Imperial College. The results of careful Atterberg limit tests are indicated to show that most of the soils have been consolidated to water contents near their plastic limits. The water contents measured at Imperial College also tend to fall below the Fugro trend line by about 0.5%. This may be explained by differences in oven temperature, rounding errors or drying out of the samples during storage. Figure 6.9 plots the plasticity indices from the Fugro report and compares these with the Imperial College results. Both sets of data show PI increasing with depth from around 19% in the upper layers to approximately 24% at 90 metres depth. The Imperial College tests show a smaller scatter and consistently give indices approximately 2% less than the Fugro results. Atterberg limits taken from the reconstituted soil mixes indicate slightly lower values; the mean results of four tests carried out between February 1982 and June 1983 gave LL 35% PL 17.2% and PI 17.8%. These changes might be related to the sieving of the powdered soil or the reduction of organic material during drying.

6.3.5 Oedometer experiments provide valuable information concerning soil properties and can give indications of stress history. Eleven one dimensional consolidation tests were carried out by Fugro and the voids-ratio, log σ'_v plots are shown in Figure 6.10. The curves naturally divide into three distinct groups, 0 to 28 metres, 35 to 80 metres and below 90 metres. The virgin compression lines for the

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1 Unfortunately such rounding is the recommended commercial practice.

soils were constructed by assuming these to be coincident with the final section of the measured curves, where σ'_r increased from 1,000 to 3,200 Kpa. The slope and intercepts of the lines are plotted against depth in Figure 6.11. These results indicate lower values for VCL gradient and initial voids ratio in the top 30 metres, and a tendency for both parameters to reduce from their maximum values below 70 metres depth. The data from a single check test carried out at Imperial College gave good agreement with the plotted trends.

6.3.6 The Fugro unconsolidated triaxial shear strength data are plotted in detail on Figure 6.12 (a). The author's interpreted mean line is shown and this was used to obtain the profile of apparent preconsolidation pressure given on 6.12 (b). The relationship between Cu/σ'_r and ORC found by Gens (1982) was assumed for the calculations. The graph of insitu vertical effective stress, σ'_v, was constructed using the variation of bulk density with depth shown in Figure 6.8 b). The changes in γ_{bulk} are consistent with the profiles of shear strength and plasticity index.

6.3.7 The preconsolidation profile sketched in Figure 6.12 b) shows the upper layers to be in an apparently heavily overconsolidated state. Between 40 and 84 metres the strata seem to be only lightly overconsolidated, and a transition zone extends from 20 to 40 metres. The plot also implies a slight increase in overconsolidation ratio for soils below 70 metres depth.

6.3.8 From the summarized interpretation of the site investigation data, it was decided to divide the sequence into five main geotechnical units, which are denoted in the profiles shown from Figure 6.3 onwards as groups I to V. The thin sand layers are excluded from these groups, as they extend over less than 6% of the depth of interest, and little is known concerning their properties.

6.4 PART 2, TEST PROGRAMME ON RECONSTITUTED SOIL : OEDOMETER TESTS

6.4.1 In addition to the index tests already described, the re-constituted experiments comprised four principal elements - oedometer ,

triaxial, ring shear and resonant column tests. Although the triaxial programme was by far the most extensive, it is convenient to first describe the behaviour seen in the oedometer experiments; these include two tests on reconstituted clay, MROD1 and MROD2, and an intact test, MIOD1. Sample MROD1 was compressed from slurry, MROD2 was trimmed from a clay cake which had been previously consolidated from slurryand MIOD1 was trimmed from intact material sampled at 67.2 metres depth. The tests were carried out in general accordance with routine practice, but care was taken to calibrate the test system compliance using a steel dummy sample, and the procedures advocated by Adresen et al. (1979) for the identification of the preconsolidation pressures were adopted for MROD2 and MIOD1.

6.4.2 The three oedometer voids-ratio log σ'_v plots are shown on Figure 6.13, which also gives the curve for reconstituted Magnus soil during the K_o consolidation stage of a typical stress path test. The compression curves of the reconstituted material are consistent, and show the slope of the virgin consolidation line Cc, to gently reduce with increasing stress in the way described by Butterfield (1979). The stress range of interest is $50 > \sigma'_v > 1{,}000$ Kpa, and over this portion the VCL could be described as;

$$e = 1.33 - 0.285 \log_{10} \sigma'_v \qquad \text{Eq. 6.1}$$

At the higher stress levels considered in Figure 6.11 the VCL is better expressed by;

$$e = 1.22 - 0.210 \log_{10} \sigma'_v \qquad \text{Eq. 6.2}$$

The curve for MIOD1 gives a similar slope to the reconstituted soil, but indicates an initial voids-ratio which is approximately 0.055 higher. This shift corresponds to a disparity in moisture content of around 2%, which was also indicated by the trend of the Atterberg limits.

6.4.3 The compressibility and time dependent behaviour of the reconstituted material is summarized in Figure 6.14 a) to e). The first two plots show how normally consolidated compressibility, m_v, falls steeply with σ'_v, and the initial swelling stages show very stiff behaviour. With increasing overconsolidation m_s becomes larger, and can exceed the normally consolidated compressibility for OCR 50. Figure 6.14 d) indicates that the permeability is also sensitive to σ'_v and that k reduces by two orders of magnitude when the clay is consolidated from slurry to σ'_v = 8,000 Kpa; furthermore permeability shows little recovery through swelling. These compressibility and permeability characteristics combine to make C_v and C_{vs} dependent on σ'_v and OCR in the way shown by Figure 6.14 c). For normally consolidated conditions, C_v slowly increases from around 0.1 m^2/yr at low stress to 1.05 m^2/yr at σ'_v = 8,000 Kpa. Unloading gives high initial values of C_{vs}, but these reduce with σ'_v and become very small at high OCR. The coefficients of secondary consolidation plotted in Figure 6.14 e) are similarly sensitive to overconsolidation ratio. For normally consolidated conditions C_α is almost constant at 3×10^{-3}, but the initial portions of swelling or recompression stages give much lower coefficients. The slopes of the e - logσ'_v swelling lines are also far from linear, and Figure 6.15 illustrates this feature using some of the Fugro tests and the three experiments carried out at Imperial College.

6.4.4 The results of the intact test MIOD1 are summarized in Figure 6.16; the behaviour generally fits into the pattern given by the reconstituted material.

6.4.5 In principle, it should be possible to recover the pre-consolidation pressures from the e - logσ'_v plots. The Casagrande construction was applied to the data for MROD2 and MIOD1, and respective σ'_{vc} values of 225 and 1,050 Kpa were obtained. As the first test had been consolidated to a fixed pressure of 200 Kpa, the method gave a 12% overprediction. With the intact sample, the difference between the constructed σ'_{vc} and that evaluated on Figure 6.12 b) was 22%. Greater difficulties were found in interpreting the Fugro tests in this manner; swelling during setting up and the large

stress increments between the stages made the Casagrande construction somewhat arbitrary.

6.5 TRIAXIAL TESTS ON RECONSTITUTED SOIL : SERIES R

6.5.1 The triaxial programme with reconstituted Magnus clay comprised a total of 31 experiments, and in 29 tests local measurements were made of the axial strain response during shearing. Table 6.1 divides the experiments into seven groups, according to the pre-test conditions and the type of loading. The two unlisted tests MR4' and MRU(i)' did not use the electrolevel gauges, but were otherwise identical to MR4 and MRU(i). The results of each series will be discussed in the sequence given in Table 6.1, and a summary of the more important parameters is given in Table 6.2.

6.5.2 The behaviour of K_o consolidated Magnus clay in undrained compression is first illustrated by reference to the stress paths shown in Figure 6.17 a), together with the deduced contours of developed axial strain (the strains shown are the means from diametrically opposite pairs of electrolevels). From this figure two important observations can be made :

(i) The effective stress paths followed by the tests were initially both straight and nearly vertical, although there was a tendency for the direction to rotate anticlockwise with increasing OCR. In each case there was a certain stress level where the paths sharply deviated and then travelled on to failure. The latter portions of the effective stress paths were taken as representing the post yield portions. In each case yield was approached after the development of only very small strains; sample MRI reached peak deviator at an axial strain of 0.1% and the remaining tests all demonstrated sharp changes in stress path direction at axial strains less than 0.2%.

(ii) The stress path for the normally consolidated sample, MR1, showed brittle behaviour with a marked reduction in strength after 0.1% strain; tests carried out by Gens (1982) on Lower Cromer Till showed similar behaviour. Small loops are apparent in the stress paths for samples MR1.4 and MR2 close to failure. If, instead of measuring pore pressure at the base, a central piezometer probe had been used it is probable that these loops would not have been observed, Hight (1983), (1985).

6.5.3 The effect of ageing under normally consolidated conditions is demonstrated in Figure 6.17 b). Here the stress path for MR1 is contrasted with that of a sample which had been allowed a 20 day pause for secondary consolidation before shearing. Although ageing extended the yielding surface to allow higher strengths and stiffness to be developed, the general pattern of behaviour was little effected.

6.5.4 The undrained brittleness displayed in the series R tests is summarized in Figure 6.18 where the ratio $C_u peak/C_u ult$ is plotted against OCR. Ultimate strength is defined here as that developed at strains greater than 20%, and as MR1 and MR8 exhausted their travel before reaching 12% strain, both are omitted from the plot. Brittleness clearly reduces with OCR, but even samples on the 'dry' side showed strength reductions after obtaining their peak deviator conditions.

6.5.5 The stress paths from series R all tended towards a critical state line defined by $\phi' \simeq 30°$. The peak and ultimate strengths are related to water content in Figure 6.19, where the K_o virgin consolidation lines determined from the oedometer tests are also shown. The best fit line drawn through the ultimate strengths is virtually parallel to the oedometer lines, and the ω/c-log p'_f relationship can be drawn in by noting that $\phi' = 30°$. However, the peak strengths do not fit the consistent pattern found for the ultimate conditions. The maximum deviator stresses were developed at ϕ' values considerably below 30° for OCR's less than 2, and the ω/c-log Cu relationship is curved and not parallel to the one dimensional VCL.

6.5.6 Figures 6.20 a) and b) show the stress-strain characteristics of the reconstituted soil after K_o consolidation. Again it can be seen that the strains over the initial range of stresses are exceedingly small. In order to allow an analysis of the initial stiff zone, the strains have been replotted to a logarithmic scale in Figure 6.21. The latter Figure shows a remarkably consistent trend, with the strain required to achieve peak strength steadily increasing with OCR. The scatter in the early stages of test MR2 was caused by vibrations from a nearby motor and demonstrates that the new electrolevel gauges perform best in a still environment.

6.5.7 In Figure 6.22 the stiffness characteristics of the samples are examined by plotting the secant modulus E_u up to and including peak deviator using the same strain axes. It should be noted that the use of the secant modulus E_u is not meant to imply that the soil behaviour is strictly elastic, and has merely been taken as a convenient measure of soil stiffness. Each of the curves shows high initial moduli which rapidly reduce with strain, but it is difficult to separate the magnitude of a sample's stiffness from its degree of non-linearity.

6.5.8 For ease of comparison and presentation, the initial undrained stress-strain characteristics may be represented by the following parameters relating to stiffness and linearity;

(i) "Stiffness" is given by the undrained secant modulus at 0.01% axial strain $E_{u(0.01)}$. It may be expressed non-dimensionally as $(E_u/C_u)_{0.01}$, $(E_u/p'_o)_{0.01}$ etc.

(ii) The linearity parameter is defined as $L(\varepsilon) = E_u(\varepsilon)/E_{u(0.01)}$. $E_u(\varepsilon)$ is the secant modulus at strain ε, and the simple index L is defined as the value of $L(\varepsilon)$ when $\varepsilon = 0.1\%$. Straight line behaviour gives $L = 1.0$, and if the modulus decreases with strain $L < 1.0$.

6.5.9 Values of $E_{u(o.o1)}$ and L for series R are given in Table 6.2, and in general L increased with overconsolidation ratio. In particular, test MR1 showed the smallest L value of 0.185. A more complete description of the dependence of non-linearity on OCR is given in Figure 6.23 which shows the variations of $L(\varepsilon)$ with strain for series R. The data show a consistent trend, although sample MR21 reverses the pattern at high OCR. The $L(\varepsilon)$ plot for OCR = 2.0 gives an approximately median line through the family of curves.

6.5.10 Figure 6.24 shows how the normalised scales of stiffness vary with OCR. The ratios of $E_{u(o.o1)}$ to σ'_v, p_o and Cu are all far higher than those usually inferred from laboratory tests, and this demonstrates both the importance of internal instrumentation and the need to define modulus carefully. The stiffness ratios are usually specified for fixed proportions of the shear stress increment, $\Delta\tau_f$, required to produce failure. Reference to Figure 6.17 a) shows the strains at 0.5 $\Delta\tau_f$ for tests MR1 and MR22 to be around 0.01% and 2.0% respectively, and thus a conventional interpretation would compare the stiffest behaviour at OCR 1.0 with a post yield modulus for the overconsolidated clay. Finally, the plots of Figures 6.24 and 6.22 show that lightly overconsolidated conditions give the stiffest behaviour in both absolute and normalised terms.

6.6 TRIAXIAL TESTS ON RECONSTITUTED SOIL : SERIES RE

6.6.1 The group of extension tests carried out on K_o consolidated soil will be reported in a similar way to series R. None of the tests were taken to large strains as necking and membrane restraint make the measurements beyond 5% strain ambiguous; it was because of these difficulties that Bjerrum (1973) invariably quoted extension strengths at shear strains of 2%. Nevertheless, it is possible that the deduced deviator stresses at 5% axial strain are less than the maxima that the soil could sustain. The results of Gens' (1982) careful experiments indicate that strain hardening in extension may allow samples to develop a further 10 to 15% in strength after the adopted limiting condition.

6.6.2 Figure 6.25 gives the stress paths and strain contours for the RE series, excluding MRE4d and MRE1s. In comparison with the plots for series R in Figure 6.17, three points are noteworthy;

(i) The initial portions of the undrained stress paths are almost straight and were gently inclined to the right, except for test RE1. In each instance the path changed direction when strain levels of 0.1 to 0.3% had been obtained. After an apparent yield, the paths continued in a similar direction until the $\phi' = 30°$ line was reached. At this stage the samples dilated and started to climb the failure line.

(ii) None of the tests indicated brittle behaviour, and the $\phi' = 30°$ line was only intercepted after the attainment of axial strains between 2 and 5%.

(iii) Although the RE series shows a closer spacing of strain contours near to the swelling line, there remains a considerable region of stress pace in which the strains are small, and the soil behaviour stiff.

6.6.3 The undrained strengths in extension are far smaller than those developed from the same initial conditions in compression tests; the values listed in Table 6.2 show reductions of almost 50% for the full range of OCR's. Even allowing for a hardening of 15% with strain, the data indicate a marked anisotropy in peak Cu, and probable shortfalls of 15% to 30% in the critical state strengths. Most of the RE series tests were carried out on the second block of reconstituted soil, for which the Atterberg limits differed from the first batch by 1 to 2%. Although these slight changes would produce little effect on the stress-strain curves or strengths, they produce major difficulties in comparing the ω-c $\log\sigma'_v$ relationships found for the two series.

6.6.4 Figure 6.26 summarizes the undrained stress-strain response in extension. A consistent family of smooth curves is obtained with the semi-logarithmic plot, which shows a tendency for stiffness to

reduce with strain and OCR. Comparing the curves of MRE1 and MRE1s it can be seen that undrained stiffness also depends on the rate of straining. Tests MRE1s and MRE2s comprised the unloading stages of the perfect sampling series RS, and travelled no further than the $\sigma'_a = \sigma'_r$ line; both were carried out slowly under stress control.

6.6.5 The variations of stiffness with strain are indicated in Figure 6.27 where $L(\varepsilon)$ is plotted against strain. The graphs are similar to those for series R, but in extension the degree of non-linearity tends to decrease with OCR. The most linear behaviour is shown with $L = 0.43$ for MRE1, and the least by MRE17 for which L was 0.32.

6.6.6 Table 6.2 and Figure 6.28 summarize the parameters describing the scale of stiffness. In extension, the highest initial Young's Modulus is found with normally consolidated soil. Although the failure strain is at least 50 times that for the compression test, the modulus at 0.01% strain is 42% greater. The plots of $(E_u/p'_o)_{0.01}$ and $(E_u/\sigma'_v)_{0.01}$ show gentle variations; the former rises slowly but the latter tends to fall with OCR. In considering the plot for E_u/C_u a difficulty arises as to the choice of normalising shear strength. For consistency, the peak value from series R has been used but the expression is clearly the least easily defined of the stiffness ratios. The sketched curves correspond to the values expected for tests at the nominally standard rate, and pass above the points indicated (s), where much slower rates of shearing were imposed. The tabulated results suggest that the scale of stiffness is more affected by strain rate than is the $L(\varepsilon)$ function.

6.6.7 The test MRE4d was not planned, but was the result of a motor malfunction. Instead of travelling along the K_o swelling line, the test followed the drained stress path shown in Figure 6.29. The path was inclined slightly to the right of a drained extension path with constant σ'_r. Although accidental, the results are still of interest, and are summarized in Figure 6.29 and the stress-strain curve given on Figure 6.30. The observed ratios of volumetric to axial strain were small, as the path followed a similar direction to the undrained test

MRE4. The stress strain data were analysed to find the variations of the drained modulus E', with strain. The results are given in Table 6.2 and show a similar value of L to the undrained test, MRE4, and a ratio E'/E_u of 0.52 at 0.01% strain. However, the rate of stress change for MRE4d was very slow and, recalling the comparison between tests MRE1 and MRE1s, a larger ratio of E'/E_u between 0.70 and 0.80 might be expected for tests performed at comparable rates of strain.

6.7 TRIAXIAL TESTS ON RECONSTITUTED SOIL : SERIES RS AND RR

6.7.1 Series RR and RS were conducted to assess the effects on K_o consolidated clay of severe undrained disturbance. The RR tests show the effects of unloading to the hydrostatic axis for OCR's 1 and 2. The initial conditions for the series are indicated on Figure 6.31 and reference to Figure 6.25 shows that such perfect sampling involves relatively small strains, but leads to a marked change in stress path direction on reloading. The RR tests consider clay taken to failure in extension, and the stress paths shown on Figures 6.25 and 6.33 show that tensile axial strains of around 5% were developed before reaching the initial conditions of the recompression tests. In both series the soil was allowed to attain a stable condition with negligible creep rates before the recompression phase was commenced.

6.7.2 The stress paths and strain contours for the RS series are shown in Figure 6.31. The data from MR4 is also shown, as this can be considered as a 'perfectly sampled' test for OCR4. The path found from test MR1 is replotted on the same figure and provides an upper bounding surface for the RS experiments. The three tests MRS1, MRS2 and MR4 show similar initial behaviour; the stress paths are almost vertical up to 0.1% strain and then turn to the right. For MRS1 and MRS2 a second yield occurs at about 1% strain when the paths intercept the stress path for MR1. At this point the paths turn almost full circle, follow MR1 towards the $\phi = 30°$ line, and finally reach ultimate strengths similar to those found in the series R tests. The path for MR4 climbs the State Boundary Surface from the dry side until peak deviator is obtained at around 3% strain, after which it turns back and retraces part of the $\phi' = 30°$ line. 6.3

6.7.3 The undrained brittleness observed in the RS tests is summarized on Figure 6.18 where the ratio of peak to ultimate strength plots slightly below the K_o consolidated compression tests. Figure 6.32 shows the stress-log-strain curves for MRS1 and MRS2, and Table 6.2 lists the values of L and $E_{u(0.01)}$.

6.7.4 Figure 6.33 gives the stress paths for the three RR tests, and shows how pre-extension locates the starting points in the lower left quadrant of triaxial stress space. The initial paths for MRR1, MRR4 and MRR17 are similar and incline steeply to the left, and the first two samples continue to rapidly generate pore pressure until dilation starts at strains between 1 and 2%. Unfortunately, a data logging malfunction developed shortly after the start of test MRR17, and information was only recorded up to 0.6% strain. Strain contours are sketched and show the recompression paths to trace out a region, between the $\phi' = 30°$ line and the hydrostatic axis, where the strains are small and the soil response stiff. The paths for MRR1 and MRR4 climb a Hvorslev type surface until peak strengths are achieved at strains of 12 to 13%; these strengths are approximately equal to the ultimate values determined in the series R tests, and show only small reductions with post peak straining. The effect of pre-shearing the samples in extension is to produce characteristics similar to those of the heavily-over-consolidated specimen MR21.

6.7.5 Figure 6.34 shows the family of stress-strain curves for the RR series, and Table 6.2 summarizes the results. The tabulated values of L show that normally consolidated clay gives the least linear response on unloading from failure in extension.

6.7.6 The effects of perfect sampling on the scale of stiffness can be discussed by reference to Table 6.2 and Figure 6.35. For soil sampled at OCR1 the recompression stage shows a stiffer response than the comparable test from K_o conditions, but indicates softer behaviour than the extension test MRE1. AT OCR2 the pattern is reversed, but the normalised stiffness ratios are broadly comparable to those plotted on Figure 6.24 for the K_o compression series R.

6.7.7 Table 6.2 and Figure 6.36 show an altered pattern for samples recompressed from failure in extension. With normally consolidated soil the response is softer than for both extension and compression from K_o conditions. At OCR4 recompression gives moduli between the K_o compression and extension values. Finally, the recompression initial stiffnesses at OCR 17 exceed both the K_o compression and extension moduli. The ratios plotted in Figure 6.36 reflect these changes in behaviour, but show that the normalised stiffnesses do not depart from the series R results by more than 50%.

6.8 TRIAXIAL TESTS ON RECONSTITUTED SOIL : SERIES RI

6.8.1 Series RI was performed by isotropically consolidating samples cut from the blocks prepared in the 229 mm oedometer. The tests model insitu material which has been disturbed undrained, and then reconsolidated to an unfamiliar stress condition. This might represent the consolidation stage of a commercial "effective stress" test, or a particular point insitu at which an isotropic state of stress has been produced by a process such as pile installation.

6.8.2 The three samples MRI1, MRI2 and MRI4 were consolidated in stages to an isotropic stress of 400 Kpa before swelling back, or shearing at OCR1. Undrained compression tests were carried out, and the stress paths are shown in Figure 6.37 with strain contours indicated at suitable intervals. The divergence from the K_o consolidated pattern is most striking with the normally consolidated sample. MRI1 shows an initially linear stress path which inclines to the right but curves gently to the left after attaining a strain of 0.02%. The path slowly climbs to a peak deviator condition, which develops at an axial strain of 7% with ϕ' approximately equal to 30°. After reaching this point, the sample follows a strain softening path close to the $\phi' = 30°$ line. The two overconsolidated samples gave stress paths which were linear and inclined to the left up to strains of 0.1%. MRI2 then curved further to the left until a yield developed between 1 and 2% strain. From this point the stress path turned 270 degrees and followed a bounding surface which was of similar shape to

the undrained stress path of MRI1. After an apparent yield at 0.1%, MRI4 curved to the left and climbed above the $\phi' = 30°$ line until the travel of the apparatus was exhausted at around 9% strain.

6.8.3 The undrained brittleness found in the RI tests is compared in Figure 6.18 with the behaviour of the other series, and the isotropic tests show the least tendency to soften post-peak. Despite the more ductile behaviour of the normally consolidated soil, the strain contours point to an extensive zone, centred on the isotropic axis, in which the strains are small and the response to shear is stiff.

6.8.4 The stress strain curves for the tests are shown in Figure 6.38, and reference to Table 6.2 shows MRI1 and MRI2 to have more linear stress strain characteristics than equivalent K_o consolidated samples, although MRI4 indicated a rather low L value. The Young's Moduli found at 0.01% strain in the three tests are comparable with those found in the R series at the same overconsolidation ratios; MRI1 is slightly stiffer, MRI2 is significantly softer but there is only a 3% difference between MRI4 and the OCR4 test from series R. The relationships between normalised stiffness and OCR are summarized in Figure 6.39.

6.9 TRIAXIAL TESTS ON RECONSTITUTED SOIL : SERIES RU AND RD

6.9.1 The purpose of the two series RU and RD was to obtain preliminary information concerning the effects of rate, drainage and end fixity on the stress strain behaviour of the reconstituted soil. Time limitations ruled out a lengthy programme with K_o consolidated samples, but a limited set of experiments on specimens cut from the oedometer blocks was feasible. The vagaries of sampling and setting up interfered with the goal of testing identical samples, but the initial effective stresses were controlled and each of the specimens was allowed a two day rest period in the triaxial apparatus before shearing was commenced. The rates of testing are best assessed by the times required to develop 0.1% strain, and these are listed in Table 6.2 for each experiment.

6.9.2 The stress paths of four un-consolidated undrained tests are presented in Figure 6.40 a) and b). The fast test MRU (iii) is omitted as base pore pressures measurements cannot be considered reliable at this rate, but the path of MRU (i) is included[1]. All four tests give similar stress paths and 6.40 a) shows lubricated ends to have only a slight effect for experiments conducted at the standard rate. Figure 6.40 b) further suggests that rate changes do not drastically alter the response to undrained shear. The apparently greater strength shown by MRU (iv) is not thought to be representative of the soil behaviour but to have resulted from a contact made between an electrolevel gauge and the base pedestal at a strain of around 1.0 percent.

6.9.3 The purpose of the RU test programme was to examine undrained stress strain behaviour, and the data are compared on semi-logarithmic axes on Figure 6.41 a) and b). The four curves are consistent and the $L(\varepsilon)$ functions plotted in Figure 6.42 a) and b) show little difference in pattern, although a slight tendency is shown for the index L to increase with rate of shearing. Table 6.2 lists the values of E_u at 0.01% strain and the variation about the mean is generally within 10%.

6.9.4 The drained stress paths simply followed the $\sigma'_3 = 0$ line, inclined at 45 degrees to the hydrostatic axis. The stress-strain plots for the series are given in Figure 6.43 and appear to show a stiffer response in the faster tests. Whilst it is believed that the fast tests were drained over their stiff initial portion the rapid changes of modulus with strain imply similar reductions in C_v. If continued to large strains, the fast tests would become only partly drained and could eventually fail in an undrained fashion.

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1 It will be recalled that electrolevel measurements were not made for the pilot test MRU(i).

6.9.5 An overall impression of the findings for series RU and RD is given in Figure 6.44, where the values of $(E_u/p'_o)_{0.01}$ and $(E'/p'_o)_{0.01}$ are plotted against $t_{0.01}$. Also shown are the results of the two extension tests from K_o conditions, MRE1 and MRE1s, which had been carried out at different rates. The degree of scatter must be recognised, but the trends of this preliminary test series point to the following observations;

(i) The RU series with lubricated ends suggest a dependence of $(E_u/p'_o)_{0.01}$ on rate. This amounts to between 10 to 15% per log cycle of time, with the considered strain rates of 10^{-3} to 10^{-5}% per minute.

(ii) The same series point to fixed ended samples giving stiffness results which are perhaps 10% greater than those found with lubricated ends.

(iii) Extension tests for K_o normally consolidated soil, and the drained tests from the unconsolidated samples, appear to show a similar rate dependence. There is slight evidence that rate effects become less important at slower strain rates.

(iv) Comparing the undrained data with the RD series indicates the ratio[1] E'/E_u to approximately equal 0.65 for identical samples sheared at the same rate.

6.9.6 The two series of simple tests permit the preliminary conclusion that allowing drainage or slowing the rates of strain can reduce the scale of stiffness, but have less effect on the shape of the stiffness-strain characteristics.

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1 Linear elastic theory gives $E'/E_u = {}^2/3\ (1 + \nu')$, with $0.3 > \nu' > 0.2$ the equation predicts $0.87 > E'/E_u > 0.8$.

6.10 LOAD-UNLOAD CHARACTERISTICS AT SMALL STRAINS

6.10.1 The degree to which strains recover on unloading and the amount of energy dissipated in a load-unload cycle are crucially important when characterising a material's behaviour as being elastic or plastic. The test series RS and RR showed that irrecoverable strains develop after undrained loadings to moderate or large strains. To supplement these findings further tests were carried out with load-unload loops made at smaller strain levels. Three such experiments are described in this section; the first two represent the unloading stages of the unconsolidated tests MRU (iii) and MRUD (iv), the third consists of a series of loops carried out with the isotropically consolidated sample, MRI2C. For these tests the digitally controlled ram pressure source described in Section 5.5.11 was used to smoothly control the deviator forces developed by the stress path cell.

6.10.2 The stress-strain loops for the unconsolidated samples are shown on Figure 6.45; MRUD (iv) was performed drained with lubricated ends, sample MRU (iii) was undrained and employed fixed ends. Nevertheless, the characteristics show many similarities; both show unloading stiffnesses which are initially high and reduce rapidly with the size of the unloading strain increment.

6.10.3 Test MRUD (iv) included 10 minute pause periods at the maximum and minimum deviator stress conditions. An overall plastic strain, ε_p, of 0.027% was accumulated in the cycle which involved a maximum strain, ε_m, of 0.095%. For MRU (iii) the corresponding figures are 0.032% and 0.175%.

6.10.4 Sample MRI2C was subjected to three cycles of loading as illustrated in Figure 6.46, with 10 minute pauses allowed after each stage. In cycle 1 no permanent strains were observed for ε_m = 0.009%, but cycles 2 and 3 gave plastic strains of 0.01 and 0.025 for respective ε_m values of 0.045 and 0.116. It is interesting to note that the creep developed during rest periods was closely related to the plastic straining, and that no creep was observed for the first loading loop.

Each of the cycles can be considered separately. By defining an initial condition for the six individual phases, values of $E_{u(0.01)}$ and plots of $L(\varepsilon)$ can be produced. Figure 6.47 shows the $L(\varepsilon)$ graphs and indicates the curvature of the unloading phase to be slightly steeper than the loading one for cycles 2 and 3, although the patterns are similar in each case. The incremental loading values of undrained stiffness at 0.008% strain increased from 1.27×10^5 for cycle 1 to 1.38×10^5 and 1.44×10^5 for cycles 2 and 3.

6.10.5 The unload-reload experiments show a clear tendency for the ratio $\varepsilon_p/\varepsilon_m$ to increase with strain. The results are summarized by Figure 6.48, which includes data from some of the RR and RS experiments. For other experiments in the two K_o series' the imposed stress cycles were sufficiently large to induce failure or yield before the original deviator could be recovered. The plotted relationship is therefore only valid for moderate stress cycles which take place well within the outer State Boundary Surface.

6.10.6 The observation of non-linear stress-strain relations of even the smallest strains implies a degree of hysteresis and energy dissipation for very low amplitude cyclic loading. The data plotted on Figure 6.46 show this to be true, even when the imposed maximum strain falls below the level required for permanent strain development.

6.11 SUMMARY OF RECONSTITUTED TEST RESULTS

6.11.1 Reference to Section 3.2 shows many common features between the results found for reconstituted Magnus clay and the behaviour of Lower Cromer Till (LCT) described by Gens (1982). The one dimensional compression characteristics are similar, and over the stress range of interest, straight line relationships are found between ω/c and the logarithms of σ'_v, C_{ult} and p'_f. Although the critical state strengths were more clearly defined with LCT, the K_o consolidated Magnus clay demonstrated a similar response to undrained shear. The comparison is illustrated in Figures 6.49 and 6.50 which summarize the stress paths and stress-strain curves for extension and compression tests on the two clays. There is a close correspondence between the

results, with similar patterns of undrained stress paths, high stiffness at small strains and ultimate angles of friction.

6.11.2 The behaviour of the Magnus clay after isotropic consolidation diverged from that of K_o consolidated material in a way that parallelled Gens' observations with LCT. For both soils the differences are most marked at OCR1; this pattern is best summarized by comparing Figures 6.30 and 6.37.

6.11.3 Uniformity between Magnus clay and LCT is also found in comparisons of the anisotropy demonstrated by K_o consolidated soil. Gens' division of LCT strength anisotropy into a peak effect and a σ_2, or 'b', effect is applicable to Magnus clay. In the same way, the two soils show undrained stiffness in extension to generally fall approximately 50% below comparable compression test moduli.

6.11.4 The relationships between strength, initial stress and OCR for Magnus clay are illustrated in Figure 6.31 and Table 6.3. The peak ratio[1] of C_u/σ'_{vo} increases with OCR, but depends on the type of consolidation and the shearing direction; compression after isotropic consolidation gives the highest C_u/σ'_{vo} $^{+ocr\,1}_{k}$ Both perfect sampling and preshearing in extension reduce the peak strength, and hence the tabulated ratios. The data suggest that peak strength from a UU test on an intact retrieved sample is unlikely to fall below the ultimate value interpolated from series R. This result is important, as it places bounds to the potential differences between laboratory and insitu shear strengths for good quality retrieved specimens. Brittleness reduces with OCR and thus the maximum strength loss due to sampling is around 50% for OCR1 and 17% for OCR10. With push in samplers of small area ratio, the strength changes are unlikely to reach these maxima.

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1 The value σ'_{vo} is taken as the vertical consolidation pressure prior to undrained shear, perfect sampling or failure in extension.

6.11.5 The use of the electrolevel gauges for local strain measurements allowed new insights into the detailed stress-strain behaviour. The tests all showed secant moduli that were very high in the early stages of the shearing phase, but rapidly reduced with strain. This behaviour could be compared with the observed marked non-linearity in the oedometer swelling lines, which was summarized on Figure 6.15. To facilitate interpretation, it was found convenient to separate the scale of stiffness and the degree of non-linearity. The modulus at 0.01% strain, and its normalised ratios, provided suitable parameters with which to compare the scale of initial stiffness for tests carried out from a variety of initial conditions. The interpretation showed this parameter to depend on several factors, Figure 6.52 compares the five undrained series by plotting the ratio of $(E_u/p'_o)_{0.01}$ given by each particular test, to the corresponding value from the K_o compression series R. A range of OCR's is considered and it is encouraging to note that the full span for this comparative ratio falls between 0.7 and 2.4.

6.11.6 The data from the reconstituted tests showed the proposed function $L(\varepsilon)$ and the index L, to give convenient non-dimensional ways of expressing the degree of linearity. The ranges in $L(\varepsilon)$ and L were also surprisingly narrow. Figure 6.53 shows the envelope for all the reconstituted experiments and the curve for test MR2 appears to provide a representative median to the observed range. Figures 6.54 to 6.57 show how the $L(\varepsilon)$ curves for K_o consolidated soil at OCR's 1, 2, 4 and 8 depend on preshearing conditions and loading direction. It is evident that compression tests at OCR 1 give the only marked departure from the trend typified by test MR2.

6.11.7 The observed changes in secant moduli imply steeper reductions in tangent modulus with strain. Although differentiating to obtain tangent data produces more scatter and less consistent trends, the three typical curves illustrated in Figure 6.58 show the expected results with large reductions in tangent stiffness between 0.001 and 0.1% strain.

6.11.8 The summarized experimental data show that stiffness depends not only on mean effective stress and strain level, but is also influenced by; (i) the soil element's current position in stress space, and its proximity to the state boundary surface, (ii) the path which the soil followed to arrive at the current position, (iii) the duration for which the soil rests or drains at that point, (iv) the direction and rate of loading from the specified initial conditions, and (v) whether drainage is permitted during shear.

6.12 PART 3, DISCUSSION OF THE NATURE OF SOIL BEHAVIOUR AT SMALL STRAINS

6.12.1 It is of value at this stage to briefly consider the nature of the observed soil behaviour at small strains. The discussion will be useful in establishing a framework for understanding the tests on reconstituted soil, and will also be helpful when parallels are drawn with the behaviour shown by the intact Magnus samples.

6.12.2 The classical ideas of elasticity and plasticity have been crucial in developing models of soil behaviour, and it is important to establish the extent to which the experimental data conform to these simple descriptions of material properties. The models of the critical state type often assume the scheme in normalised stress space of an outer State Boundary Surface (SBS), on which the soil must deform plastically, and an inner region where the behaviour is elastic. Gens (1982) conducted a comprehensive series of experiments to test the existence of a unique SBS and showed the simple scheme to be unrealistic for anisotropically consolidated soil. He located an outer SBS using drained probes at constant stress ratios, but this lay beyond the undrained stress paths traced out by anisotropically or isotropically consolidated samples. His interpretation of the data led to the reclassification of the possible areas of triaxial stress space which is illustrated in Figure 3.1, and only the inner area between the compression and extension undrained stress paths was considered for convenience as "elastic".

6.12.3 In many experiments carried out on the Magnus clay the stress paths were initially directed within this space and the foregoing sections make it clear that the characteristics could not be strictly considered as elastic. Invariably, the stress-strain behaviour was strongly non-linear, plastic strains developed at relatively early stages in each test, and energy was dissipated in very small amplitude load-unload cycles. Indeed, a close examination of the Magnus undrained test data suggests the behaviour is best interpreted by considering four distinct stages of axial stress-strain behaviour;

(i) Up to the point where ε_a reaches 0.01%, the stress-strain curves are non-linear and stress path dependent, but plastic strains and creep displacements are negligible. The undrained stress paths plot as straight lines, but load-unload cycles show significant hysteresis.

(ii) For axial strains between 0.01% and 0.1% the soil behaviour remains stress path dependent and non-linear, but plastic deformations start to develop and creep under sustained load becomes significant. Between the stated strain limits, secant moduli fall to between 0.18 and 0.45 of their initial values. (The tangent moduli show correspondingly swifter changes and fall to less than 20% of their values at 0.01% axial strain).

(iii) After attaining 0.1 to 0.2% strain, the samples show a form of volumetric yield. If the stress path is approaching an outer bounding surface, a rapid change in stress path direction might take place. In other cases, the stress paths still deviate, but in a less marked way.

(iv) In an intermediate strain region, between 0.1 and 2.0%, tests from different conditions show widely dissimilar characteristics. This is clearest when comparing the normally consolidated tests, MR1, MRE1, MRS1 and MRI1. Nevertheless, in all tests the stress paths have reached the outer boundary surface after developing axial strains between 2 and 5%. From this surface, the samples tend towards their ultimate conditions.

6.12.4 It would thus appear that at least 3 stages of yielding must be considered to understand the undrained soil behaviour. Before these stages can be plotted as yield surfaces in even triaxial stress space it is necessary to know how the loci would be intercepted by more general drained stress paths. Gens (1982) carried out many tests to identify the outermost boundaries for Lower Cromer Till, and the various drained stages of the Magnus series may be used to help sketch the two inner zones indicated on Figure 6.59. The boundaries I and II respectively correspond to the limits of (i) recoverable straining and (ii) stiff behaviour without volumetric yield.

6.12.5 The proper definition of these regions would require many careful tests, for which internal instruments for radial strains measurement would be vital. It was not possible to include such a full investigation in the Magnus programme, but data collected from the K_o swelling and compression stages of the tests provide a preliminary means of evaluating the drained characteristics. Plots of equivalent isotropic bulk and shear moduli have been derived from the swelling stages of the MR series and these are presented in Figures 6.60 and 6.61. It is interesting that in each case the soil stiffnesses had reduced to small fractions of their initial values with the attainment of 0.1% axial strain, and that the reductions of modulus between 0.01 and 0.1% were comparable with those observed in undrained tests. Similar observations were made from the drained stages of other the reconstituted test series including those involving isotropic consolidation (see Figure 6.62).

6.12.6 Clearly, it is not possible to specify a set of yield surfaces in terms of axial strain alone. This can be simply shown by recalling that for any point in triaxial stress space there will be stress paths that result in large volume and shear strains without any significant changes in sample length. Indeed any strain criteria should be formulated in terms of combinations of shear and volume strain invariants. Attempts to do this have been made by the author, using the limited data available, but the results have not been encouraging.

6.12.7 However, it is important to note that the major principal strain at which yield at surface II was interpreted in all cases fell between 0.1 and 0.2%. In order to provide a provisional definition of Zones I and II that allows the extent of the regions to be discussed, the two limits ε_1 = 0.01% and ε_1 = 0.1% are proposed as boundaries for triaxial stress paths. The looseness of these criteria is recognised, but improvements are only feasible if suites of further tests are performed in which the radial strains are measured directly, and the development of plastic strains is studied from appropriate load-unload cycles. The test equipment and procedure for such a programme would have to be carefully chosen; recommendations were given in Section 5.8.18 for ways of improving strain instrumentation and it is vital that quiet, precisely controlled, pressure sources should be used.

6.12.8 Having accepted provisional definitions for the two inner Zones, it is possible to sketch these regions for a variety of conditions. Figure 6.63 shows the results for K_o consolidated soil at OCR1. The two undrained paths identify points on the boundaries of I and II, and K_o unloading indicated major principal strains of 0.01 and 0.1% at the range of co-ordinates plotted on the swelling line. The K_o consolidation path for MR8 was temporarily halted at σ'_v = 360 Kpa and, after a two day pause, redirected towards point 0. The stress increments required to develop 0.01 and 0.1% strain from this condition were proportioned to find the co-ordinates for I and II expected for a similar probe from σ'_v = 400 Kpa. The remaining areas of uncertainty fall in the regions where ε_r is the major principal strain, and a shape has been sketched which is ovoid and does not cross the upper bounding surface given by the undrained stress path of test MR1.

6.12.9 Following a similar procedure, the zones have been sketched for points with their origins on the K_o swelling line at OCR's 2, 4 and 18. The results are indicated on Figures 6.63 b) and c) and a number of interesting points emerge :

(i) In travelling from OCR1 to 2 there is a slight increase in the total areas of Zones I and II. For OCR1 the Zones appear to be constrained by the proximity of the state boundary surface. Swelling from OCR2 to 4, and from 4 to 18, induces diminutions of the overall size of both regions. At OCR18 the limits for Zones I and II again appear to be constrained by the short distance to the failure line.

(ii) The Zones are not precisely centred on the K_o lines, and the points of origin, 0 to 0''' do not sit at mid-points of the regions. For the considered OCR's, the shortest distance between the origin and the boundary lies in the current direction of the stress path. When the stress path is reversed, for example between OCR's 1 and 2, the origin has to track across the long axes of the zones and especially stiff behaviour could be expected.

6.12.10 The small strain zones are thus translated and altered by consolidation and swelling. Data from the test series RS and RR can be used to show that undrained straining can produce similar effects. Figure 6.64 traces the Zones for K_o normally consolidated Magnus clay following perfect sampling and extension to failure without drainage. The lower limits for the first case were projected from the undrained extension test MRE1, and the $\phi' = 30°$ line was used to help sketch the Zones for the second. In a similar way to the swelling process, the overall size of the small strain zones are reduced by shear straining, and the stress origin moves towards the boundaries in the direction of the current stress path. Following the shifts and alterations of the two zones thus provides an approximate way of tracking the Kinematic Yield loci within the permissible stress space.

6.12.11 Although the criteria for the Zones have been loosely defined, and their extent incompletely mapped out, the sketching of the regions offers illuminating insights. The characteristics of the small strain Zones can be used, in at least a qualitative way, to describe the observed sensitivity of stiffness variations to stress path direction, consolidation type and undrained disturbance.

6.12.12 Consider, for example, two perfect samples taken from normally and heavily overconsolidated conditions, as shown in Figure 6.65. For soil at OCR1 the position of the boundary to Zone II is not symmetrical, and it can be seen that the response when extended in direction OB is stiffer than that expected for compressive loading on path OA. For the perfectly taken sample, the pattern is reversed, and compression loading on BC is likely to show far higher moduli than further extension on BD. The heavily overconsolidated sample shows the mirror image of this behaviour; insitu the response in extension is softer than that for compression, but after traversing from O to E undrained, the response to further compression is not likely to be as stiff as that for a extension path in the direction EO.

6.12.13 The scheme of the bounding Zones provides a helpful way of explaining an otherwise difficult argument, i.e., that even perfect sampling can completely invert the stress path dependence of stiffness for intact soils. The ideas can also be applied to consider the growth and shifts of Zones I and II with time. The results of the test MRIA, which examined the undrained response of aged K_o consolidated clay at OCR1, showed that both the outer state boundary surface and the limits to Zones I and II extended out from the stress origin with age. Conversely, the tests discussed in Section 6.9 suggest that the extent of each Zone contracts when slower rates of probing are employed.

6.12.14 The conclusions drawn regarding the nature of the reconstituted soil behaviour at small strains were useful in formulating the test programme on intact samples, and also helped in the interpretation of these experiments. It was clear that unconsolidated material could respond differently to anisotropically consolidated soil, and that the details of the re-consolidation paths

would be important for intact samples. For example, re-imposing the estimated insitu stresses on lightly overconsolidated clay without performing a swelling or ageing stage would be likely to produce undrained characteristics which are more compatible with normally consolidated conditions. Similarly, there appeared to be little benefit in attempting to study the stress path dependence of stiffness or strength using unconsolidated samples. The following, fourth, part of the chapter discusses the results obtained from intact programme that was finally undertaken.

6.13 PART 4,TRIAXIAL TESTS ON INTACT MAGNUS SAMPLES

6.13.1 The reconstituted test series evaluated the behaviour of sedimented clay and showed how this provides a framework for understanding the properties of the Magnus soils. The comparison may be justified when it is recalled that almost the entire soil profile under the location of Leg A4 was interpreted in Section 6.3 as being clays of glacio-marine or marine origin, although the apparent overconsolidation of the upper layers was thought to have resulted from wave action rather than static loading.

6.13.2 The aim of the intact tests was to complement the work with reconstituted soil, and to check for any special features of the intact materials that were not reproduced by consolidation from slurry. The reconstituted experiments showed that the sampling processes outlined in Section 5.2 are likely to modify the response of intact samples so that their response in unconsolidated tests will differ in a number of ways from insitu behaviour. It is also unlikely that insitu behaviour can be completely recovered by simply reimposing the estimated field stresses. The intact programme included both unconsolidated and anisotropically reconsolidated experiments, but it is recognised that both types of test are unlikely to perfectly match insitu behaviour. These data will be combined with the reconstituted results to bracket a range of parameters for the soil layers at Magnus.

6.13.3 In order to carry out anisotropically consolidated (CAU) tests, it is first necessary to estimate the insitu effective

stresses. Vertical pressures are simply obtained from submerged densities, but the horizontal stresses present difficulties. The relationships between C_u/σ'_{vo} and OCR found from the reconstituted tests were presented in Figure 6.51. These data were combined with the Fugro undrained shear strength profile and the K_o formula of Mayne and Kulhawy (1982)[1] to calculate a variation of K_o with depth. The procedure was similar to that outlined in Section 3.4 and allowance was made for sample disturbance, but the profile is only strictly valid for statically consolidated sediments. The field measurements given by Powell et al. (1983), Powell and Uglow (1985) and Clarke (1984) for terestial tills might be considered relevant, and these are shown with the 'sedimentary' profile on Figure 6.66. Also shown is the lower bound for K_o given by the Jaky formula, and a line interpreted by the author; this was constructed by reference to the assembled data, and the argument of Hight (1983) that wave induced consolidation would generate smaller horizontal stresses than literal overconsolidation. The potential for inaccuracy is largest near the surface and significant errors are unlikely below 40 metres depth.

6.13.4 The interpreted K_o profile was used to evaluate the initial stresses for the CAU intact series. The reconsolidation paths were constructed so that the desired initial conditions could be obtained by continuous pressure changes. These gave smooth transitions from the sampled state to a final condition on an assumed K_o line.

6.13.5 Fifteen undrained tests on intact samples will be discussed in this section. Three of these were pilot tests that were carried out before the electrolevel gauges were available, and of the 12 instrumented experiments, six were anisotropically reconsolidated. Table 6.4 divides the list of tests into the instrumented Series I and the pilot Series P. The table also summarizes the initial conditions and main results of the two series.

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1 See Equation 3.1.

6.13.6 After being saturated following the procedures discussed in Chapter 5, the samples' mean effective stresses reflected the insitu pre-sampling conditions and chain of events from sampling to testing. The observed values of p' were relatively high and varied from around 400 Kpa near the sea bed to 800 Kpa at 93 metres depth. Four of the samples from the overconsolidated upper layers were swelled back isotropically to produce more appropriate pre-shear effective stresses. The quoted values of p' cannot be compared with those determined by routine methods in commercial laboratories, where wet side drains and filter stones are usually applied to samples, as these allow significant and uncontrolled increases in water content to take place before any measurements are made.

6.13.7 The behaviour of the intact soil is first discussed by reference to the stress paths shown in Figures 6.67 to 6.72. The samples are divided by their soil groups, and for II and III the CAU tests are separated from the UU and CIU experiments. The developed strains are indicated on the plots, as are the reconsolidation paths for the CAU experiments. The following paragraphs consider the results obtained from groups I, II, III and V. No test data are available from group IV, although two of the group V tests were conducted on samples taken at the junction of the two geotechnical units.

6.13.8 The characteristics observed for group I are most similar to those noted for heavily overconsolidated reconstituted soil. The stress paths show initial straight line portions, which tend to the right after 0.1% axial strain. Between 0.1 and 1.0% strain all three paths curved to follow a Hvorslev type surface until ultimate conditions are achieved when axial strains of 15 to 20% had developed. Comparing the shape of the stress paths with those of the reconstituted series R, the group I experiments show apparent OCR's greater than 20. Sample MI1 failed with an angle of friction of 30°, but MI2 and MI3 gave final ϕ' values of 29 and 28 degrees respectively. The anisotropically consolidated test MI2, showed broadly similar features to the two isotropically swelled samples, MI1 and MI3.

6.13.9 Four tests were carried out for group II, and their stress paths are given in Figures 6.68 and 6.69. The general characteristics are comparable with the reconstituted soil at OCR's between 2 and 8. From an unconsolidated condition the stress paths for triaxial compression were slightly inclined to the right, and deviated from their initial direction from 0.5% strain onwards. After strains of approximately 2% had been reached, both samples followed Hvorslev type surfaces until peak strengths developed at 12 to 15% strain. From the peak strength condition, the two experiments showed a relatively rapid strain softening with ϕ' falling from 29° to around 25° at 30% strain. Anisotropic consolidation led to similar behaviour, although the initial stress path directions were more steeply inclined to the right, and the peak angles of friction were slightly lower. All four experiments gave ratios of initial to final $(\sigma'_V + \sigma'_H)/2$ of around 1.8, and may thus be compared most closely with the reconstituted test MR4.

6.13.10 The stress paths for group III are shown in Figures 6.70 and 6.71. The UU tests I9, P1 and P2 are supplemented by the reloading path for test I8, which had accidentally unloaded from failure in compression. The three unconsolidated tests show features different to those of groups I and II; the paths are initially vertical and turn leftwards at strains[1] of 0.1%, but curve back to the right at approximately 0.6% strain. Two of the tests follow a short section of the dilatant part of the State Boundary Surface before reaching their peak strengths, whilst P2 follows a contractant path for a similarly short interval. The observed pattern has common features with both the perfectly sampled tests at OCR's 1 and 2, and the recompression test MRR1. The stress paths show little overall change in $(\sigma'_V + \sigma'_H)/2$ from start to failure, and are therefore comparable with the behaviour expected for retrieved samples of lightly overconsolidated clay. It is also noteworthy that the tests all show reductions in ϕ' from around 29° at peak to 26° at ultimate conditions.

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1 The smaller strain levels are not marked for the un-instrumented samples.

6.13.11 The stress paths for the two CAU tests on group III material are plotted in Figure 6.71 and show a similar response to lightly overconsolidated, reconstituted soil at strains less than 0.5%. The paths are initially almost straight but a form of yielding develops at 0.1% strain when the paths turn to the left. However, both samples are able to carry increasing deviator stresses up to strains of 2 to 3% in a way that could not be compared with the response of the reconstituted test series R. In particular, neither test shows the large reduction in mean effective stress between peak and ultimate conditions that was typical of the reconstituted soil at small OCR's. Furthermore, the experiments indicate reductions in ϕ' from peak strength, rather than the increases observed in series R.

6.13.12 Figure 6.72 shows three stress paths for soil group V; the general pattern suggests a higher degree of overconsolidation than group III; for failure to occur $(\sigma'_V + \sigma'_H)/2$ has to increase to approximately 1.5 times its initial value. The UU stress paths show only a slight tendency to curve to the left after 0.1% strain and show similar trends to the paths noted for group II. The response of the CAU test, MI12 also shares common features with the second Magnus group, although the group V experiments give slightly lower peak and ultimate ϕ' values.

6.13.13 The peak strengths from the intact experiments are summarised in Table 6.4 and on Figure 6.73. The latter figure compares the data with the trend line from the Fugro UU tests and a lower bound, normally consolidated, profile. Within group I there is good agreement between the MI series and the Fugro data, but for the lower soil layers the tests carried out at Imperial College give strengths which are 15 to 40% higher. Reference to Table 6.4 shows no tendency for the CAU samples to be stronger than unconsolidated samples, so water content changes during reconsolidation are unlikely to have been responsible for the higher strengths. One possible source for the observed difference is the magnitude of the cell pressures applied before testing. The Fugro procedure was to apply a pressure of 600 Kpa, which is far smaller than the field mean total stress. With the samples tested at Imperial College, a 600 Kpa cell

pressure would only be sufficient to just develop positive initial pore pressures in the upper layers, and residual suctions would be expected in groups III, IV and V. The much larger cell pressures applied to the MI samples could have thus resulted in greater initial mean effective stresses than those acting in the Fugro samples; the relief and re-imposition of large total stresses are unlikely to have been reversible processes. A second possible source of the observed strength difference is the extended storage time of the MI samples.

6.13.14 Figure 6.73 gives a profile of insitu triaxial shear strength which was constructed by allowing for the 'sampling' strength losses observed in the reconstituted test series. For this calculation, the above discussed total stress effects were neglected. The same figure shows the variation in apparent OCR with depth, and it will be seen to be in broad agreement with the conclusions formed from inspection of stress paths of the MI series.

6.13.15 The stress-strain response of the intact samples is illustrated in more detail by the semi-logarithmic plots given in Figures 6.74 to 6.77. Where the failure strains exceed 10%, the peak conditions are indicated numerically.

6.13.16 The small strain characteristics are again considered by plotting the $L(\varepsilon)$ functions for each test, and by comparing values of $E_{u(0.01)}$ and $(E_u/p'_o)_{0.01}$. Table 6.4 and Figures 6.78 to 6.81 summarize the data and the plots also show the results from the typical reconstituted test, MR2, for comparison. The intact samples show similar trends to the reconstituted samples; the curves are all of the same general shape, and the steepest non-linearity is exhibited by the lightly overconsolidated layers in Group III.

6.13.17 The scale of stiffness for the intact tests is summarized in three Figures. Figure 6.82 shows the ratio $(E_u/p'_o)_{0.01}$ as a function of apparent OCR, and illustrates two main points; (i) the ratio is lowest where the soil is almost normally consolidated, (ii) that CAU experiments tend to give ratios which exceed those for CIU or UU tests.

6.13.18 The trend of the CAU data is compared in Figure 6.52 with the ratios found in the reconstituted test programme, and it is remarkable that the intact data fall within 25% of the results for Series R. This close agreement is shown again in the profiles of $E_{u(0.01)}$ given in Figure 6.83, where the results of the MI series are compared with lines constructed from the Series R and RE tests by using the derived profiles of OCR, K_o and σ'_v.

6.13.19 The aim of the triaxial programme with intact samples was to highlight any features that might be lost in the preparation of reconstituted blocks of clay, and to help bracket the soil parameters for the Magnus site. With this aim in mind the results can be summarized as follows;

(i) The general patterns of behaviour were similar to those expected from the reconstituted tests, when allowance was made for the probable effects of sampling. Exceptions to this trend were found in the tendency for post peak reductions in ϕ' which suggest the development of residual fabric, and the absence of the strong contraction observed with lightly overconsolidated samples. These features might be explained by sampling effects, macro-fabric, bonding or soil ageing.

(ii) The peak angles of ϕ' in the intact soil were typically 1 to 2° lower than those for the re-constituted clay. This might be related to the reductions in PI that accompanied reconstitution, or might be a result of macrofabric phenomena.

(iii) The stiffness characteristics of the soil at small strains showed remarkable agreement with the reconstituted compression tests. This similarity covered both the scales of stiffness and the degrees of non-linearity.

6.14 PART 5, SPECIAL TESTS : INSITU PRESSUREMETER DATA

6.14.1 As part of the field work at the Magnus site, a programme of Push in Pressuremeter (PIP) tests was carried out, and the detailed results are given in Appendix 3. Although these experiments will be discussed in more detail in Chapter 8, it is worthwhile to briefly consider the relationship between the field results and the laboratory derived data.

6.14.2 Each PIP test consists of an expansion stage which involves a 16 to 17% volume increase, a pause period, a 3% reduction in volume, a second pause, and finally a reflation stage. Undrained shear strength is calculated from the first part of the test, and shear modulus G, is determined from the unload-reload loop. The parameters interpreted by the operators, Stressprobe Ltd., are listed in Appendix 3, but it is useful to consider the unload-reload loops in more detail. Inspection of the volume change - pressures curves shows a different pattern to the straight line predicted by linear elasticity. It is clear that the deduced shear modulus is highly sensitive to the size of the pressure increment considered, and that the quoted results are many times smaller than the initial stiffness which control the response to small changes in cavity pressure. This behaviour is analogous to the triaxial test characteristics of both intact and reconstituted Magnus clay.

6.14.3 The quoted PIP results for Cu and $3.G^1$ are compared with the triaxial data on Figures 6.73 and 6.83. Considering the strength data first, it may be seen that the pressuremeter results plot above the Fugro UU line at depths greater than 20 meters, and that the trend is similar to that of the triaxial tests carried out at Imperial College. Turning to the stiffness profiles, it is apparent that the triaxial $E_{u(0.01)}$ values exceed the interpreted pressuremeter line by a large factor. Recalling the steep curvature of the pressuremeter

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1 For these purposes 3G is assumed to be equivalent to Eu in the way predicted by linear elastic theory.

plots, it is probable that better agreement would be found if account was taken of the non-linear response of the field tests, and shear characteristics were interpreted from the behaviour at far smaller cavity strains.

6.15 RING SHEAR EXPERIMENTS

6.15.1 Martins (1983) showed that the development of residual fabric in the soil close to a pile can have an important influence on shaft capacity, and it was thus desirable to investigate the residual strength of Magnus clay using soil-soil and soil-interface direct shear tests. The review given in Chapter 3 concluded that the ring shear apparatus is the most appropriate tool for such work, and Mr. L. Lemos kindly conducted a series of experiments with samples prepared from reconstituted Magnus clay. The following paragraphs summarize his results, and readers are referred to Lemos (1985) for further details of the equipment, method of testing and interpretation.

6.15.2 The behaviour of Magnus clay in a slow ring shear experiment is illustrated in Figure 6.84. The remoulded soil had been consolidated to a vertical effective stress of 500 Kpa, and then sheared at a rate of 0.058 mm per minute. A peak ϕ' of 29.7° was observed after a small displacement, and this slowly reduced until an ultimate value of 27.5° was obtained. Tests were carried out at different rates and the ultimate angle was found to vary as shown in Table 6.5. The quoted values are in good agreement with the tendency towards residual fabric that was observed in the triaxial tests on intact samples, but suggest that in soil to soil shear the behaviour is more turbulent than transitional, (see Lupini et al. (1981)).

6.15.3 In Section 3.3 it was shown that soil-interface shear can produce very different results to soil-soil shear for low plasticity clays. In order to investigate possible residual fabrics developed at the side of the driven piles on which the Magnus platform rests, a series of tests was carried out with a mild steel interface. The stages of the experiment were as follows :

(i) Consolidate the soil to σ'_v = 200 Kpa.

(ii) Rapidly shear the sample against the interface, with five successive 'pulses', each consisting of 20 minutes shearing at 133 mm/minute with 10 minute pauses between pulses.

(iii) Consolidate to 500 Kpa.

(iv) Rapidly shear the soil against the interface for 200 mm at 133 mm/minute.

(v) Unload to σ'_v = 200 Kpa.

(vi) Repeat stage (ii) and (iii).

(vii) Slowly shear the soil against the interface for 200 mm at 0.002 mm/minute.

6.15.4 The above sequence attempts to simulate pile driving, followed by consolidation and sudden storm loading in the case of (iv), or slow statical testing in the case of (vii). The stresses were judged to be appropriate to conditions at mid-pile depth for the Magnus site, and the interface was made from mild steel that was sand blasted to give a typical surface roughness[1] of 1.3 μm. Curves relating τ/σ'_n to displacement for the various test stages are given in Figures 6.85 to 6.87. The first details the response during the 'driving' stage (ii). It can be seen that the friction angle for the first pulse is very low, and reaches a minimum of around 12.7° after 10mm displacement. With continuing displacement ϕ' increases and gives a final value of 19°. The behaviour during the first pulse is therefore similar to the Lower Cromer Till characteristic illustrated on Figure 3.2. After the 10 minute pause period, the start of the second pulse shows a transient peak of around 21° but again tends towards 19° after

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1 Measurements were made of the centre line average (CLA) roughness.

a further metre of travel. Pulses 3, 4 and 5 give similar results, and confirm a ϕ'_r for large displacements of 19° when shearing at 133 mm/minute with σ'_n = 200 Kpa.

6.15.5 The results for the simulation of storm loading are shown in Figure 6.87. The consolidation to 500 Kpa, and subsequent reshearing at 133 mm per minute, produce a transient peak with ϕ' = 20° that reduces to a minimum angle of 16° after approximately 70 mm travel. The friction angle then shows signs of slowly increasing with displacement.

6.15.6 The second 'driving' phase was preceded by a swelling stage that overconsolidated the sample, and the trace for the first 'pulse' of shearing at 133 mm per minute reflects the higher peak ϕ' for the overconsolidated clay. The plots of Figure 6.86 indicate an initial angle in excess of 35°, which falls to 19° after 1 metre of displacement. Subsequent pulses produce far smaller peaks and tend to the same ultimate value of ϕ'_r. After the driving phase, the sample was again consolidated to 500 Kpa, and then slowly sheared. The trace shown in Figure 6.87 is unsteady, but indicates a slow fall in ϕ'_r from around 18.5° to a final value of 16.5°.

6.15.7 In summary, the interface tests show that the response after driving and full equilibration is not greatly dependent on rate of displacement. Both fast and slow shear will invoke initial angles, ϕ'_r of around 19° for normally consolidated soil, and these gently fall with displacement to give minima of between 16 and 17° after tens of millimetres relative displacement. Such angles are considerably smaller than the peak or residual soil-soil values, but are 4 to 5 degrees higher than the absolute minimum obtained in the first shear phase after setting and consolidating against a smooth interface. If equilibration leaves the soil in an overconsolidated state, initial ϕ' values greater than 30° might be mobilised.

6.16 RESONANT COLUMN TESTS

6.16.1 One of the unusual features of the triaxial programme undertaken at Imperial College was the use of local instrumentation to

measure the initial response of soils to shear. The data contradict the common assumption of linear elastic behaviour and are also at variance with stiffness profiles interpreted from the pressuremeter tests by Stressprobe. It was therefore decided to carry out some dynamic laboratory testing on reconstituted soil, which would offer an entirely independent approach to the determination of stiffness properties at small strains.

6.16.2 Fugro Ltd. had recently commisioned a Stokoe resonant column apparatus and kindly offered to perform three tests on samples cut from a prepared block. The principles of the experiment are discussed by Richart et al. (1970), and the determination of resonant frequencies of vibration allows apparent shear moduli to be calculated for a range of strain amplitudes, albeit through an approximate elastic theory. The difficulties of test interpretation were briefly discussed in Section 5.7.

6.16.3 Three samples were isotropically consolidated in the apparatus, and the use of radial drains allowed each step to be completed in 24 hours. Table 6.6 lists the initial conditions for the tests, and summarizes the results. Characteristic curves of deduced G and damping ratio are plotted against 'typical' peak shear strain in Figures 6.88 a) and b). It is interesting to note the following general points from these data :

(i) The characteristic shapes of the G-log γ curves are similar to those found in the triaxial experiments.

(ii) The damping ratio curves rise steeply after 0.005% shear strain, indicating sharp increases in the hysteresis of the materials under test.

(iii) The tests become unstable before reaching 0.1% shear strain.

(iv) The data ~~infer~~ imply constant shear moduli for shear strains less than 0.001%.

6.16.4 The resonant column characteristics are thus in agreement with the interpretation of the triaxial data presented in Section 6.12. However, it is difficult to discuss the results in quantitative way as the shear strains, $\gamma_{z\theta}$, cycle with time and vary with radius. The quoted strain is the peak value that occurs at a 'typical point' whose radius is approximately $^2/3$ that of the sample. In order to compare the results with the triaxial data, an equivalence is required between the quoted $\gamma_{z\theta}$ and the axial strain in an undrained test. Assuming $\gamma_{z\theta}$ to be the only component of strain for the chosen axes, the two tests can be interrelated through invariant measures of shear strain such as γ_{oct} or E. Such a comparison leads to the equivalence $\gamma_{z\theta} \equiv \sqrt{3}\,\varepsilon_a$. Recalling that $\gamma_{z\theta}$ is the peak shear strain, the equivalence $\gamma_{z\theta} \equiv \varepsilon_a$ might be more appropriate.

6.16.5 The scales of stiffness can be more easily compared by conversion to an equivalent isotropic Young's modulus, $E_u = 3G$. The maximum ratios of E_u/p'_o from Table 6.6 can then be compared with the results of triaxial tests MRI1, MRI4 and the fastest MRUU test, and these are given in Table 6.7. Recalling that the triaxial $E_{u(0.01)}$ is typically 60 to 70% of the maximum indicated at the start of each test, it may be concluded that the dynamic shear stiffness is only slightly larger than the triaxial isotropic equivalent at OCR1, and smaller by up to 40% at OCR4. The unconsolidated test, MRC3, showed a particularly small modulus, but this may have been the result of poor contact between the top cap and sample, as the consolidation pressures for this test were relatively low.

6.16.6 The uncertainty regarding the conditions in the resonant column test, and the instability beyond the 0.05% shear strain, makes the definition of an $L(\gamma)$ function which is compatible with the triaxial $L(\varepsilon)$ curves difficult. The term L quoted in Table 6.6 is defined as the ratio of moduli at 0.05%[1] and 0.005%[1] shear strain, and gives values between 0.28 and 0.51; it is encouraging to note that this range virtually coincides with the span of $E_{u(0.1)}/E_{u(0.01)}$ from the triaxial tests.

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[1] Approximately equivalent to similar axial strains in undrained triaxial tests.

6.16.7 In summary, the resonant column results are compatible with the triaxial test data. Despite the differences in stress path, rate of shear and time for consolidation, the same general patterns of behaviour are found at small strains. The dynamic tests also showed that linear elastic behaviour is only found when the peak shear strains remain below 0.002%, and this confirms the observation of a non-linear response at even the smallest strains in the triaxial tests.

6.17 SUMMARY

6.17.1 Chapter 6 reviewed the field investigations at the Magnus site, and reported laboratory research into the properties of the soil layers. It was shown that the soil profile could be divided into five geotechnical units over the depth of interest. With the exception of two relatively thin sand layers, the five groups were predominantly glaciomarine clays of similar composition, which were mainly differentiated by changes in stress history and undrained shear strength.

6.17.2 Comparisons between the undrained shear strength plots derived from different commercial site investigations, from the push in pressuremeter and the triaxial tests carried out at Imperial College, showed the difficulties in defining a unique profile of Cu with depth. Peak strengths were found to depend on sampling method, specimen size, type of loading and stress history prior to testing.

6.17.3 The programme of experiments on reconstituted soil was used to show the patterns of insitu behaviour that could be expected for sedimented clays. The characteristics of the reconstituted Magnus soil were shown to be similar in many respects to the low plasticity clay investigated by Gens (1982). In particular, the response of the K_o consolidated clay to undrained shear was strongly anisotropic and there was a clear distinction between peak and ultimate strength conditions; the marked brittleness in compression was shown to reduce with OCR. Similarly, isotropically and K_o consolidated samples of Magnus clay showed differences that were comparable with those noted by Gens.

6.17.4 Test series were undertaken to investigate the effects of perfect sampling, and the large strain reversals that were thought to accompany field sampling operations. The results illustrated how these processes alter the undrained stress paths and peak strengths of K_o consolidated soil.

6.17.5 A special feature of the laboratory work was the use of electrolevel gauges to measure local axial strains in triaxial tests. The experiments revealed that the stress-strain characteristics before yield are both stiff initially, and strongly non-linear. The results were interpreted by separating the scale of stiffness from the degree of linearity and by estimating limits to the zones of recoverable straining and stiff, small strain, behaviour. The data showed consistent trends with similar reductions of stiffness with strain in most experiments. The scales of stiffness were found to vary with stress history, loading direction and rate of strain.

6.17.6 A programme of triaxial tests on intact samples was described, and an interpretation made within the framework established from the reconstituted experiments. The stress paths observed for unconsolidated samples and specimens consolidated under various conditions generally fitted the expected patterns. Differences noted between intact and reconstituted samples included slightly smaller peak ϕ' values, the absence of strongly contractant behaviour in compression tests from lightly overconsolidated conditions, and the tendency towards a residual strength at large strains. The small strain response of the intact soil showed similar trends to the reconstituted samples, and the two test programmes allowed the profiles of $E_{u(0.01)}$ at Magnus to be closely bracketed.

6.17.7 Evidence to support the interpreted high initial stiffnesses and marked non-linearity was found in both the field pressuremeter data and the resonant column tests on reconstituted soil.

6.17.8 Ring shear tests were carried out on the Magnus clay to investigate the effects of large shear strains on macrofabric. A soil-soil test confirmed the tendency of the intact samples to develop

a residual angle, ϕ'_r, of approximately 26°. Soil-interface tests showed the first phase of shearing against a mild steel surface to produce a remarkably low residual angle in the same way as Lower Cromer Till. On further shearing this angle climbed to around 18° and became almost independent of shearing rate. This latter feature is likely to be of major importance in analysing the development of shaft friction for the Magnus foundations.

CHAPTER 7

INVESTIGATIONS AT THE CANONS PARK SITE AND TESTS ON RECONSTITUTED LONDON CLAY

7.1 INTRODUCTION

7.1.1 It was shown in Chapter 3 that the current understanding of the processes of pile installation, equilibration and loading is not sufficiently developed for confident predictions to be made for many important parameters, especially with overconsolidated clays. An extensive laboratory test programme has been carried out to determine the initial conditions and fundamental properties of the Magnus soil profile, but further semi-empirical data are required before the pile groups can be properly analysed. In the absence of the instrumented pile data from Magnus, field scale instrumented model pile studies were identified as being an important means of relating laboratory test results to full-scale behaviour. The experiments could not be carried out in North Sea soils, but a programme of tests conducted in well documented strata could give valuable insights into the effects of installation, consolidation and monotonic loading. It was decided to perform such a test series in the London Clay, and this chapter discusses the site investigations at the test site, and also reports a more general programme of laboratory work with reconstituted London clay.

7.1.2 The Building Research Station's (BRS) piling research group kindly offered the use of their test bed facility at Canons Park for the field experiments. The BRS test site has been established inside a Government storage depot in North London, the location of which is shown on Figure 7.1. Although 12 miles from South Kensington, the provision of site accommodation, power, water and good security offered many advantages.

7.1.3 The Geology at the test bed site consists of a thin layer of recent soils overlying London clay : beneath are found the Woolwich

and Reading Beds, and the Upper chalk. The "1-inch" geological map, No.256, shows a cross section adjacent to the site from which the thicknesses of the strata may be estimated. The total present Eocene thickness is indicated as 45 to 60 metres, of which the upper 30 are shown as London clay. Boreholes through the full Eocene succession found under the high ground at Stanmore and Hampstead show the maximum thickness of the upper Bagshot and Claygate beds as 35 metres, with approximately 70 metres for the London clay. The greatest probable historical overburden at Canons Park is therefore likely to be equivalent to around 65 metres of surcharge.

7.1.4 Burnett and Fookes (1974) discuss the engineering geology of the London clay and the regional variations in fabric and composition; it is not intended to repeat their summary here. Many detailed studies have been made of the geotechnical properties of London clay, and interested readers are referred to Bishop, Webb and Lewin (1965), Marsland (1971), Atkinson (1973), Sandroni (1977), Apted (1977) and Costa Filho (1980). However, little specific site investigation information was available for Canons Park, although Price and Wardle (1982) had reported the results of a series of cone penetrometer soundings. It was therefore important to carry out field work to establish the variations in soil type and properties with depth. The studies of the Magnus soils also emphasized the need to perform high quality laboratory studies to investigate the properties of intact and reconstituted soil.

7.1.5 Surprisingly, a literature review showed that a comprehensive[1] study of the undrained behaviour of K_o consolidated reconstituted soil in the triaxial apparatus had yet to be carried out for London clay. The required test programme was therefore likely to be extensive. Fortunately, Dr. Fourie was also interested in the properties of London clay, and the laboratory work was divided into approximately equal parts.

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[1] i.e. compression and extension tests from K_o stresses for a range of OCR's.

7.1.6 The following sections describe the field investigations at Canons Park, and the laboratory studies made with retrieved samples. The approach and techniques were developed from the Magnus test programmes, and the laboratory experimental procedures were fully described in Chapter 5. A parallel, but smaller scale, study was carried out using samples taken from the Bell common site, which is near Epping Forest. The data from the latter study were discussed by Fourie (1984) and are synthesized with the Canons Park results by Jardine et al. (1985).

7.2 GEOTECHNICAL PROFILE AT CANONS PARK

7.2.1 The soil layering at Canons Park has been established by means of three investigations;

(i) cone penetration tests carried out by Fugro (Ltd.) under a contract with the BRS;

(ii) the sinking of two 10 metre deep, fully sampled, boreholes by a BRS drilling team (under the supervision of the author);

(iii) the use of self boring pressuremeter tests, in a contract between P.M. Insitu and Imperial College.

Supplementary site investigation work has since been carried out by the BRS, but the results were not available at the time of writing. Workers from the BRS have also installed a number of soil instruments at the site including a group of Casagrande piezometers, and two spade-shaped total stress cells.

7.2.2 Continuous sampling of the boreholes allowed the geotechnical profile to be fully described to 10 metres depth. Figure 7.2 summarizes the interpreted log; this indicates between 1.0 and 1.75 metres of coarse gravel overlying a relatively structureless head layer which extends to around 2.5 metres depth. Beneath the head, there was found a 1.6 metre thickness of London clay which displayed the bedding

and fissure patterns typical of intact material, but also showed signs of moderate disturbance. The gleying of fissure surfaces and the disruption of fissure patterns all suggested movement by periglacial or other processes. The occurrence of highly polished horizontal shear surfaces at 4.3 and 4.4 metres, and the presence of rounded clean quartz gravel erratics just above the first surface, were noted in both borings. The combined evidence suggests that the disturbed layer has been subjected to mass movement at some stage, even though the ground surface is almost flat at the present time.

7.2.3 The samples taken below the disturbed layer showed a variable succession of intact marine clay, which could be divided into 'Brown' and 'Blue' London clay at a depth of 7.0 metres. The layers were frequently laminated, particularly between 4.5 and 6.5 metres. Claystone bands were located at depths of 5.7, 6.0, 8.4 and 10.0 metres. Photographs of each of the main soil layers are presented in Figures 7.3 to 7.5

7.2.4 Eight static cone tests were carried out by Fugro, and were reported in their document No. UO 662. The soundings were mostly sited within 25 metres of the pile test and borehole locations, and indicated similar cone resistance profiles at each position. Figure 7.6 shows the records for the deepest of the soundings, CP3, which included both friction sleeve and cone resistance measurements. The traces may be interpreted by reference to the charts of Schmertmann (1969) who classifies soils by the combination of cone resistance and friction ratio. On this basis, five main units may be identified; a surface layer from 0 to 1 metres, a sandy gravel between 1 and 1.75 metres, a stiff clay layer of constant strength from 1.75 to 4.3 metres, a very stiff inorganic insensitive clay sequence between 4.3 and 16 metres, and finally a dense clayey silty sand below 16 metres. The simplified borehole log is shown on Figure 7.6 for comparison, and good agreement is found. Assuming a cone factor, Nk, of 16, the undrained shear strength of the Head and Disturbed London Clay is calculated as approximately 90 kpa, and the Intact London clay layers show Cu between 125 and 170 kpa. The laminae and claystones of the intact sequence are evident in the rapid fluctuations of the cone and

friction sleeve traces. The change in strata at 16 metres depth probably represents the upper surface of the Woolwich and Reading beds, and evidence can be seen of a transition layer between 18.5 and 18 metres depth. Overall, the London clay thickness is 10 to 15 metres less than that interpolated from the geological map.

7.2.5 A similar correlation between insitu test data and geology can be made with the profiles of Cu and G, interpreted from the Self Boring Pressuremeter curves. The plots are compared in Figure 7.7 and the stratification is again apparent. The bands of sand and claystone between 5 and 6.5 metres depth provide a particularly stiff response to pressuremeter inflation. The scatter in the measurements is considerable, but the mean ratio of G/Cu falls around 150; for an isotropic linear elastic material this implies Eu/Cu = 450. Inspection of the recorded inflation-deflation pressure curves, given in Appendix 4, shows that the soil response was in fact highly non-linear with stiffness reducing rapidly with pressure increment size. These features are further discussed in Chapter 8.

7.2.6 Laboratory tests carried out at Imperial College provided the profiles of water content, Atterberg limits and bulk density plotted on Figure 7.8 and listed in Table 7.1. These index tests also confirm the division of the soil layers. The intact London clay shows distinctly lower liquid limits than the head and disturbed material, although the plastic limits are comparable; the change from disturbed to intact London clay at 4.1 metres is marked by a sharp reduction in P.I. The water content profile gently dips from around 30% at the surface to 25.5% at 10 metres, and in each case plots close to the plastic limit; bulk densities in the upper layers fall around 19.2 kN/m^3, but rise to 19.66 ± 0.35 in the intact London clay. The variations in γ bulk reflect changes in the frequency and thickness of sand laminae. Grading curves were determined at four depths and these are reproduced on Figure 7.9, plots a to d. The curves are similar, although there is a slightly higher clay fraction in the upper layers.

7.2.7 The laboratory and insitu test data are seen to be both consistent and compatible with the above described geological

descriptions. It is therefore proposed to divide the clay layers into four London clay groups; Head, Disturbed, Brown and Blue.

7.2.8 As with the Magnus profile, it was not feasible to faithfully model each of the layers present with a single reconstituted soil. The composite mix used for the slurry preparation produced the grading given in Figure 7.9 (e), and Atterberg limits as follows; LL 62.3%, PL 24.3% and PI 38.0%. The grading curve most closely resembles the trend from the upper layers, but the Atterberg limits fall nearer to the results of the intact London clay.

7.3 OEDOMETER TESTS ON INTACT AND RECONSTITUTED LONDON CLAY

7.3.1 Three oedometer tests were carried out on 76 mm diameter samples of intact and reconstituted soil. The first aim of the experiments was to investigate the stress history of the intact soil layers, the second was to determine the voids-ratio, pressure curves and consolidation coefficients of both natural and reconstituted material.

7.3.2 In order to clarify the definition of the preconsolidation pressures, a special procedure was adopted for the early stages of the intact tests. The samples were placed in the oedometer with the bath dry, and an initial load of 300 Kpa applied. Settlement was plotted against log time, and primary consolidation was typically complete within a few minutes. The water bath was then filled, and no swelling was observed. A succession of small vertical stress increments was then applied, with extra loads added when the first cycle of secondary consolidation was complete. The variations in t_{100} and c_α were used to assess the stage by which the preconsolidation pressures had been exceeded, and from this point larger load increments were employed and 24 hour periods used for consolidation. Figures 7.10 and 7.11 illustrate the e-log σ'_v curves from experiments LOED1 and LOED2. These tests cover compression to 3,000 Kpa, and swelling back to 200 Kpa. On loading, the two plots showed relatively sharp yields before joining practically log-linear virgin consolidation lines. Casagrande constructions were employed to estimate the preconsolidation

pressures, with respective values of 630 and 540 Kpa being found. Deducting the estimated insitu vertical stress, both σ'_{vc} values indicate maximum vertical stress conditions in the intact London clay which are around 500 Kpa greater than present. This excess pressure is approximately equivalent to a 55 metre high surcharge under submerged conditions. It will be recalled that the geological section inferred a surcharge of 60 metres at this site, and fair agreement is thus found.

7.3.3 The swelling curves for tests LOED1 and LOED2 initially show rapidly changing gradients, although the trend is less marked than that shown by the Magnus oedometer data. A similar tendency is apparent in the e-log σ'_v plot found from the compression phase of a test carried out from slurry with reconstituted soil. Figure 7.14 shows the curves for the slurry test LOED 3, and typical data from the K_o-consolidation and swelling stage of a stress-path triaxial test from the reconstituted clay series is given for comparison. As was noted for the Magnus clay, the e-log σ'_v curve for virgin consolidation shows a reducing slope with increasing stress, and close correspondence is found between oedometer and K_o triaxial data.

7.3.4 A summary of the one-dimensional characteristics is provided on Figure 7.15, where the intact and reconstituted data are plotted to the same scale. Also shown is an oedometer test on a sample of London clay taken from 20.6 metres at Bell Common; only slight differences in the value of e_1 separate the four curves at high stresses. Table 7.2 gives constants for the virgin consolidation lines over appropriate stress ranges, and the reconstituted mix shows a somewhat higher compressibility. This is partly due to the differences in considered stress range, but insitu processes such as bonding and ageing appear to have reduced the slope of the virgin consolidation line by around 50%. The initial slopes of the swelling lines are, however, comparable for all the London clay samples.

7.3.5 The variations with σ'_v of the coefficients of compressibility, consolidation, permeability and secondary consolidation for intact and reconstituted soil are plotted on Figures 7.15 to 7.18. The com-

pressibility curves of Figure 7.15 show the familiar trend of m_v rapidly reducing with σ'_v. Although the scale for the intact sample is 25 times that for the soil consolidated from slurry, the two experiments are tending towards similar m_v values at high stresses. The swelling response of both samples was initially stiff, but became progressively softer with increasing OCR.

7.3.6 The sensitivity of the consolidation parameters, c_v and c_{vs}, to stress level and OCR is demonstrated on Figure 7.16. The intact sample showed a steep fall in c_v as preconsolidation was exceeded, and gave a virgin compression c_v value of between 0.5 and 0.25 m^2/yr. On swelling, c_{vs} exceeded 2 m^2/yr initially, but rapidly reduced with σ'_v and gave values below 0.5 m^2/yr at OCR's greater than 3. The reconstituted soil indicated an almost constant c_v value of 0.2 m^2/yr in compression, but showed rapid swelling on unloading. Again, c_{vs} reduced as OCR increased and fell below 0.5 m^2/yr at OCR $\simeq$ 3.

7.3.7 Permeability data can be deduced from the measurements of c_v and m_v, and the variations of k with σ'_v are indicated in Figure 7.17. It is interesting that the curves for intact and reconstituted soil practically overlap at the higher stress levels. It is apparent that the permeability of the insitu soil is sensitive to consolidation pressure, and that an irreversible reduction from around 2×10^{-11} m/s to 5×10^{-12} m/s takes place as preconsolidation is exceeded.

7.3.8 Finally, Figure 7.18 plots the secondary consolidation coefficients for the same two tests. Increasing stress has little effect for the intact material, and c_α does not approach the reconstituted value after exceeding σ'_{vc}. On unloading, both samples showed similarly low creep rates, but these accelerated with increasing OCR.

7.3.9 In summary, at comparable stress levels and overconsolidation ratios, intact and reconstituted soil share many common characteristics in one dimensional compression. The principal differences are shown in the coefficients c_c and c_α for normally consolidated soil at high stresses; these features could be due to the effects of ageing, bonding or macrofabric.

7.4 INITIAL STRESS CONDITIONS

7.4.1 The importance of knowing the initial ground stresses before embarking on an effective stress analysis of pile behaviour was emphasized in Chapter 3. At Canons Park the vertical stress distribution can be obtained by reference to the bulk unit weights, and the self boring pressuremeter tests provide estimates for horizontal ground stresses. A number of Casagrande piezometers have been installed at different depths near the borehole locations, and these allow the pore water conditions to be monitored. Finally, the oedometer data and geological information permit the calculation of overconsolidation ratios for the intact and disturbed layers.

7.4.2 The variation of σ'_v and pore pressure with depth are shown in Figure 7.19. The piezometers indicate a clear pattern of under drainage from the London clay to the lower, more permeable, strata. Indeed, the two instruments positioned below 15 metres were usually dry. The pore pressures near the surface fluctuate seasonally; the plotted data refer to summer conditions, which prevailed at the time of sampling and pile installation.

7.4.3 A principal aim of the pressuremeter field work was to obtain profiles of the insitu horizontal stress, but estimating σ_h from the inflation stage of a self boring pressuremeter test is not straight forward. The tendency of the instrument to rotate during the early stages of a test, the very high initial stiffness of the soil to loading, and electrical instability make the process difficult. Figure 7.20 shows the average cavity strain-pressure curve found from the movement of the three camkometer sensing arms during the initial portion of a typical test carried out at Canons Park and the value of σ_h interpolated by Dr. B. Clarke is also shown. The data are plotted as σ'_h against depth on Figure 7.19 and a smooth line has been drawn through the considerable scatter. Also shown are the two results obtained by Tedd and Charles (1981) using spade shaped lateral stress cells; the latter tests are known to overestimate insitu σ'_h.

7.4.4 The insitu stress data are re-expressed using the ratio $K_o = \sigma'h/\sigma'_v$ on Figure 7.21. Also shown is the trend drawn through the scatter of the camkometer data reported by Windle and Wroth (1977), for a London clay profile at Hendon. Part of the discrepancy between the two pressuremeter profiles may be due to the assumptions made by Windle and Wroth concerning vertical stresses and pore water pressures.

7.4.5 The oedometer tests indicated a previous surcharge of around 500 Kpa for the intact and disturbed soil layers. Figure 7.22(a) plots the relationship between OCR and depth predicted assuming this stress history, which was used to derive the theoretical K_o - depth curve given in Figure 7.21 by substituting $\phi' = 23.5°$, into the equation;

$$K_o = (1 - \sin\phi')OCR^{\sin\phi'} \qquad \text{Eq. 7.1}$$

The theoretical line plots far below the pressuremeter smoothed profile, perhaps showing that aged soils maintain relatively higher horizontal stresses on unloading than the young clays discussed by Mayne and Kulhawy (1982).

7.5 TRIAXIAL TESTS ON INTACT SAMPLES

7.5.1 The Canons Park triaxial experiments are divided into four groups, as set out in Table 7.3, and this section discusses the first, LI. The series comprises 14 unconsolidated undrained tests carried out on 100 mm diameter intact specimens. Over the depth considered, the density of testing amounted to one per 600 mm; this allows the consistency of the results to be assessed and permits the rapid changes in soil layering to be followed. The experiments were all provided with lubricated ends, a mid-height pore pressure probe and electrolevel gauges; these ensure accuracy in the recordings of the stress paths, small strain stiffnesses and post peak behaviour.

7.5.2 The fourteen tests are divided into the four geological units described in Section 7.2; for convenience the Blue London clay layer is subdivided at the 8.9 metre level. The data will be reported in a similar way to the Magnus experiments, firstly the initial isotropic

test conditions will be discussed, then the recorded stress paths, the stress-strain curves and finally the stiffness-strain characteristics. In the subsequent sections summaries and correlations of the interpreted results will be presented.

7.5.3 The saturation procedure described in Chapter 5 involved subjecting the specimens to all round pressures in excess of their insitu total stresses. Equilibrium mean effective stresses were thus established before testing commenced. The steady values of p'_o are likely to have exceeded the samples' insitu mean effective stresses, as a result of both the irreversible processes of sampling, and de-saturation on partings and laminae of sand. The pre-test values of p'_o are listed in Table 7.4; at 2 metres the mean falls near 140 kpa, but p'_o increases with depth to give around 230 Kpa at 9 metres depth. The values of p'_o may be compared with the mean effective stresses calculated from the profiles of σ'_v and σ'_h reported in Section 7.4, and the data are plotted in Figure 7.22b; the retrieved samples show mean initial effective stresses that are 40 to 110 Kpa higher than those estimated from the estimated insitu stresses.

7.5.4 The stress paths of the fourteen intact tests are reproduced in Figure 7.23 a) to e), and strains are indicated at appropriate intervals on each of the plots. Regarding these data, four points are of interest;

(i) the stress paths are initially vertical, or slightly directed to the left. After yield they travel horizontally, then turn to strain soften on vertical or oblique stress paths.

(ii) The stress paths of the first two groups show clear differences to those of the Brown and Blue London clay specimens. Firstly, they demonstrate a more prolonged migration to the right after yield, and secondly there is a less pronounced loss of strength with strain after the attainment of peak conditions.

(iii) Each of the plots shows a considerable increment of deviator stress prior to the development of 0.1% axial strain.

(iv) The post peak behaviour of the deeper samples involves rapid reductions in strength with residual conditions being approached at 10 to 15% axial strain; well developed mechanisms with polished shear surfaces were seen in the specimens after testing. It was also noted that the lubricated ends allowed lateral translation of moving blocks of soil near the sample ends, and reduced any tendency for the samples to bulge at large strains.

7.5.5 Stress-log strain curves are plotted for the same five groups of tests in Figure 7.24 a) to e). The results within each sub-division are generally consistent; stiffness and strength increase with depth for the layers of Head, Disturbed and Intact London clay, although the progressions within the Blue and Brown London clay are less distinct.

7.5.6 The intial response of the samples to undrained compression can be gauged by reference to Table 7.5, and Figure 7.25 a) to e). The normalised scale of stiffness $(E_u/p'_o)_{0.01}$, and the index of non-linearity L, show only minor variations with depth; the upper two layers tend to give lower values of $(E_u/p'_o)_{0.01}$ and higher values of L. The relationships between secant modulus and strain show the same general features as the Magnus soils, with continuous reductions in stiffnesses from high initial values. A quantitative comparison with the intact Magnus test series, MI, indicates the London clay intact samples to behave in a slightly more linear fashion and to exhibit some what smaller ratios of $E_{u(0.01)}$ to p'_o.

7.5.7 Figure 7.26 plots the undrained shear strengths and initial effective stresses for Series LI against depth. The Cu plot clearly traces out the junction between the upper soil layers and the Intact London clay, with little variation above and below the 4.3 metre

level. It is interesting to note that Cu is invariably close to 0.5 p'_o, and that the profiles of initial effective stress and undrained shear strength are almost congruent. It has been shown that the recorded values of p'_o probably exceed the insitu values, and as Cu and p'_o are so closely related, this implies that the recorded Cu profile overestimates the insitu triaxial compression strengths. However, the data are consistent, and agree reasonably with the self boring pressuremeter data. The latter strengths generally fall 5 to 30 Kpa below the triaxial trend in the upper layers, and plot slightly above the line in the Intact London clay. It is also worth recalling that the Fugro cone resistances imply Cu values 10 to 20 Kpa greater than the triaxial tests over the full 10 metre depth.

7.5.8 The observation of brittle fracture and residual fabric in the intact samples suggests that little would be gained from analysing the peak or residual strength data in the terms of critical state soil mechanics. Indeed, the finding that the water contents tend towards the plastic limit at all depths implies a sensibly constant critical state undrained shear strength profile. Nevertheless, the peak values of Cu show a consistent pattern when plotted against water content, and this is illustrated in Figure 7.27.

7.5.9 The conditions at failure are further explored in Figure 7.28, which summarizes the effective stress path co-ordinates for peak conditions in all the tests. The range of $(\sigma'_A + \sigma'_r)/2$ is narrow, but two plausible effective stress envelopes have been assessed by eye. Recalling that the samples originate from a rapidly varying geological sequence, the deviations from the lines are smaller than might be anticipated.

7.5.10 Stiffness measurements made with the pressuremeter and triaxial tests are compared in Figure 7.29. The profile of $E_{u(o.o1)}$ is sharply divided at the junction with the Intact London clay, but there is little trend for the modulus to vary with depth within the two main units, and the overall scatter is small. The interpreted pressuremeter data, plotted as $3G \triangleq E_u$, fall significantly below the $E_{u(o.o1)}$ line for the LI series, and show a greater sensitivity to depth and soil fabric.

7.6 SUMMARY OF CONDITIONS AT CANONS PARK

7.6.1 The investigations at Canons Park show the geology to consist of a variable sequence of Quaternary soils overlying a London clay succession that extends to around 17 metres depth. The uppermost 1.5 to 2.0 metres include a dense gravel layer which could dominate the response to load of the proposed test piles, unless prebored and cased holes are provided. Between the gravel and the Woolwich and Reading Beds the clay layers divide into two main units. The first consists of the Head and Disturbed London clay, the second comprises the Brown and Blue intact London clay. The stronger Intact soils are more variable, with frequent laminations of sand and occasional layers of claystone. The junction of the two main units was located between 4.1 and 4.3 metres, and this division is supported by visual inspection, the profiles of insitu test measurements, the index tests and the unconsolidated undrained triaxial experiments. The recorded pore water pressures show near hydrostatic conditions between 1 and 5 metres, with more pronounced underdrainage at greater depths.

7.6.2 The properties of the clay layers have been investigated by the combination of laboratory and insitu testing techniques. Consistent data have been obtained regarding the profiles of index properties, undrained shear strength and stiffness at small strains. The triaxial tests showed two important features of the behaviour of the soils present;

(i) Initially very stiff and non-linear behaviour before yield.

(ii) Brittle failure with the clear development of residual fabric after peak conditions.

7.6.3 Compared with the Magnus soils, the clays showed significantly higher compressibility and lower initial undrained stiffness. The undrained triaxial tests indicated a slightly less steep non-linearity than that noted for the unconsolidated intact Magnus samples, and there was a much greater reduction in strength between peak and residual

conditions. The values of ϕ' mobilised at peak conditions were also smaller than those noted with the Magnus soils.

7.6.4 Oedometer tests have been used to examine the dependence of one dimensional consolidation rates, compressibility and permeability on stress level and OCR. The results show that for purely vertical drainage, equally slow rates of consolidation and similar compressibilities can be expected for consolidation and swelling from overconsolidated conditions. The experiments also showed that the insitu layers have probably been surcharged by around 500 Kpa within the span of their geological history.

7.6.5. The laboratory and field measurements were used to derive profiles of σ'_v and σ'_h for the clay soils. The resulting variation of K_o with depth is compatible with the overconsolidated conditions, with $\sigma'_{ho}/\sigma'_{vo}$ ranging between 1.5 and 2.3 over the depth of interest.

7.6.6 It was noted in chapters 5 and 6 that triaxial tests on unconsolidated retrieved samples could not be expected to completely reproduce the behaviour of insitu soil in triaxial compression. Similarly, the series could not provide information regarding the response to other undrained stress paths, or the effects of consolidation to less overconsolidated conditions. In order to supplement the intact test programme, and to extend the range of resedimented soils for which detailed studies have been made of stress-strain and strength characteristics, a programme of triaxial tests was also carried out on reconstituted London clay.

7.7 TRIAXIAL TESTS ON RECONSTITUTED CLAY; UNDRAINED TESTS FROM K_o CONDITIONS

7.7.1 Blocks of reconstituted London clay were prepared from slurry using a large oedometer. After sampling, 38 x 76 mm specimens were further consolidated in stress path cells and taken to K_o conditions for a range of OCR's, before being sheared undrained in either compression or extension. The procedures are fully described in Chapter 5, and Table 7.6 lists the test conditions for the compression series LR, and the extension series LRE.

7.7.2 The results will first be discussed by reference to the undrained stress paths given in Figure 7.30, and by the summary data given in Table 7.7. Considering the family of stress paths, the following points are of interest;

(i) The initial portions of all the paths, except LR 1.5, are nearly vertical. In compression there is a slight tendency for the direction to swing from right to left with increasing OCR, for extension there is the opposite trend.

(ii) Excepting LR1.5 and LRE3, each of the tests showed a change in stress path direction at the attainment of 0.1 to 0.2% axial strain. The deviation was to the left for OCR's 1 and 1.5, and to the right for the more overconsolidated samples.

(iii) Brittle failures developed at strains of 0.2 and 0.4% for the compression tests LR1 and LR1.5. The compression tests at higher OCR's, and the entire extension test series, showed strain hardening up to a peak strength condition, which was developed between 6 and 15% strain. The undrained shearing of LRE 3 was halted at 3% strain, and Fourie used the sample for a special drained "passive stress relief" test.

(iv) The tests show only slight signs of ϕ' reducing with strain after peak conditions; the tendency for residual fabric to form is far less marked than that observed with intact material. The ultimate states for all stress paths fall close to the Mohr-Coulomb line $c' = 0$, $\phi' = 22.5°$, with a typical scatter of $\pm$ 4 Kpa. At peak conditions the compression tests from OCR's 3 and 7 plot 10 to 12 Kpa above this line; it will be recalled that the intact tests indicated $c' = 15$ Kpa for $\phi' = 22.5°$.

(v) A consistent family of contours can be drawn through the axial strains indicated on the stress paths. The regions enclosed by the 0.01 and 0.1% lines are significant, but less extensive than those plotted for the Magnus clay on Figure 6.50.

7.7.3 The undrained brittleness observed in the compression tests is further explored in Figure 7.31a), where ratios of peak to ultimate strength are plotted against overconsolidation ratio. The comparable Magnus test data from series MR are plotted on Figure 6.18 and show values of Cu_{peak}/Cu_{ult} which are typically 10% higher than the corresponding London clay results. The small differences between the ultimate strengths for the series LR tests, suggests that there is little value in plotting Cu_{ult} against water content. Reference to Table 7.7 shows a clear anisotropy of peak strengths for the London clay, with respective ratios between compression and extension tests of 1.3, 1.1 and 1.6 for OCR's 1, 1.5 and 7. Slightly higher values of C_{ult} were found in extension than compression for OCR 1 and 1.5, but test LR7 showed an ultimate strength 70% greater than the comparable extension test.

7.7.4 Derived curves of Cu_{peak}/σ'_{vo} are plotted against OCR in Figure 7.31 b), and show lower ratios than the equivalent Magnus test series. It is intriguing to use the curves to predict a shear strength profile for Canons Park, by combining these data with the previously described profiles of σ'_{vo} and OCR. Values are plotted on Figure 7.26 for 3 depths and show a surprising agreement with the intact triaxial tests.

7.7.5 The stress-strain relationships for the two reconstituted series are plotted on Figures 7.31 and 7.32, using semilogarithmic axes. The compression tests show a family of smooth curves which reach peak deviator at strain levels that systematically increase with OCR, and the electrolevel instruments allow credible measurements at axial strains as low as 0.002%. Inspection of Figure 7.32 shows that the behaviour in extension also follows an orderly pattern, although the strains required to mobilise peak strength generally exceed the limit of 10% which was selected for the stress-strain plots.

7.7.6 In the analysis of the stiffness-strain relationships deduced from the two curves, it is convenient to separate the scales of stiffness and linearity in the way described in Chapter 6. Figures 7.34 and 7.35 plot the variations of L(ε) with axial strain for the two test series, and the similarities are striking; the mean value of L in compression tests is 0.41, with 0.38 for extension. Although the ranges of results are close, there is a tendency for the compression curves to become more linear with increasing OCR, whilst in extension the converse trend is found.

7.7.7 The scales of stiffness may be assessed by reference to Table 7.7 and the curves of Figures 7.36 and 7.37, where normalised measures of $E_{u(0.01)}$ are plotted against OCR. With the exception of normally consolidated soil, the data show the compression stiffness for each particular OCR to be larger than that found in extension. The plots also demonstrate the sensitivity of E_u/σ'_v to OCR; in compression, lightly overconsolidated conditions give the highest stiffness parameters, with the exception of $(E_u/\sigma'_v)_{0.01}$. For extension tests, the highest values of $Eu_{(0.01)}$ and $(E_u/p'_o)_{0.01}$ are found with OCR 1.

7.7.8 It is useful to compare the results from the two series with the intact London clay tests, and the equivalent experiments on reconstituted Magnus soil. The Intact series LI represented soil at OCR's between 12 and 4, and gave mean values for $(E_u/p'_o)_{0.01}$ and L of 602 and 0.385 respectively. The corresponding figures for tests LR7 and LR3 indicate initial stiffnesses which are 30 to 40% higher, and values of L between 0.42 and 0.48. The stress-strain characteristics up to 0.1% strain are therefore comparable, if slightly different in detail. However, the correspondence becomes less close in the strain range between 0.1% and failure.

7.7.9 A similar comparison with the Magnus data shows three significant features with regard to the small strain characteristics;

(i) The London clay shows slightly more linear behaviour, with values of L which are typically larger by 0.05 to 0.15.

(ii) The Magnus clay gives higher normalised compression stiffnesses with $(^{Eu}/p'_o)_{0.01}$ exceeding the London clay results by 130 to 160% in similar tests.

(iii) The ratios of extension to compression stiffness show different variations with OCR, although broadly similar trends are found, and these are illustrated in Figure 7.38.

7.8 TRIAXIAL TESTS ON UNCONSOLIDATED SAMPLES FROM THE RECONSTITUTED BLOCKS

7.8.1 The shearing phases of series LR and LRE were carried out by applying the 'standard' rate of external displacement, which is equivalent to 4.5% axial strain per day. As in the Magnus programme, it was desirable to assess the sensitivity of stiffness to strain rate, but time constraints made a full study with K_o consolidated samples impractical. Nevertheless, a useful series could be undertaken using unconsolidated samples of reconstituted clay.

7.8.2 The test descriptions and initial conditions for series LRUU are set out in Table 7.8. Four identical samples were cut from a block prepared in the large oedometer, and were brought to controlled initial mean effective stresses of 90 Kpa under isotropic conditions. Three samples were sheared in undrained compression at different rates, the fourth was compressed in a drained fashion. In each case, fixed sample ends were employed.

7.8.3 The stress paths for the undrained tests are shown on Figure 7.39, and the stress strain curves on Figure 7.40. A complete test to ultimate conditions was carried out with a single sample, and the stress path shows many common features with the intact London clay experiments. In order to save time, tests LRUU 1 and LRUU 3 were not taken beyond 0.5% axial strain. Table 7.8 summarizes the small strain stiffness characteristics determined in the four experiments, and the following points are noteworthy;

(i) The mean ratio for $(^{Eu}/p'_o)_{0.01}$ was 611, ± 12%. This value falls 40% below the ratio interpolated from the K_o compression tests at the same OCR.

(ii) There is an inconclusive trend for $(^{Eu}/p'_o)_{0.01}$ to increase with strain rate, but the data are generally comparable with a 10% gain in stiffness for a tenfold change in $t_{0.01}$.

(iii) The range in the index L shows no particular tendency, but the mean value of 0.425 is close to that found in series LR for lightly over-consolidated conditions.

(iv) The scatter in the undrained tests probably reflects differences in the samples' stress histories, which might be due to non-homogeneities in the oedometer blocks, or variations in sampling and setting up disturbance.

(v) The single drained test indicated a lower than typical L value.

(vi) At 0.01% strain the drained stiffness ratio E'/p'_o, was around 30% less than the mean $^{Eu}/p'_o$. If allowance is made for the relatively low strain rate in test LRUD1, the difference between $E_{u(0.01)}$ and $E'_{(0.01)}$ becomes smaller.

7.8.4 The choice of unconsolidated conditions for the rate study makes the interpretation of the results inexact, but the preliminary data suggest that strain rate variations and the provision of drainage did not greatly modify the small strain characteristics of the reconstituted London clay.

7.9 RESONANT COLUMN TESTS

7.9.1 Resonant column tests on Magnus clay were reported in Chapter 6 and found to be a useful way of confirming the behaviour observed at small strains in triaxial tests. Fugro Ltd., who carried out the Magnus tests with a Stokoe apparatus, have recently replaced their test equipment with the Hardin design produced by Soil Dynamics Instruments Inc. of Lexington, Kentucky. The directors of the company kindly offered to carry out further work with the new equipment and this section describes the results of two tests on K_o consolidated reconstituted London clay.

7.9.2 Material from a cake of London clay prepared in the large oedometer was trimmed by Fugro to provide cylindrical samples 38 mm in diameter and 76 mm high. These were consolidated in steps[1] to follow the same consolidation stress paths as the LR and LRE series. The initial conditions and summary of results for the two experiments LRC1 and LRC2, are given in Table 7.9; the stiffness-strain and damping ratio-strain characteristics are presented in Figure 7.41.

7.9.3 Regarding these data, the results can be summarized as follows;

(i) Both tests show almost constant stiffness up to 0.01% 'typical' shear strain, thereafter modulus decreases steeply with strain.

(ii) This behaviour is reflected in the damping ratio characteristics which show damping to rapidly climb after attaining 0.01% shear strain. For smaller strain amplitudes the ratio is finite, but sensibly constant.

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1 Four steps were taken in consolidating both samples to σ'_v = 400 Kpa, and a further 3 were required for the swelling portion of LRC2.

(iii) When values of $3G/p'_o$ are compared, at 0.01% strain[1], with the K_o consolidated triaxial test data in Table 7.7, the resonant column results fall slightly below the compression test stiffness indices. The shortfalls amount to 3% for OCR1 and around 38% for OCR2.

(iv) The reduction of stiffness between 0.01 and 0.1% axial strain in the triaxial rests was more rapid than that seen over the equivalent strain range in the resonant column tests.

(v) The results of the London clay experiments generally reinforce the conclusions drawn in Section 6.16 from the Magnus resonant column series.

7.10 RING SHEAR EXPERIMENTS WITH LONDON CLAY

7.10.1 The possible development of residual fabric at, or near, the interface between pile and soil is an important consideration in the work of this Thesis. Fortunately, ring shear tests have been carried out by Lupini (1981) and Lemos (1985), and their data allow the microfabric effects to be assessed for the London clay. Both workers used soil with the following index properties;

(i) clay fraction 58%

(ii) PL = 26%, L.L. = 71%, P.I. = 45%

The tested clays were almost identical to the intact layers at Canons Park.

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[1] The correspondence between resonant column 'shear strain' and axial strain in undrained triaxial tests was discussed in Section 6.16.

7.10.2 Lupini (1981) investigated the material's response in soil to soil tests, and Figures 7.42 and 7.43 illustrate the effects of rate of shearing and cumulative displacement in tests where σ'_n was fixed. Slow initial loading produces a peak ϕ' of around 24°, which reduces with displacement. When the shearing rate is increased, as in stage 2, the mobilised ϕ' rises sharply, but continuing rotation gives a steady reduction of τ/σ'_n with displacement. The consolidation pause, 3, and the reshearing stage 4, showed that only a minor portion of the reduction in ϕ'_r can be due to the accumulation of undissipated pore water pressure, and at the end of the fourth stage ϕ'_r reduced to 9.3°. Slowing the rate of displacement in stages 5 and 6 caused further slight changes in ϕ'_r, and for the lowest speed of 0.001 mm/minute an ultimate angle of 7.5° was indicated. The high velocity shear of the seventh phase generated a pronounced increase in effective strength, but this also reduced with displacement, from a 'peak' of 16° to a residual of 11°. Slow testing at the end of this period showed that a less aligned soil fabric had been produced by the fast shearing; a peak of 15.1° was found which gradually reduced to a final value of 9.4° after 30 to 40 millimetres displacement.

7.10.3 The pattern of behaviour is complex, but may be summarized as follows;

(i) Tested at a slow rate, ϕ' reduces from peak to residual conditions after displacements of around 50 mm.

(ii) Fast shearing from peak or residual conditions generates a larger strength, possibly due to viscous effects.

(iii) Continuous shearing at an intermediate rate eventually produces an ultimate angle similar to that for slow shear. However, displacements of up to 200 mm are required to effect this reduction.

(iv) Slow shear from the latter condition produces the lowest recorded strengths.

(v) With fast shear, the minimum strength is obtained after around 300 mm displacement. Subsequent slow shearing gives a noticeable peak, but residual conditions are recovered relatively quickly.

7.10.4 Lemos (1985) performed similar experiments with London clay shearing against a sand blasted mild steel interface, and a summary of his results is given in Figure 7.44. Slow initial shearing produces a small peak angle of 17.7°, but residual conditions are developed after a relatively small displacement. Increasing the rate of shear gives a pronouced 'viscous' increase in ϕ'_r, which decays only after a further travel of 300 to 500 mm. The final value is, however, comparable to that produced by slow shear. Lemos further notes that the principal shear surface in the ring shear tests forms within the soil mass, and not at the interface.

7.10.5 When compared with the Magnus data discussed in Section 3.3 the following differences in behaviour are clear;

(i) The London clay shows its lowest strength in soil-soil shear. With the Magnus clay, only interface tests produce a well developed residual fabric.

(ii) The effects of rate are more pronounced in the London clay.

(iii) The residual strengths after large displacements are far smaller with the London clay.

7.10.6 It is possible to draw conclusions from these data concerning the likely effects of installation for a displacement pile in London clay. Firstly, the rates of shear are likely to be relatively fast, even for jacked piles. A principal displacement surface, with a less

oriented fabric than the slow residual, will probably be formed in the soil near to the pile shaft. Near the pile tip the relative displacements on this surface will not be large, and the fabric incompletely aligned; further up on the shaft the ultimate conditions appropriate to the displacement rate may pertain. If displaced gradually in a pile test, some brittleness may be observed as slow residual conditions are developed throughout; the more viscous response to further rapid loading could allow a relatively high ϕ' to develop in fast tests, even where a polished surface has been formed during installation. Observations of reoriented London clay fabric close to a jacked pile have been made by Kitching (1983) and are discussed in Chapters 3 and 9.

7.11 SUMMARY

7.11.1 The site investigations at Canons Park have been reviewed in Chapter 7, and the results of the London clay research programme presented. Dutch cone soundings, self boring pressuremeter experiments, careful sample logging and an extensive suite of laboratory tests have been used to provide a comprehensive description of the soil conditions at the field pile testing site.

7.11.2 The laboratory data obtained reinforce many of the conclusions drawn from the Magnus studies. Concerning the shear strength properties, the more plastic reconstituted London clay displayed weaker, undrained, characteristics and a lower friction angle. In the highly structured intact form, there was a rapid transition from peak conditions to a residual state associated with polished shear surfaces.

7.11.3 The stress-strain characteristics of the London clay, in both intact and reconstituted samples, are generally comparable with those of the Magnus soils. Oedometer, triaxial, pressuremeter and resonant column tests all point to high initial stiffnesses and marked non-linearity. However, when compared quantitatively London clay samples show significantly softer and more linear behaviour than Magnus clay tested under the same conditions.

7.11.4 Differences between the two groups of soil are also evident in the results obtained from ring shear experiments. With London clay, soil to soil shearing produces a weak, polished, residual surface in a way that was not observed with the low plasticity clays. Conversely, in interface tests the London clay showed few signs of the special phenomena noted with Magnus clay or Lower Cromer Till.

CHAPTER 8

FIELD MEASUREMENTS OF SOIL STRESS-STRAIN BEHAVIOUR, AND THE USE OF NON-LINEAR CHARACTERISTICS

8.1 INTRODUCTION

8.1.1 The laboratory experiments described in Chapters 6 and 7 showed stress-strain characteristics which were both initially very stiff, and highly non-linear. These observations present difficulties when considering, for example, the load displacement characteristics of large pile groups, as most analyses assume the theory of linear elasticity to be applicable at working load levels. The questioning of this central assumption has many implications, and it is important to validate the laboratory findings through comparisons with insitu behaviour. Chapter 8 attempts such an exercise and consists of an introduction followed by three main sub-sections. Part 1 is concerned with an interpretation of some high quality field tests, and shows that characteristics similar to those measured in the laboratory can also be recovered from the field, provided that the experiments are carefully instrumented. Part 2 explores the engineering significance of the results in a series of parametric boundary value analyses of practical problems, and conclusions are made regarding the importance of the non-linear characteristics in predicting ground movements and stress changes. In Part 3, the load-displacement relationships expected from the analyses are compared with field data from trial footings, pile and pressuremeter tests. Good general agreement is found, and the analyses are seen to be helpful in understanding full-scale behaviour and insitu test data.

8.1.2 Before considering the instrumented field experiments reviewed in Part 1 it is useful to compare the unconsolidated triaxial test results for Canons Park with previously established field profiles of London clay stiffness. When contrasted with the average 'elastic' moduli determined from field data, laboratory tests frequently give

stiffnesses which are more than an order of magnitude smaller than the interpreted insitu values. In the past these large differences have been attributed to sample disturbance, e.g. Simpson et al. (1979), but the use of local measurements in the laboratory has shown that bedding, tilting and other experimental difficulties mask the true initial behaviour of soils in conventional testing.

8.1.3 Figure 8.1 reproduces the profile of undrained stiffness determined at Canons Park. For convenience, the E_u value at 0.01% strain is plotted, and it will be recalled that the secant modulus varies steeply with strain level above and below this reference point. The soil layering is indicated on the same figure, as are the profiles of $E_{u(0.01)}$ reported by Fourie (1984) for Bell Common and that deduced from the Canons Park pressuremeter results. For comparison, the distributions of E_u from camkometer and large plate tests carried out nearby at Hendon (Windle and Wroth (1977)) are plotted, and also shown is the distribution of E_u with depth obtained by elastic back analysis of the ground movements developed around an excavated retaining wall, after Cole and Burland (1972).

8.1.4 The plotted results show that $E_{u(0.01)}$ is substantially influenced by geological history, but does not increase rapidly with depth within the disturbed or undisturbed London clay. The values of $E_{u(0.01)}$ are considerably larger than the apparent values of E_u obtained from high quality insitu tests and field measurements of the settlements of structures; Butler (1975) noted that foundations on London clay gave settlements compatible with $E_u/C_u \simeq 400$ which is less than half the typical triaxial test ratio, $(E_u/C_u)_{0.01}$.

8.1.5 These results might be anticipated since the measured boundary displacements for surface loading or pressuremeter tests are strongly influenced by the more highly stressed, lower stiffness, regions close to the loading surfaces. Therefore, the back analysis of boundary displacements using linear elasticity will give stiffnesses which are generally lower than the maximum values, and specifically smaller than $E_{u(0.01)}$ determined in the triaxial apparatus. However, the profile of E_u deduced from the observation of ground movements lies

closer to the $E_{u(0.01)}$ profile. This is thought to result from the weaker influence of locally overstressed zones in such problems, Simpson et al. (1979).

8.1.6 It is of considerable interest to note that dynamic values of shear modulus, G, obtained by Abiss (1981) using geophysical methods give an equivalent Young's modulus profile which is only marginally stiffer than the trend of $E_{u(0.01)}$ determined from unconsolidated triaxial experiments.

PART 1. FIELD MEASUREMENTS OF STRESS-STRAIN BEHAVIOUR

8.2 INSTRUMENTED LARGE INSITU TESTS

8.2.1 From the introductory discussion, it is apparent that the back analysis of field behaviour from boundary displacements can lead to stiffnesses which are lower than the small strain laboratory values. Each problem will produce a different degree of strain concentration, and the field response of a given soil undergoing pile loading could infer a different modulus to that deduced from excavation monitoring, or the settlement of footings. The understanding of insitu stress-strain behaviour is therefore greatly facilitated when local strains can be correlated with local stress changes. In this section analyses of data from three such instrumented field tests on London clay are presented.

8.2.2 Marsland and Eason (1973) report the results of a plate loading test carried out in Chelsea, in which displacements of the ground were measured, at various depths, during loading at a constant rate of penetration of 2.5 mm per minute. The plate was 865 mm diameter and the test was carried out in the base of a 900 mm diameter 17.5 m deep shaft. Figure 8.4 a) illustrates the general arrangements of the tests, and shows how the experiments allowed the direct observation of average strains for three levels beneath the plate for various applied loads. The secant Young's modulus E_u corresponding to each value of strain can be evaluated using elastic theory to obtain the stress changes. At this depth the average laboratory undrained

strength C_u was 200 kN/m^2, and Figure 8.2a) shows the results for the three levels plotted as E_u/C_u vs log vertical strain. The Chelsea site is overlain by about 7 m of gravel and soft clay, and it is estimated that at a depth of 17.5 m, the value of K_o is about 1.5 giving p'_o = 240 KN/m^2. Figure 8.2b) shows the data replotted as E_u/p'_o vs log vertical strain.

8.2.3 An instrumented surface loading test has recently been carried out on London clay at Bracknell, Burland (1984). The test is illustrated in Figure 8.4 b) and consisted of loading a 3 m square area at ground level to a mean pressure of 110 Kpa, with settlements measured at depths of 0, 1, 2, 3 and 5 metres beneath the centre. The average strains between each level were evaluated three hours after loading and the corresponding values of E_u were calculated using elastic theory. In Figure 8.2a) the values of E_u/C_u for each level can be seen to agree with the results obtained from the deep plate test. At this site the London clay extends to the surface and a value of K_o = 2.5 has been assumed in order to obtain values of p'_o. In Figure 8.2b) the values of E_u/p'_o are plotted and close agreement is again obtained with the plate test. The high apparent value of E_u/p'_o for 0.5 m depth is thought to be due to surface desiccation.

8.2.4 The stress changes beneath the plate and footing centre lines are equivalent to those of an undrained triaxial compression test from K_o conditions. The stiffness strain plots given in Figure 8.2 can therefore be directly compared with the triaxial data discussed in Chapter 7, and summarized in Tables 7.4 and 7.7. In relation to the K_o consolidated tests the 'field' value of $(E_u/p'_o)_{0.01}$ is only marginally below the ratio expected, and the index L slightly higher; in summary the agreement is remarkable. Comparisons with the intact experiments shows the field values of $(E_u/p'_o)_{0.01}$ and L to exceed the laboratory means by 25 and 45 per cent respectively. The instrumentation in both field tests extended to relatively shallow depths, with maximum ratios for Z/D of 0.62 and 1.6 for the plate and footing. Had settlement points been installed at lower levels, data would be available for smaller strains, and the suggestion that the L values exceeded 0.5 could have been further investigated.

8.2.5 Field data from a completely different type of loading is given by Cooke, Price and Tarr (1979). Their experiments at Hendon included the measurement of vertical shear strains at three depths close to an instrumented jacked-in-place tubular steel pile, the arrangements of which are shown in Figure 8.4 c). Since the shear stresses transferred to the ground were accurately measured, the data can be used to determine directly the relationship between shear stress τ_{VH} and shear strain γ_{VH} for the London clay. In Figure 8.3a) the relationships between secant G_{VH} and log γ_{VH} are plotted for the three instrumented levels remote from the pile tip. The marked non-linearity of the stress-strain relationships is evident and is similar in form to both the laboratory results discussed in Chapter 7, and the field loading data discussed above.

8.2.6 In order to carry out more quantitative comparisons between the pile test data and the triaxial and other tests, certain assumptions have to be made. Firstly, the process of jacking the pile into the ground increases the mean effective stresses from p'_o to p'_i, and estimates have to be made for this change. The theories and experimental data reviewed in Chapter 3 suggest that p'_i after equilibration might be approximately calculated as 3.5 Cu_o. For the Hendon site, p'_o insitu is typically larger than Cu_o, and this implies an increase in mean effective stress of around 200% as a result of installation, which could be considered an upper bound. To compare the stiffness data with the triaxial results for E_u/p'_o, it only remains to substitute 3G for E_u, and calculate the equivalent ratio $3G/p'_i$.

8.2.7 It is also necessary to find an equivalence between the measured shear strains and the axial strains recorded in the triaxial tests. Assuming undrained plane strain gives $\varepsilon_1 = \gamma_{VH}/2$, $\varepsilon_2 = 0$ and $\varepsilon_3 = -\gamma_{VH}/2$. The comparisons can then be made either in terms of major principal strain, which gives $\varepsilon_A = \gamma_{VH}/2$, or at equivalent levels of the shear strain invariant E. The latter comparison gives $\varepsilon_A = 0.527\ \gamma_{VH}$, as $E = \sqrt{3}\ \varepsilon_A$ in the undrained triaxial test and $E = \gamma VH$ for undrained plane strain. For present purposes it is thus adequate to calculate equivalent ε_A values as $\gamma_{VH}/2$.

8.2.8 The traces given on Figure 8.3b) therefore give a lower bound to the equivalent triaxial plots of E_u/p'_o against ε_A. In the piling example the measured shear strains are smaller than those under the footing centre line, even though the measurements were made close to the pile shaft; indeed at failure the equivalent values of ε_A typically amount to 0.1%. The parameter L is therefore more clearly defined, and falls slightly below the triaxial range. Recalling that p'_i is probably overestimated in this case, the calculated $(^{3G}/p'_i)_{0.01}$ values are likely to underestimate $(E_u/p'_o)_{0.01}$. Nevertheless, the recorded ratio of 600 is close to the mean of the intact tests, and only slightly below the triaxial compression value for K_o consolidated soil.

8.2.9 In summary, the analysis of the field tests shows insitu stress-strain properties that are in close agreement with those measured the laboratory at small strains. Despite the possible effects of anisotropy, strain rate and macro-fabric, the triaxial compression data give accurate predictions for the bulk secant moduli as functions of strain in tests where ε_1 ranged from 0.005% to 0.7%. The field measurements also emphasize that the strain levels induced by piles and surface loadings are generally small, even in the zones of greatest stress change.

8.3 PRESSUREMETER DATA ANALYSED BY THE METHOD OF PALMER

8.3.1 Although it is generally not possible to determine elemental stress strain properties from field loading tests without resort to special instrumentation, a special case exists in the analysis of the undrained expansion of a cylindrical cavity, as presented by Palmer (1972). It was noted in Chapter 3 that the strain fields around a long expanding cylindrical cavity are dictated by kinematics alone, provided that the soil is incompressible. This feature allows the undrained stress strain relations for soil at the cavity boundary to be deduced from the pressure-volume change characteristics alone.

8.3.2 It is well known that the initial expansion of a pressuremeter produces a curve which is difficult to interpret, as a result of installation disturbance and other experimental difficulties, and soil parameters are most often deduced from unload-reload loops performed from an inflated state. With the Magnus PIP tests reported in Appendix 3, the unloading was made after the cavity volume had increased by 20%; this implies a pre-test major principal soil strain of 13.5% near the membrane boundary. The Self Boring Pressuremeter (SBP) tests conducted at Canons Park included an unload-reload cycle after ΔV had reached 4 to 6% of the uninflated volume, which implies a lower value of 2 to 3% for ε_1 in the soil close to the instrument.

8.3.3 The effect of these large strains on the soil stiffness in unloading is uncertain. However, it is worth recalling that the undrained pre-shearing of K_o consolidated Magnus soil to $\varepsilon_1 \triangleq 5\%$ in extension gave reloading values of $E_{u(o.o1)}$ which fell below the undisturbed compression moduli by 15 to 50%. Moreover, the pressuremeter expansion is accompanied by a degree of partial drainage, and the holding of a 20% volume strain in the PIP tests allows the soil to commence an 'infinitely stiff' unloading portion, through consolidation and creep, before the controlled deflation commences. These various considerations suggest that the inferred boundary stress-strain characteristics will not be of the greatest relevance, although it is clearly of interest to compare them with the triaxial data.

8.3.4 In addition to the summary plots given in Appendix 4, the SBP operators, PM insitu, made the full records of the Canons Park pressuremeter tests available to the author. From these data it was possible to carry out detailed analyses using both the Palmer (1972) method and the simplified treatment given by Gibson and Anderson (1961). To illustrate the application of the Palmer method, two typical tests will be discussed, and further analysis is presented using the simplified analysis in Section 8.12.

8.3.5. The selected pair of tests were carried out at 3.25 and 8.3 metres depth, and both included a first unloading from around 5% $^{\Delta V}/_{V}$, and a second unloading stage after full inflation. The Palmer analysis was applied to finite increments of pressure and cavity strain, and produced curves of $(\sigma_1 - \sigma_3)$ against ε_1 for unloading with plane strain conditions. By assuming σ_2 to equal $(\sigma_1 + \sigma_3)/2$, and $\varepsilon_v = \varepsilon_2 = 0$, the use of deviatoric strain and stress invariants allows these data to be converted to give a plot of 3G/Cu against equivalent axial strain for an undrained triaxial test.

8.3.6 Figure 8.5 a) shows four such curves, with a plot for each unloading stage, and three points are noteworthy;

(i) The resolution of the pressuremeter instruments is such that the smallest calculable equivalent strain is around 0.1%. In comparison with the triaxial measurements this is a large strain.

(ii) There are considerable reductions in stiffness between the first and second unloading stages; at strain levels equivalent to $\varepsilon_A = 0.4\%$ these range between 40 and 50%. It is not clear whether the stiffness changes were due to the monotonic expansion of the cavity to its full level, or the effect of the first unload-reload loop.

(iii) Each of the curves shows marked non-linearity: the first unloading data imply respective $(E_u/C_u)_{0.1}$ ratios of 200 and 300 for the tests at depths of 3.25 and 8.3 metres.

Calculating the same ratio from the intact triaxial test data reported in Table 7.4 gives corresponding ratios of 460 and 350, whilst the results for tests LR3 and LR7 listed in Table 7.7 give respective values of 620 and 460.

8.3.7 Figure 8.5 b) plots the results obtained by analysing the Magnus PIP tests in the same way. Generally similar results are obtained, although the unloading stage was performed after full inflation, with no earlier load-unload loop being made. The limiting resolution is equivalent to around 0.06% ε_A, and modulus is seen to fall steeply with strain. Regarding the scales of stiffness, the PIP tests indicate $350 > (E_u/C_u)_{0.1} > 150$, whilst the K_o consolidated triaxial compression series MR gave a range of 990 to 340, and the intact series, MI, indicated $780 > (E_u/C_u)_{0.1} > 180$; see Tables 6.2 and 6.4.

8.3.8 In summary, analysing the pressuremeter data by the Palmer method produces results of limited value. It is not possible to resolve small strains, and the stiffness data are sensitive to the disturbance that inevitably occurs close to the cavity boundary. Although the obtained data reinforce the description of the soil response as non-linear, the scales of stiffness fall below those estimated at comparable strain levels in triaxial tests. Given the uncertain influences of disturbance, rate and dependence on stress path direction, the inferred discrepancies are not surprising. Despite the elegance of Palmer's analysis, practical considerations hamper the method and it is not feasible to directly obtain the full insitu stress-strain characteristics of the ground from simple boundary measurements.

PART 2, STUDIES OF THE INFLUENCE OF NON-LINEAR STRESS-STRAIN CHARACTERISTICS IN BOUNDARY VALUE PROBLEMS

8.4 INTRODUCTION

8.4.1 The laboratory and field measurements of elemental soil behaviour described in the earlier sections consistently showed pre-yield characteristics which were both highly non-linear, and initially very stiff. The second part of this chapter describes how these undrained characteristics can be reproduced with a simple numerical model, which is suitable for both simplified calculations and incorporation in a non-linear finite element program. An empirical formulation is given for a material which is non-linear before failure

and also obeys a Tresca yield criterion. The formulation is general and alternative criteria can be used to describe the yielding and plastic flow.

8.4.2 After setting out the formulation, a simplified analysis is presented for the case of an axi-symmetric surface loading, which illustrates the use of the non-linear characteristics in boundary value analysis, and gives an impression of the results that can be expected. In the subsequent sections more rigorous finite element analyses are presented for a series of problems including vertically loaded footings, cavity expansion and axially loaded piles. In each case linear elastic solutions are compared with the complete load displacement curves predicted for a typical non-linear soil. As a result it has been possible to identify a number of important features which stem from the pre and post yield non-linearity. The data also allow the problems of linear elastic theory, and the evaluation of appropriate soil moduli, to be discussed and explanations are offered for differences in the stiffnesses which can be interpreted from laboratory data, insitu tests and full-scale behaviour.

8.4.3 The analyses show that in many practical problems the distribution of ground stresses and surface displacements are markedly influenced by non-linearity, both before and after yield. For each of the boundary value problems the same non-linear soil is employed, and the modelling is based on the characteristics of K_o consolidated Magnus reconstituted clay at OCR 2, as shown in the undrained compression test MR2.

8.4.4 The numerical studies are taken from a larger project which was carried out in conjunction with Dr. D. Potts and, at a later date, Dr. A. Fourie. The work is reported by Jardine et al. (1985) and included analyses of raft foundations and strutted excavations. Fourie (1984) extended the approach to model the Bell Common tunnel excavations, using the laboratory data described in Chapter 7 to obtain the parameters for the empirical stress-strain model. It is encouraging that excellent agreement was noted between the finite element predictions and field measurements of tunnel wall displacements, structural forces and surrounding ground strains.

8.5 EMPIRICAL UNDRAINED STRESS-STRAIN RELATIONSHIP

8.5.1 The data presented in Chapters 6 and 7 suggest that the general form of the relationship between secant E_u and the logarithm of axial strain is as shown in Figure 8.6. The relationship before yield can be conveniently represented by a periodic logarithmic function :

$$\frac{E_u}{C_u} = A + B \cos \alpha \left(\log_{10} \frac{\varepsilon_a}{C}\right)^{\gamma} \qquad \text{Eq. 8.1}$$

The empirical constants A, B, C, α and γ are determined from the test data as described in Appendix 5. Equation 8.1 only holds for a specified range of strain values. For strains below a lower limit, ε_{min} and above an upper limit ε_{max}, fixed tangent stiffnesses are assumed. Over this 'elastic' range a Poisson's ratio of 0.49 is specified, and, if yield is to be modelled, a suitable criterion and flow rule must be included. Care is required to ensure compatibility between ε_{max} and the onset of plastic yield.

8.5.2 In Figure 8.8, Equation 8.1 is fitted to a selection of the stiffness test data given by the low plasticity clay. The values of the associated empirical constants are given in Table 8.1. It can be seen that the formulation is appropriate for soil behaviour before yield.

8.5.3 The majority of numerical procedures make use of the tangent modulus E_{ut} rather than the secant modulus. Differentiating and rearranging Equation 8.1 gives :

$$\frac{E_{ut}}{C_u} = A + B.\cos\alpha I^{\gamma} - \frac{B.\alpha\gamma I.(\gamma-1)\ \sin\alpha I\gamma}{2.303} \qquad \text{Eq. 8.2}$$

where $I = \log_{10} (\varepsilon_a/C)$

Equation 8.2 can be generalised by substituting the deviatoric strain invariant E for $\sqrt{3}\varepsilon_a$ and this allows Equation 8.2 to be incorporated into non-linear elastic finite element computer programs. For the purposes of the analyses described here the above empirical non-linear elastic formulation has been combined with a perfectly plastic (non-hardening) Tresca failure criterion and plastic potential. The undrained stress-strain relationship chosen for this study is referred to as LPC2 and corresponds to the one obtained for test MR2 given in Figures 6.20 to 6.22 (see Table 8.1 for the appropriate empirical constants).

8.5.4 Figure 8.7 shows a comparison of the source data from test MR2 and the finite element simulation using material LPC2. The agreement is excellent over almost four log cycles of strain. This particular test was chosen for the study as its post yield behaviour most closely approximated to the perfectly plastic Tresca yield criterion. The stiffnesses from test MR2 at 0.01% strain can be summarized as giving E_u/C_u equal to 3320 and E_u/p'_o equal to 2270, with a linearity index L = 0.35. In comparison with the other Magnus K_o compression tests MR2 is stiffer than the average, but is not particularly non-linear.

8.6 SIMPLE ILLUSTRATIVE PROBLEM

8.6.1 In this section some aspects of the influence of non- linear stress-strain behaviour will be illustrated by means of a simplified problem. More detailed studies are described in later sections. The problem is that of a rigid circular smooth footing of diameter D resting on a layer of uniform clay of thickness 5 x D underlain by a rigid layer, shown in Figure 8.9. The footing is loaded undrained to a mean bearing pressure of $3C_u$, where C_u is the undrained strength of the clay. Hence the load factor on undrained bearing failure is approximately 0.5. Throughout the clay layer it is assumed that the soil has the same initial stress history and stress-strain properties as given by the low plasticity clay in test MR2.

8.6.2 For the purposes of this illustrative problem it is assumed that the changes in the total stresses σ_v and σ_r beneath the centre of the footing can be obtained by means of linear elasticity. At any depth z/D the value of $(\sigma_v - \sigma_r)/2$ can be calculated and the corresponding value of vertical strain is obtained from Figure 8.7. Figure 8.9b) shows the distribution of vertical strain beneath the centre of the footing obtained using this procedure. In Figure 8.9c) the variation of normalised settlement $\delta/\delta c$ is plotted against depth and compared with the distribution for a linear elastic material. Figure 8.9d) shows the variation of apparent secant modulus E_u/C_u with depth corresponding to the strain distribution given in Figure 8.9b), assuming linearly elastic behaviour.

This illustrative problem points to the following important conclusions :

(i) From Figure 8.9b) it can be seen that the strains beneath the centre line are always less than 0.1%, even though the load factor is 0.5. As stiffness changes rapidly with strain over this range significant variations of stiffness with depth are also indicated.

(ii) From Figure 8.9c) it is evident that the settlement for LPC2 reduces much more rapidly with depth than for a homogenous linear elastic material.

(iii) The results given in Figure 8.9d) show that, had field measurements of settlement at various depths been made down to Z/D = 3.5, a linear elastic interpretation would have suggested an approximately straight line increase of stiffness with depth. Such features would normally be attributed to non-homogeneity with depth, rather than non-linearity.

(iv) Although the soil below the centre line has not reached yield at a load factor of 0.5, the high bearing stresses predicted by elastic theory would give rise to zones of local yield beneath the perimeter of the footing.

8.7 ANALYSIS OF SOME BOUNDARY VALUE PROBLEMS

8.7.1 The purpose of this study is to analyse a variety of boundary value problems using both non-linear elastoplastic and linear elastic undrained stress-strain characteristics in order to identify any major differences in the results obtained. The boundary value problems that have been considered fall in two groups which will be discussed separately :

(i) Footings and cavity expansion problems.
(ii) Axially loaded piles.

The analyses were carried out by means of the finite element program ICFEP which is described in Appendix 1.

8.7.2 The first problem considered is that of a vertically loaded rigid smooth circular footing of diameter D resting on the surface of a uniform layer of clay. The depth assumed for the clay layer was 5D which approaches the asymptote for infinite depth with a linear elastic material, Poulos and Davis (1974). Two soil types were considered for the analysis;

(i) The undrained non-linear soil model LPC2 described in the previous section, having K_o = 0.72, C_u = 220 Kpa.[1]

(ii) Undrained linear elastic soil with E_u = 1056 MN/m^2 (i.e. the initial maximum value for LPC2) and Poisson's ratio = 0.49.

8.7.3 The finite element mesh is shown in Figure 8.10, and loading was carried out by applying increments of uniform vertical displacements to the ground surface. The predicted relationship between average bearing pressure and settlement (expressed as δ/D) is shown in Figure 8.11 for the LPC2 soil model, and yielding develops under the centre line at a load factor of 0.58. If an infinitely fine mesh were to be used for the analysis, yield would occur under the

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[1] Note that the strength and initial stresses are double those for test MR2.

perimeter at even the lightest loads. However, the chosen mesh did not permit local yielding until a load factor of 0.36 had been attained. This feature fortunately allows the effects of pre-yield non-linearity and post yield flow to be separately assessed.

8.7.4 Figure 8.12 shows plots of normalised settlement, and the non-linear analysis gives a profile before yield in which the settlement reduces far more rapdily with depth than elastic theory predicts. The profile at L_f = 0.52 shows this trend to be accelerated by the onset of local yielding.

8.7.5 Figure 8.13 shows the predicted ground surface profiles at load factors of 0.3 and 0.52 (i.e. before and after first yield) for the LPC2 model compared with linear elasticity. It can be seen that the influence of the stiffness variation before yield, compared to linear elasticity, is to strongly concentrate the settlements around the loaded area. The onset of local yield further accentuates this behaviour. It is interesting to note that a linear elastic material with stiffness increasing with depth (a Gibson soil) also exhibits a concentration of settlement towards the loaded area (Gibson 1967, Burland et al. 1973). Such behaviour has important implications for soil-structure interaction.

8.7.6 Figure 8.14 shows contours of deviatoric strain at a load factor of 0.52. The small region of plastic yield is shown shaded. It can be seen that at this stage most of the soil is experiencing smaller shear strains than those developed in a triaxial test at 0.1% axial strain.

8.7.7 Figure 8.15 shows the distributions of stress change beneath the centre of the loaded area. The distributions for load factors of 0.3 and 0.52 for the LPC2 model are compared with linear elasticity. It can be seen that the influence of the non-linearity of the stress-strain characteristics is to increase the vertical and horizontal stresses. The effects are most pronounced for the radial stresses some depth beneath the footing and are less marked for the

vertical stresses. It is of considerable interest to note that the changes in deviator stress are much less sensitive to non-linearity; the approximation employed earlier in the simple illustrative problem does not therefore lead to large errors in the calculation of undrained centre line settlement. Figure 8.16 shows the vertical base contact stress distribution; it can can be seen that the influence of non-linearity and local yield is to make the stresses more uniform. The effects are to shed load towards the centre and to considerably decrease the stress concentrations at the edges (which are of course infinite in the linear elastic cases).

8.7.8 The assumption that soil stresses can be calculated from linear elasticity is central to routine foundation engineering. To further check the results for the rigid footing calculations were carried out for the case of a circular flexible load with Z/D = 5.0. The plots of centre line stresses for load factors of 0.3 and 0.5 are compared with elastic profiles in Figure 8.17. In this case the vertical stresses are insensitive to the constitutive law, but linear elasticity underestimates the radial stresses and overpredicts the deviator stress profile.

8.7.9 In summary, with footings the non-linear stress-strain characteristics have a dominant influence on the form and scale of the displacement distributions and a less marked, but nonetheless significant, influence on the stress distribution.

8.8 EXPANSION OF A LONG CYLINDRICAL CAVITY

8.8.1 This problem was chosen for analysis since it simulates the behaviour of an ideal pressuremeter and also has relevance to the behaviour of piles. An axi-symmetric strip mesh was used for the finite element analysis which extended radially to 100 times the initial radius of the cavity. A plane strain condition was maintained in the vertical direction and increments of radial displacement were imposed at the inner surface of the cavity. Two soil types were modelled : (i) the undrained non-linear model LPC2 and (ii) an undrained linear elastic material with E_u = 1056 MN/m^2 and ν_u = 0.49.

8.8.2 Figure 8.18 shows the predicted relationship between P/P_{max} and $\Delta V/(V_o + \Delta V)$. Here, P is the increase in cavity pressure from the initial radial stress, V_o is the initial volume of the cavity and ΔV is the change in volume of the cavity. The ratio P/P_{max} is a load factor, L_f, and it can be seen that first yield occurs at a relatively early stage. Interpreting the results as from an ideal pressuremeter, it would be conventional to derive the "modulus" either from the data prior to yield ($Lf < 0.3$) or from unload/reload cycles after yield. For the pre-yield conditions the pressure-volumetric strain curve can be seen to be markedly non-linear when compared with the linear elastic case having the same initial stiffness. The results of finite element calculations will be further analysed in Section 8.10.

8.8.3 The analysis of an unload/reload cycle was not attempted because of the assumptions which would have to be made concerning the stress-strain characteristics associated with loading reversals.

8.9 AXIALLY LOADED PILES

8.9.1 The type of finite element mesh used to study the cavity expansion problem can also be used to investigate the axial loading of an element of a long rigid pile, Potts and Martins (1982). Instead of prescribing radial displacements at the inner boundary, vertical displacements are imposed. The same non-linear and linear elastic soil models were used.

8.9.2 It has been noted that the radius of integration, r_m, and hence the width of the mesh, control the absolute vertical displacements of the pile, Cooke (1974). Randolph and Wroth (1978) give a relationship for r_m in terms of Poisson's ratio and pile slenderness, and the mesh used here is appropriate for a pile with a length 40 times its diameter.

8.9.3 The predicted load-displacement relationships are given in Figure 8.19. The radial profiles of vertical displacement away from the pile shaft corresponding to various load factors for the LPC2 model are given in Figure 8.20. For comparison the profile from the linear

elastic calculation is also shown. It can be seen that the effect of non-linearity is to concentrate the vertical displacements towards the pile as the load is increased. This behaviour is in general agreement with direct measurements of displacements and strains around loaded piles as reported by Cooke, Price and Tarr (1979).

8.9.4 The pile element analysis is highly idealised and it was decided to investigate the effects of non-linear soil properties using the more representative case of a compressible pile of practical dimensions. A solid pile, 0.75 metres in diameter and 30 metres long was selected with a modulus of 30×10^3 MN/M^2. Such a stiffness is appropriate for either a steel pipe pile or a reinforced concrete pile, and was 28 times the maximum soil stiffness. The finite element mesh for the study is shown in Figure 8.21, and no account was taken of any effects of installation on soil properties or initial conditions. Loading was simulated by applying increments of vertical displacement to the top of the pile. Six cases were considered :

P1. The soil was everywhere represented by the non-linear model LPC2 including the soil immediately adjacent to the pile shaft (i.e. $\alpha = 1$).

P2. As for P1 but with the undrained strength at the pile/soil interface reduced by a factor of two (i.e. $\alpha = 0.5$). The stiffness of this weaker zone was not reduced.

P3. The soil was linear elastic - perfectly plastic with C_u everywhere equal to that of LCP2 ($\alpha = 1.0$). The stiffness was given by $E_u = 2400\ C_u$ which is half the maximum initial stiffness of LPC2.

P4. The soil was linear elastic with E_u taken at the maximum value given by LPC2. Additional elastic runs were carried out with various E_u values to relate load settlement ratio to soil stiffness.

P5. As for P1 but with the pile modulus increased by a factor of 10^3.

P6. As for P4 but with pile modulus increased by a factor of 10^3.

8.9.5 The relationships between load and settlement of the pile head are given in Figure 8.22. In all four of the elasto-plastic cases local plastic yield (indicated by arrows) was reached at settlements of less than 2 mm. It is of interest to note the close agreement between cases P1 and P3. The stiffness of the pile itself can also be seen to have a major influence on settlement.

8.9.6 Figure 8.23 shows the radial profiles of relative surface settlement (δ_r/δ_p) for cases P1 to P5 where δ_r is the settlement at radius r from the pile centre and δ_p is the settlement of the pile. The profiles, except for case P4, correspond to a load factor of 0.5. It can be seen that the settlement profiles are very significantly concentrated towards the pile for the first three elasto-plastic cases in comparison with the linear elastic analysis. By comparing the settlement profiles for cases P3 and P5 with cases P1 and P2 it can be seen that much of this concentration is due to local plastic yield - an effect which is related to pile length and relative compressibility. Further from the pile, the settlement profiles are strongly influenced by the pre-yield stress-strain characteristics. These features would be most important in the evaluation of pile group interaction.

8.9.7 The location and extent of local yielding is shown in Figure 8.24 in which contours of deviatoric strain E are plotted for case P1 with a load factor of 0.5. It can be seen that yielding has occurred near the ground surface and that, except for a very narrow zone immediately adjacent to the pile, the bulk of the soil experiences only very small strains, with E less than 0.02%. Figure 8.25 shows the mobilisation of shaft resistance τ/c_a with depth for P1, P3 and P5 at $L/L_f = 0.5$. It can be seen that progressive failure is taking place and that the extent of this is dependent on both the relative compressibility of the pile and the pre-failure stress-strain characteristics of the soil.

8.10 INTERPRETATION OF LOAD DISPLACEMENT BEHAVIOUR USING LINEAR ELASTICITY

8.10.1 It is common to interpret measured displacements in the field in terms of linear elasticity. An apparent Young's modulus, E^a_u, is calculated by relating a characteristic displacement to a known loading condition, such as centre line settlement to mean bearing pressure. This same method has been applied to the load displacement data calculated using model LPC2. The variations of apparent modulus with load factor, L_f, produced by the different boundary value problems, could then be compared. In this sense the finite element results are considered as field data, and the section explores the properties that would be inferred by elastic interpretations.

8.10.2 Figure 8.26 shows the variation in E^a_u/c_u with L_f for the case of the rigid footing on a deep clay layer. It can be seen that even for load factors as low as one third, the value of E^a_u reduces from its initial value by about 40%. The broken line in Figure 8.26 represents the variation in secant E^a_u with L_f for a triaxial test with the soil model LPC2 (i.e. test MR2). In this case $L_f = (q-q_o)/(q_f-q_o)$ where q is the deviator stress and q_o and q_f are the initial and failure values respectively. It can be seen that the two curves are almost identical up to a load factor of about 0.5 i.e. for most practical ranges of working load. However, as L_f increases above 0.5 and the zones of local failure spread, the apparent moduli derived from the displacements of the footing fall below the values from the triaxial test.

8.10.3 Similar relationships between E^a_u and L_f have been derived for the other boundary value problems and are plotted in Figure 8.27 together with the result from the triaxial compression test. This latter curve may be conveniently used as a basis for comparison. The relationship for the 30 m long pile needs special mention since it is complicated by the effect of pile compressibility. The relationship between E^a_u and load settlement ratio was obtained by carrying out eight analyses in which E^a_u was varied but the pile stiffness was fixed. Hence, for any given point on curve P1 in Figure 8.22 it was

possible to obtain the apparent modulus E^a_u and thereby derive the relationship between E^a_u/C_u and L_f given in Figure 8.27.

8.10.4 For all the cases referred to, the apparent modulus E^a_u relates to the deflection of the point of application of the load (or stress), and the load factor is clearly defined. Bearing in mind that Figure 8.27 relates to a specific non-linear soil model applied to particular boundary value problems the following observations can be made :

(i) In each case the apparent modulus reduces continuously as the load factor increases.

(ii) For any given load factor there is a considerable range of values of E^a/C_u. For example, at $L_f = 0.5$ the rigid pile element gives the stiffest response ($E^a_u/C_u = 3,700$) and the cavity expansion gives the softest ($E^a_u/C_u = 300$); these two curves fall to either side of the triaxial characteristics.

(iii) First plastic yield occurred over a wide range of load factors. Hence the values of E^a_u/C_u at the onset of yield vary from 3,000 at $L_f = 0.22$ for the 30 m. long pile with $\alpha = 1.0$, to 800 for the triaxial test ($L_f = 1.0$).

8.10.5 The two extreme cases in Figure 8.27 are of interest (i.e. the expanding cavity and the rigid pile element) since both form the basis of insitu tests. It is clear that great care is needed in evaluating the stiffness of the ground from such data. The value of these tests would be increased if the full characteristic of apparent modulus with load factor was reported, rather than a single arbitrary stiffness value.

8.11 DISCUSSION OF ANALYSES

8.11.1 The series of analyses are intended to give a preliminary appraisal of the effects of the non-linear soil behaviour observed in

the Magnus and Canons Park laboratory tests. In order to restrict the number of variables, only the undrained behaviour of a homogenous layer of an isotropic material under monotonic loading has been considered.

8.11.2 The simple empirical stress-strain expression used for the calculations provides a good fit to the undrained behaviour of a lightly over-consolidated low plasticity clay in triaxial compression. Non-homogeneity could be considered without undue difficulty, but if drained conditions (or cycles of loading) were to be considered a more complex model would be required. The considered material, LPC2, is probably stiffer than most soils but is not unusually non-linear. Recognising the limitations of the study, the following preliminary conclusions can be made:-

1. In all the boundary problems considered there are wide variations of soil stiffness within the soil/mass. The response of the continuum at working load levels is to develop enclaves of plastic flow contained by masses of far stiffer material. There are, however, steep gradients of Young's modulus within the containing soil.

2. The analyses show that such behaviour leads to a number of difficulties in the use of the theory of linear elasticity to predict ground movements. The non-linear solutions often give markedly different ground surface profiles and patterns of soil straining.

3. The stress distributions within the soil mass, and the contact forces between soil and structure, are also sensitive to the stress-strain behaviour of the soil. These effects are least marked in the case of circular footings.

4. In problems such as footings it is often reasonable to combine linear elastic theory and non-linear soil properties to carry out approximate evaluations of centre line settlements. In other cases, such as the

estimation of the displacement of a group of long flexible piles, linear elastic theory is not satisfactory.

5. The study shows that the back analysis of field performance and insitu tests is likely to lead to a wide range of possible values for the linear elastic modulus, E^a_u, for a single material. Whilst linear elasticity remains the most convenient tool for expressing measurements of soil stiffness, its limitations must be recognised. In particular, the importance of load factor and the tendency towards concentrations of strain close to loading boundaries must be taken into account. If the non-linear nature of soils is not acknowledged, comparisons of field and laboratory measurements can be confusing.

6. The use of instrumentation systems in field monitoring which allow the direct comparison of insitu stresses and strains is likely to be of great value in understanding soil behaviour and assessing the applications of the laboratory techniques.

PART 3, REANALYSIS OF FIELD LOADING TESTS

8.12 LONDON CLAY DATA

8.12.1 Following from the non-linear parametric studies, it is useful to carry out a reanalysis of a number of field loading tests, both for London clay and low plasticity clays similar to the Magnus soils. Such an exercise will be described in the remaining sections of this chapter.

8.12.2 One of the principal reasons for carrying out the investigations at Canons Park was the existence of a substantial body of high quality field test data with which comparisons could be made. Price and Wardle (1982) report load-displacement curves for tests with Driven, Bored and Jacked piles at the Canons Park site, and Marsland

and Eason (1973) present detailed data for plate loading tests in London clay. Combining these records with the triaxial and pressuremeter data from Canons Park provides sufficient information for a comparitive plot of field E^a_u against load factor to be constructed, using four dissimilar boundary conditions.

8.12.3 The bored pile load test described by Price and Wardle (1982) was conducted with a 7 metre long, 168 mm diameter cast-insitu reinforced concrete pile. The uppermost 2 metres was sleeved to avoid the gravel layer, and the load-settlement curve is plotted on Figure 8.28. Considering the installation processes, the stress changes generated by augering and swelling would have caused local reductions in C_u and p'_o near the shaft. The bored pile can therefore be seen, as a lower bound to the undisturbed response to pile loading.

8.12.4 Randolph and Wroth (1978) present a method of calculating variations of shear modulus with depth from the results of a pile load test. The derivation assumes linear elastic behaviour in the soil and the 'influence factor' is found to depend on the degree inhomogeneity and the ratio of E_{pile}/G. In the present analysis a uniform soil was assumed with $E_{pile}/G \simeq 1{,}000$ and Poisson's ratio taken as 0.5. By making one further assumption recommended by Randolph and Wroth, that the radius of influence is equal to the pile length, it was found that $E^a_u = 0.602\ L/\delta$. Taking points from the experimental load-settlement curve thus allows E^a_u to be deduced as a function of load factor. A value of 100 Kpa was evaluated from Table 7.4 for the undrained shear strength, and this allows E^a_u/C_u to be plotted against L_f in a similar way to the data given in Figure 8.27.

8.12.5 The plate loading tests reported by Marsland and Eason (1973) have already been discussed. The authors present a detailed curve of load-settlement up to $L_f = 0.5$ for a test conducted at 17.5 metres in Chelsea, and from other data it is possible to deduce that the full pressure of 1,200 Kpa was mobilised after around 60 millimetres displacement. The first part of the loading curve is reproduced in Figure 8.29, and the substitution of $v_u = 0.5$ into Equation 8.3 gives a formula with which to deduce the curve relating L_f and E^a_u. Noting

that for this test C_u can be taken as 200 Kpa, the stiffness data can be plotted non-dimensionally as E^a_u/C_u by taking;

$$E^a_u = q/\delta . \frac{\pi}{4} . B(1 - \nu^2) \qquad \text{Eq. 8.3}$$

The resulting curve of E^a_u/C_u against load factor is given in Figure 8.29 a).

8.12.6 The pressuremeter tests carried out at Canons Park are reported in Appendix 3, and were discussed in Section 8.3. For each test two unloading stages were carried out from which relationships between apparent shear modulus, G^a, and ΔP can be deduced from the equation $G^a = 2\Delta P r_o/\Delta r$, given by Gibson and Anderson (1961). The pressure-volume change curves obtained from the first unloading stages are plotted in Figure 8.30, a) and b) using the normalised axes employed for Figure 8.27, and assuming a limiting value for $\Delta P = 6C_u$.

8.12.7 The trends of the pressuremeter curves shown in Figure 8.30 a) and b) are interesting. Firstly, the expected rapid reduction of stiffness with load factor is observed, and secondly the data fall into three distinct geological groups. However, the relatively small range of the unloading stages limits the range of L/L_f considered. In Figure 8.31 plots of the fuller ranges are shown for the final unloading stages, and contrasts these with the earlier unload-reload loops; and the previous strain history clearly has an important effect on the interpreted stiffness data. The test from 3.25 metres depth might be considered to give typical results for the site, and will be used to make comparisons with data from the other boundary value problems.

8.12.8 For the Canons Park field tests the appropriate K_o consolidated triaxial compression test with which to make a 'laboratory and field' comparison is LR7, the undrained experiment from OCR7. At 17.5 metres depth, the appropriate OCR for the Chelsea site is likely to be lower, and this will imply a larger ratio of E_u/C_u at a particular strain. Reference to Table 7.7 gives values for $(E_u/C_u)_{0.01}$ of

1480 and 960 at OCR's 3 and 7 respectively. To this extent, the Plate test E^a_u/C_u - L_f characteristic shown in Figure 8.29a) can be considered an upper bound to the data expected for a test at Canons Park conducted at, for example, 3 to 4 metres depth. It is also worth recalling from Figure 7.30 that LR7 yields at a load factor of 0.5, and therefore shows a relatively soft response above this loading level.

8.12.9 The data from the four types of test are plotted together on Figure 8.32 which uses the same axes as the theoretical plot given for LPC2 in Figure 8.27. As an aid to interpretation the axial strains corresponding to the triaxial modulus curve are indicated on the right hand side of the figure. Recalling that the Plate test and Bored pile lines represent upper and lower bounds in relation to undisturbed triaxial and pressure meter tests, the following points are noteworthy;

(i) The levels of secant stiffness are generally lower than those plotted in Figure 8.29. This is to be expected as the London clay triaxial tests gave lower stiffness ratios than the corresponding Magnus series, and the values of E_u/C_u for overconsolidated samples were less than those for lightly overconsolidated clay.

(ii) Accepting the differences in scale, the overall patterns for Figures 8.29 and 8.32 a) and b) have many common features. In both cases loading by a relatively stiff pile gives the slowest reduction of E^a_u with load factor, and the cavity expansion process the most rapid.

(iii) At small load factors the various field tests are tending towards similar values of E^a_u/C_u to the initial triaxial values, despite a wide variety of stress paths, initial conditions and strain rates.

8.13 MAGNUS CLAY AND SIMILAR LOW PLASTICITY TILLS

8.13.1 The scope for making comparisons between laboratory and field stress-strain characteristics is naturally limited for the Magnus

soils. Indeed, the PIP pressuremeter results are the only field measurements available. To supplement these data, reference is made to the collection of pile load test records assembled by Weltman and Healy (1978) for stiff glacial tills.

8.13.2 Considering the pressuremeter tests first, Figures 8.33 a) and b) present curves of E^a_u/C_u against load factor. These were derived using the same procedure as described for the Canons Park SBP results. The curves are divided into tests carried out in the upper overconsolidated layers, a), and the lower series b). For each plot the characteristic predicted using the soil model LPC2 is also indicated; this line corresponds to the stiffest possible response and can be considered an upper bound, particularly for the overconsolidated layers; the MR series of Magnus triaxial tests indicated that $(E_u/C_u)_{0.01}$ falls by 50% between OCR's 2 and 8.

8.13.3 The PIP modulus characteristics emphasize the strongly non-linear nature of the soil reponse. For the upper layers, the maximum values of E_u/C_u fall below the prediction made with LPC2, but tend towards the ratios observed in the initial portions of the triaxial tests on heavily overconsolidated samples. The lower clays show good initial agreement with the LPC2 prediction. For the latter group of soils, the reduction of stiffness with load factor is more rapid than expected, and this could be explained by disturbance, anisotropy or rate effects. Overall, the field data show very good agreement with the calculated behaviour.

8.13.4 The work of Weltman and Healy provides a further means of testing the non-linear analysis presented in Section 8.11. The authors collected the records of nearly 300 pile load tests carried out in glacial till, and these were made available through the offices of CIRIA. The original files were reviewed by the writer in a search for cases where the following criteria were met;

(i) The data included a good load-settlement record.

(ii) The pile lengths were between 10 and 30 metres.

(iii) The foundations were almost entirely composed of stiff low plasticity clay.

(iv) An adequate site investigation had been carried out.

(v) The records had been carefully made and the data were consistent.

A total of nine suitable tests were selected, and these are summarized in Table 8.2.

8.13.5 The nine experiments include almost equal numbers of bored and driven piles, and were all tested within a few days of installation. At this stage, the piles are unlikely to have reached their equilibrium conditions and would probably have developed higher capacities and stiffnesses to load with time. The average shaft adhesion factor, α , mobilised at failure was 0.77, the mean plasticity index 22% and the average C_u values fall between 91 and 200 Kpa. The soils parameters indicate good compatibility with parts of the Magnus soil profile.

8.13.6 In the course of their study Weltman and Healy calculated values of apparent Young's modulus, E^a_u, for a number of points on each load-displacement curve, using the iterative procedure outlined by Randolph and Wroth (1978). As mean values of undrained shear strength and ultimate capacities had been assessed, it was possible to express these data as the curves of E^a_u/C_u against load factor shown on Figure 8.34. The diagram also indicates the characteristic expected for a 30 metre long pile installed in LPC2, which reaches failure when $\alpha = 1.0$. It is useful to note that the considered tills were probably deposited in a relatively dense state, and would thus be expected to show greater similarity in triaxial tests with the more heavily overconsolidated Magnus layers. It will be recalled from Figure 6.24 that the initial values of E_u/C_u tended to fall with increasing overconsolidation for samples at OCR greater than 2.0.

8.13.7 Figure 8.34 proves the field data to have many common features with the variation calculated with LPC2; there are steep

reductions of apparent modulus with load and the initial stiffnesses are considerably higher than those normally adopted in design. The trend of the characteristics gives a mean E^a_u/C_u of 1,100 at a load factor of 0.5, which is only 30% below the LPC2 prediction. Recalling that LPC2 provides an upper bound estimate, the observed agreement is far better than could be reasonably expected.

8.14 SUMMARY

8.14.1 The purpose of Chapter 8 was to investigate the relevance of the stress-strain characteristics observed in locally instrumented triaxial tests to insitu behaviour. In the first section it was shown that special field tests at London clay sites confirm both the high initial stiffnesses, and strong non-linearity at small strains, observed in the laboratory. Although the insitu soil composition and the field stress paths differed from the laboratory experiments, surprisingly close agreement in the parameters $(E_u/p'_o)_{o.o1}$ and L was found. Attempts to recover the same information from field pressuremeter tests were less successful; Palmer's (1972) analysis was found to be an inappropriate tool, as the early part of the deduced insitu stress-strain curve was poorly defined, and was particularly susceptible to the inevitable disturbance that accompanies installation and inflation.

8.14.2 The second part of the chapter explored the analysis of a series of practical boundary value problems using the Model LPC2, which reproduces the undrained stress-strain characteristics of Magnus clay at OCR2. Many interesting features were noted including; (i) the tendency towards non-linear load-displacement relations, and (ii) patterns of soil straining which differed markedly from those predicted from linear elastic theory.

8.14.3 Two points of particular relevance were; the wide divergence between pile surface displacement profiles derived from LPC2 and the elastic analysis, and (ii) the observation that each type of field experiment could produce a different apparent modulus at a given load factor, even when the soil conditions are identical.

8.14.4 The deduced plot of E^a_u/C_u against load factor was used to reinterpret field load-displacement data in the third part of the chapter. A similar diagram to the theoretical plot was assembled for London clay using data from plate, pressuremeter and pile load tests. It is rewarding that the predicted trends were largely confirmed, and that close correspondence with the triaxial data could again be concluded. Satisfactory comparisons were also made between the theoretical studies and the PIP pressuremeter tests from Magnus and a series of nine pile tests in stiff low plasticity clay.

8.14.5 The studies reported in Chapter Eight therefore allow the triaxial stress-strain data to be confidently incorporated into the analysis of the large offshore pile groups. The observation of high initial stiffness appears to be compatible with field behaviour, providing that proper account is taken of the initial conditions. Similarly, the characterisation of soil behaviour as being strongly non-linear agrees well with field observations, and appears to be important in the study of pile-soil interaction.

CHAPTER 9

PILE TESTING AT CANONS PARK

9.1 INTRODUCTION

9.1.1 Chapter 3 considered the components of an effective stress analysis for displacement piles. It was shown that the processes of installation, consolidation and loading to failure are difficult to consider theoretically. Regarding the first two stages, the cylindrical cavity expansion analyses were seen to greatly overpredict the increases in radial effective stress, whilst the more sophisticated strain path solutions gave improved accuracy for clays at low OCR's. The various considerations of the pile loading process also involved radically different assumptions, and produced substantially different predictions. It was seen that well instrumented field scale pile tests provide the best means of validating theory, and help to explore the mechanisms that determine the development of capacity and response to load.

9.1.2 The parametric studies outlined in Chapter 4 showed how full-scale load-settlement relations depend on shaft compressibility, the effective stress distributions around the pile, and the assumed stress-strain laws in the surrounding soil mass. Chapters 5 to 7 described detailed laboratory studies of the properties of two soil groups and Chapter 8 related these to field behaviour. Chapters 10 and 11 will synthesize the preceding work in analyses of the foundation characteristics of two large North Sea platforms.

9.1.3 However, before considering the full-scale response of the groups of piles installed at the Magnus and Hutton sites, it is useful to consider the results of the experimental pile tests carried out at Canons Park. These tests were designed to help understand the mechanics of pile-soil interaction, and consider the case of displacement piles installed in stiff overconsolidated clays.

It will be recalled from the review given in Chapter 3, that there is a paucity of reliable data for these conditions, and that the strain path analysis has yet to be extended to consider clays with OCR > 1.5. As considerable depths of the soils at both North Sea sites consist of stiff overconsolidated clays, the Canons Park studies are important in forming the approach adopted in the full-scale analyses.

9.1.4 The main part of Chapter 9 therefore focusses on the development and use of a 5.5 metre long, 100 mm diameter, pile. Instrumentation is described which monitors the distributions of radial stress, shear stress and pore water pressure acting on the shaft of a closed end steel pile. Although only the data from the first test in a continuing programme is presented, many interesting and important results have been obtained. The experience gained in the 120 day duration experiment has also been used to assess the performance of the instrumentation, and it has subsequently been possible to design modifications in order to refine accuracy and improve long-term stability.

9.1.5 In addition to discussing the instrumented pile work, the first part of the chapter reviews the results obtained in earlier tests performed at Canons Park by workers from the BRS and Imperial College. These preliminary sections help to describe the testing facilities, and provide data which will later be correlated with the measurements obtained with the more sophisticated device.

9.2 PRELIMINARY PILE TESTS CARRIED OUT AT CANONS PARK

9.2.1 The Canons Park facility was first established by the BRS in 1978. Since that time comprehensive programmes of tests have been carried out with cyclic, constant rate of penetration and maintained load experiments. These have included vertically and horizontally loaded piles, installed by various means. For the purposes of this section it is useful to concentrate on the vertical load tests reported by Price and Wardle (1982), who experimented with three piles that were

168 mm in diameter, 7 metres long and sleeved for the first 2 metres. Each was of a different type, so that comparisons could be made between jacked, driven and bored piles at the same site. The two displacement piles were made from 6 mm thick steel tubing fitted with 60° conical tips; the bored pile was constructed with cast insitu reinforced concrete. The piles were all instrumented to measure axial loads at either three or four levels.

9.2.2 The jacked pile provided the most information, as it was possible to monitor the skin frictions developed during installation. The jacking was slow; a rate of 2 mm per minute was used for each 100 mm 'push' until peak capacity had been obtained, at which point the rate was increased by a factor of 10. Price and Wardle tabulate the observed shear stresses, and show that the magnitude of τ_{rz} at failure, for a particular soil level, falls with increasing pile penetration. The lowest section of the shaft develops the greatest adhesion, but this value increases with depth, as shown by profile 1 on Figure 9.1. Profile 2 of the figure indicates the maximum shear stress developed over the length of the pile in the last push of the jacking process. Profile 3 gives the corresponding plot for a constant rate of penetration test, carried out 2 months after installation. The triaxial undrained shear strength profile is also shown for reference.

9.2.3 The shaft friction data illustrates three noteworthy points;

(i) The developed stresses vary with distance from the pile tip in a way which could not be accounted for in the simple cylindrical cavity theory.

(ii) The stresses are sensitive to changes in the Cu profile.

(iii) There are considerable increases in peak skin friction with time, with an overall gain in capacity of 45% after two months.

9.2.4 Price and Wardle also assessed the dependence of capacity on rate of displacement in relatively slow CRP tests. From these, they estimate a 5% gain per tenfold increase in velocity.

9.2.5 The effects of the method of installation are illustrated in Figure 9.2, which plots the load displacement curves for the three piles, 2 months after installation. It is interesting that jacking gives the greatest capacity, and the bored pile the least. Part of the difference derives from the incomplete mobilisation of end bearing with the bored pile. Price and Wardle argue that this may be due to lack of axial pre-stress and the poor base condition that results from the construction method.

9.2.6 Considered quantitatively, the measured shaft capacities give overall α values[1] of 0.68, 0.63 and 0.58 for the jacked, driven and bored piles respectively. It is reassuring that similar coefficients were found in comparable tests conducted nearby at Stanmore, Tomlinson (1970), although the latter experiments showed driving to produce slightly higher capacities than jacking. Tomlinson's tests also showed a trend for capacity to continue increasing for at least 12 months after installation. O'Neill et al. (1982)2 reported the same trends from tests on piles driven into Beaumont clay, which they considered comparable to London clay.

9.2.7 Analysing the curves of Figure 9.2 in further detail, it is apparent that only driving produces a clear peak for the plotted range of settlement, and that the post-peak reduction amounts to less than 5% of the total capacity. This observation is compatible with the imperfectly formed residual surfaces that Martins (1983) expected to accompany dynamic installation. Whitaker and Cooke (1966) showed that bored piles also show reducing shaft capacity post-peak, but that considerably larger settlements are required to achieve residual conditions.

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[1] alpha is defined as $\overline{\tau_{rz}}/\overline{Cu}_{(o)}$, and $Cu_{(o)}$ is assessed from the triaxial tests discussed in Chapter 7.

9.2.8 The three Canons Park test piles indicate that jacking produces a stiffer response to load than driving, and that the bored pile is more compliant than both displacement types. These differences may result from changes in the soil properties over the period of consolidation, and might also be related to residual stresses locked in by pile installation, Clegg (1981).

9.3 I.C. INVOLVEMENT AT CANONS PARK

9.3.1 Following discussions with the BRS, it was agreed that field work for the instrumented pile project should begin in the summer of 1983. The area outlined in Figure 9.3 was selected for the experiments, with sufficient space allocated for a minimum of eight tests. The site work commenced with the site investigation described in Chapter 7, and three boreholes were sunk at the end of May 1983. The first, an exploratory augered hole, was positioned towards the west end of the test area. The second and third were fully sampled holes whose location is indicated on Figure 9.3.

9.3.2 In order to jack in and test the model piles, installations were required to provide reaction, vertical force and references for displacement monitoring. A jacking frame was errected, which consisted of two 10.1 m long, 0.357 m deep, I-beams. These were supported 1 metre from the ground by steel plattens resting on wooden mats, assembled from railway sleepers. The beams were fixed to the plattens using stirrups and bolts, and ran parallel to each other with a 260 mm space between the top flanges. For the first pile installation 10 tonnes of steel Kentledge was stacked on each platten, this was subsequently rearranged and augmented. A 4.5 metre high vertical frame was constructed by the BRS which ran along the I-beam flanges on retractible wheels, and could be firmly clamped to the reaction beams. The vertical frame supported a cross head which could be pinned in place at any one of the positions provided at 75 mm centres between the top of the frame and the reaction beam flanges. These arrangements are illustrated in Figures 9.4 and 9.5. Vertical force could be applied by placing a hydraulic jack between the pile head and the loading cross head. The pile displacements were to be monitored by

displacement transducers clamped to a 3.5 metre long box section reference beam, the ends of which were supported by 65 mm diameter driven piles.

9.3.3 The BRS foundations group had recently developed an apparatus for taking 26 mm diameter soil samples through retractable plugs fitted into the walls of tubular steel piles. It was decided to use the arrangements described above to install and load test a special 'sampler' pile, before beginning work with the fully instrumented device, as both Imperial College and the BRS group were keen to obtain data concerning the microfabric changes developed adjacent to jacked piles. A postgraduate student from Imperial College helped with this work, and later reported the experiments in his M.Sc. Thesis, Kitching (1983). Dr. Chandler advised on the interpretation of the microfabric studies, and the author assisted with practical and other aspects of the work. Inspection of the disturbed samples from the augered exploratory hole suggested that a pre-bored and cased hole would not be required, and the pile was jacked from a shallow starter hole to a final penetration of 4.7 metres.

9.3.4 Kitching (1983) describes the experimental work in detail, and only the jacking records and subsequent load test will be considered here; the results of the microfabric study have already been reviewed in Section 3.12.

9.3.5 The sampler pile was 5.5 metres long, 176 mm in external diameter and was fabricated from 6 mm thick mild steel tube. An instrumented conical tip was connected to the flat closed end which allowed base resistance to be separated from shaft friction. The pile was installed in mid-July 1983, with a 60-tonne double acting hydraulic jack fed by an electro-hydraulic pump being used to supply the driving force. A 30-tonne load cell was attached to the pile head to measure total jacking force, and a rotating potentiometer device was used to measure pile penetration.

9.3.6 The jacking load-displacement records are reproduced in Figure 9.6, and show that very large forces were required to penetrate

the upper two metres; the site investigation work later indicated a dense gravel layer at this elevation. As the pile tip passed below the layer the base and shaft loads fell steeply, and the total shaft resistance of the upper two metres could be estimated as 15 KN. As the pile was jacked to greater depths the shaft friction increased linearly, but the base resistance remained sensibly constant until the tip reached 4.1 metres. At this depth the end bearing increased by approximately 40%; reference to the borehole log given in Figure 7.2 shows that the jacking resistances correlate closely with the interpreted changes in strata.

9.3.7 The rates of displacement during installation varied with the resistance to penetration and the stroke of the jack, but the average velocity during each 'push' was around 50 mm/minute. At this rate, the shaft resistance developed in the clay layers between 2 and 4.7 metres was 102 KN, and the average shear stress 68 Kpa. A mean Cu value of 85 Kpa is found for the considered depths from Table 7.4, which infers an overall alpha value of 0.8. The final base load of 42 KN corresponds to a bearing pressure of 1,726 Kpa, and this implies a coefficient, N_c, of 13.3. If allowance is made for the relatively fast rates of displacement, these values of τ_{rz} and qb are similar to those found by Price and Wardle (1982).

9.3.8 During a pause period of approximately three weeks, Kitching attempted to take samples of the surrounding soil from inside the pile, and resealed each port after using the special BRS device. Twenty days after jacking, the pile was loaded to failure; a load of 100 KN was applied in two steps, after which smaller increments of load were applied until the peak capacity of 135 KN was obtained. Displacement was continued for a further 60 mm at a rate of 2 mm/minute. Figure 9.7 reproduces the load-settlement curve, and the general shape is similar to that given in Figure 9.2; both piles reach their full load at 2 mm, and neither shows any post-peak reduction in capacity.

9.3.9 The overall displacement rate of the sampler pile before failure may be estimated as 0.2 mm/minute, and at this velocity the maximum pile head load was 16% less than that observed during in-

stallation. It is interesting to note that the rate dependence of capacity assessed by Price and Wardle (1982) in slow tests can account for only 6% of the difference between the jacking and load testing phases. Kitching carried out further sampling operations after the loading test to failure, and these produced the most suitable material for microfabric studies. Clear evidence was seen in the thin sections of a principal displacement surface located a few millimetres from the pile face.

9.3.10 The conclusions drawn from the sampler pile tests can be summarized as follows;

(i) The gravel layer presented a considerable obstacle, and cased holes would be required for the instrumented pile tests.

(ii) The end bearing resistance closely followed the undrained shear strength profile.

(iii) There was microfabric evidence that slow jacking produces a well oriented shear surface close to the pile. As a peak was not seen in the overall load-displacement curve, it is likely that the observed surface was formed during the installation phase.

(iv) The effect of displacement rate on shear resistance is important.

(v) Capacity does not increase rapidly after installation.

9.3.11 After completing the sampler pile test, the loading frame and reaction system was dismantled so that sufficient preboring could be carried out for six experiments with the instrumented pile. This work was performed by BRS staff, who used a lorry mounted flight auger to bore six 2-metre deep, 170 mm diameter, holes to accommodate the pile tests, and eighteen 1-metre deep, 100 mm diameter holes for reference

point supports. Each of the bores was fully lined with PVC tubing. The cased holes for the pile tests were set out at 1.1 m centres, and reference point supports were provided 300 mm and 1.7 metres from the pile centres. Over the same period, development work continued at Imperial College with the instrumentation and data acquisition systems for the 100 mm pile. In June 1984 the equipment was ready and the reaction system was reassembled for the first instrumented pile test.

9.4 PILE INSTRUMENTATION

9.4.1 The fully instrumented pile used for the Canons Park experiments was an enhanced version of the equipment developed at Southampton University, as described by Johnston (1972). The equipment had been loaned to Imperial College, and the instrumentation was extensively refurbished and extended before the first field test in 1984.

9.4.2 Johnston (1972) describes the use of an instrumented pile for electro-osmosis experiments at a London clay site near Southampton. Sets of radial, shear and axial load cells were configured in a recoverable 4-metre long steel pile which was installed by slow jacking. During installation the pile was built up in four main parts, each 1 metre length fitting on top of the previously installed section, so that the pile could reach full penetration without moving the heavy jacking assembly.

9.4.3 The pile instrumentation consisted of two types of load cell, one of relatively simple design to measure the variation of axial force along the length of the pile, and the other to simultaneously measure the local total radial and shear stresses. The cells were arranged in the order shown in Figure 9.8, and when the equipment was handed over to Imperial College the set was complete, except for one axial load cell and a double local load cell assembly. The stack of 15 different sections was assembled using a system of opposing threads, which avoided the excessive twisting of cables that would otherwise result.

9.4.4 On arrival at Imperial College checks were made of the strain gauge circuits of the load cells. Many failed to respond, and others

indicated large zero offsets. It was therefore decided to place an order with Strainstall Ltd. to completely renew the gauges and gauge elements. This particular contractor was chosen as they had undertaken the original strain gauging work.

9.4.5 The axial load cells were designed to measure the load transfer characteristics during installation and testing. Each axial cell was manufactured from a solid bar of steel B.S. 970, En 16, Range S, and was machined to give a thinned central portion, 2 cm long. Johnston's design is illustrated on Figure 9.9, and for axial cells 1 and 2 the wall thickness was reduced to 0.75 mm; for axial cells 3 to 5 a thickness of 0.90 mm was adopted. Eight linear foil resistance strain gauges were bonded onto the thinned wall. These were arranged in pairs of axially and circum- ferentially alligned gauges, attached at the four quarter points of the inner cylinder wall. The gauges were wired into a single full bridge circuit which was connected to a four core screened instrument cable.

9.4.6 The original axial load cell design gave devices which are highly sensitive to changes in axial load; with Johnston's power supply the output was typically 1.5 mV per tonne. It was also noted that the cells were effected by radial total stress, with a typical sensitivity of 1 mV per 330 Kpa. In a typical test, the expected change in radial stress might be estimated as 300 Kpa and the average axial stress at failure as 5 tonnes. Under these conditions, the radial stress sensitivity would imply an error of approximately 10% in the axial stress measurements. The error can be eliminated by calibration, but is necessary to know the local radial stresses during a test. The interpretation of the axial load cells therefore relies on the measurements of σ_r made with the local load cells.

9.4.7 The design of the local load cells was based on the Cambridge boundary earth pressure cell, described by Arthur and Roscoe (1961). Their arrangement allows the forces acting on a surface to be resolved into components of normal stress, shear stress and turning moment. The design of these cells is shown on Figures 9.9 to 9.15. Figure 9.9 shows a schematic cross-section through a local load cell; the facing

surface is connected to the retaining block by two pairs of pillar elements, and a vertical shear element. The top and base pillar elements, and the shear strip, are strain gauged and wired into three separate circuits. Normal, radial, forces applied to the facing plate are carried almost entirely by the pillar elements, and shear forces by the shear strip.

9.4.8 The radial stress measuring elements were 26 mm long x 6.3 mm wide x 1.5 mm thick, made of ground flat stock, hardened and tempered to hardness 62-57 R.C. Two 120 ohm foil resistance gauges were attached to each side of the four pillar elements in a local load cell. The pillar elements were rigidly connected to the facing plate and retaining block by means 6 mm long x 4 B.A. stainless steel socket cap screws. A single shear element was incorporated in each cell, this was made of feeler gauge strip steel, 62 mm long x 13 mm wide x 0.45 mm thick. This was connected to the facing plate at its mid-point, and to the retaining block at its ends, again with socket cap screws. Four resistance strain gauges were attached to the shear element, with one each side of the two half sections.

9.4.9 The principal mode of deformation for the pillar elements is simple compression, although shear forces may induce slight cantilever bending stresses. The pillar gauges can only therefore be assembled in a half bridge circuit, with the dummy gauges being mounted on the thickest, unstressed, part of the facing plate. The top and bottom elements were wired into two independent circuits with output voltages V_1 and V_3.

9.4.10 The response of the shear strip to load is to produce the straining pattern illustrated in Figure 9.9. The development of both tensile and compressive stresses allows a full bridge circuit to be formed from the four foil gauges, and this gives good sensitivity and stability. Normal force changes induce relatively small bending stresses in the shear strip, and these give the slight cross sensitivity between σ_r and the shear element output, V_2.

9.4.11 Johnston carried out calibrations to assess the sensitivity and cross-sensitivity of the local load cells. He used a multiple loading procedure to derive relationships for radial and shear stresses and eccentricity of applied force in terms of V_1, V_2 and V_3. For his power supply, the direct sensitivities of circuits V_1 and V_3 to radial stress amounted to 120 Kpa/mV and the sensitivity of the shear element to shear force was typically 85 Kpa/mV. The calibrations showed the cross-sensitivities to be significant, although small. Calibrations were also made of the local load cell stiffnesses; the radial and shear compliances were assessed as 1×10^5 Kpa/mm and 5×10^3 Kpa/mm respectively.

9.4.12 The strain gauging and waterproofing of the local load cells was carried out by Strainstall Ltd., and Johnston was satisfied with the performance obtained with the equipment in a number of field load tests. Overall, one axial load cell was lost through buckling, and the waterproofing failed on a single pair of local load cells.

9.5 REFURBISHMENT AND CALIBRATION OF THE LOAD CELLS

9.5.1 In order to restore the full level of instrumentation for the Canons Park work, new metal components were made for the missing load cells, and the complete set of equipment sent to Strainstall for re-gauging, waterproofing and cabling. Attention was then turned to developing precision calibration procedures for the load cells.

9.5.2 The calibration of the axial load cells was relatively straightforward. A pair of clamps was constructed which screw connected to the ends of the cell, and fitted into the jaws of a 250 KN Amsler recirculating hydraulic tension/compression test machine. A small jacket was made which fitted over the cell and was sealed at top and bottom by O rings. The inside of the jacket was waisted to provide a hydraulic chamber which could be oil filled and connected to a Budenberg dead weight pressure supply. It was thus possible to apply combinations of known axial force and radial stress. The gauges were energised by a closely regulated Coutant 10 VDC power supply, and the response monitored by a sophisticated Solartron Orion data logger.

Load-unload cycles were performed for a range of fixed radial pressures, and a multiple linear regression program used to analyse the data; the relationships were then expressed in the form;

$$A_L = a_1 + a_2V + a_3\sigma_r \qquad \text{Eq. 9.1}$$

Where V is the output voltage, σ_r the radial pressure, A_L the axial force and a_1, a_2 and a_3 load cell constants. Typical values for these are given in Table 9.1.

9.5.3 Separate equations were evaluated for compression and tension loading and a careful analysis made of the errors expected from the best fitting equation; regression coefficients of 0.9999 and accuracies of ± 0.05 KN were typical. The 0.1 μV resolution of the Orion data logger allowed a theoretical resolution of ± 5 x 10^{-4} KN to be obtained, but the stability of devices was limited by noise levels to around ± 0.1 KN. The response of the cells was generally faster than the time taken by the Amsler machine to stabilise after a load increment.

9.5.4 To calibrate the local load cells a more complex arrangement was required, and a photograph of the equipment is shown in Figure 9.16. The load cells were clamped in a steel cradle, which was mounted onto the vertical ram of a Budenberg dead weight load calibrator. A cross piece was rested over the active face of the load cell, as sketched in Figure 9.17; this made contact with the facing plate through a curved platten which was faced with a 5 mm thick rubber pad. A special bearing system was stacked between the cross piece and a horizontal reaction beam, and this allowed vertical forces to be imposed without developing any sensible frictional resistance to horizontal force[1]. In this calibration system, the vertical forces imposed the radial stresses and the shear stresses were developed by horizontal loading.

9.5.5 A cable, pulley and weight hanger system allowed the shear force to be directly transmitted to the rubber mat pressed against the

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1 A dead weight system was used to check the bearings, and their friction under load was found to be negligible.

load cell face. With this configuration it was feasible to apply closely controlled combinations of radial and shear force to cover the full range of stresses expected in the pile tests. It was necessary to carefully align the load cell and pulley levels to eliminate secondary turning effects, and to survey the system to find the point of loading in relation to the load cell centre. The eccentricity of the applied radial force could be varied by changing the position of the cell in its cradle, and the sign of shear loading reversed by rotating the cell through 180° and retesting.

9.5.6 The standard calibration procedure involved six stages;

(i) Subject the cell to 20 cycles of the maximum shear and radial loads to 'bed in' the device.

(ii) Setting the eccentricity to approximately zero, carry out a full load-unload radial force cycle with no applied shear stress.

(iii) With the same eccentricity, set a mid-range radial force and carry out a full load-unload shear force cycle.

(iv) Setting the radial force eccentricity at around 10 mm, apply load-unload cycles of both radial and shear force.

(v) Dismantle the rig and rotate the cell through 180°, and reset a small eccentricity. Then repeat (iii) for the reverse shear direction.

(vi) With the same shear sense, repeat (iv).

Five series of calibrations were thus made, and the recorded data could be analysed by a multiple linear regression technique to find best fitting relationships between τ_{rz}, σ_r, turning moment, and the voltages V_1, V_2 and V_3.

9.5.7 When the local load cells were returned from Strainstall the first aim was to assess their response time. In fact, the devices were unacceptably slow and failed to give 95% equalisation in less than 1 hour. It was suspected that the delays resulted from the micro-crystaline wax used by Strainstall as a waterproofing agent, and a cell was tested without this filling. The response time was then found to be good, but there was considerable hysterisis in the load-unload curves. On closer inspection, play was suspected at the cap head screw fittings which secured the gauge elements. Ideally the load cell should be machined from a single block of steel to reduce such problems, but the effects were greatly reduced by making small clamping pieces, and using a loctite retaining compound on the screw threads when reassembling the gauges.

9.5.8 Replacing the waterproofing proved to be more difficult. It was finally decided to fill the cells with a soft paraffin wax, and to seal the gap between the facing plate and the load cell body with rubber. A fillet of RTV rubber was moulded insitu before filling the cell with melted wax. A final hard outer layer was added to the sealing, by carefully trimming a 1 to 2 mm deep "trench" of rubber and refilling with an epoxy coal-tar compound. This system appeared to give adequate waterproofing, and response time checks showed 95% equalisation in a period of less than 60 seconds. An additional pre-caution was taken in coating the strain gauge elements with RTV rubber before final assembly. Prior to calibrating the devices, checks were made for gauge stability, and a tendency for overheating was noted with the shear strip. The voltage for this circuit was reduced to 5 Volts, which gave a four fold reduction in power consumption and greatly improved stability. These various modifications were carried out by staff at Imperial College.

9.5.9 Each of the eight local load cells was subjected to a full calibration series, and the data carefully analysed. A standard statistics program was used to derive three best fitting equations;

$$\sigma_r = b_1 + b_2V_1 + b_3V_2 + b_4V_3 \qquad \text{Eq. 9.2}$$

$$\tau_r = c_1 + c_2V_1 + c_3V_2 + c_3V_3 \qquad \text{Eq. 9.3}$$

$$M/A = d_1 + d_2V_1 + d_3V_2 + d_3V_3 \qquad \text{Eq. 9.4}$$

Typical values for the coefficients are given in Table 9.1. The regression coefficients for σ_r and τ_{rz} were typically 0.9999, with values of 0.97 being found for M/A.

9.5.10 Expected errors from the best fitting lines were evaluated, and these typically gave ± 5 Kpa and ± 0.5 Kpa for σ_r and τ_{rz} respectively. For a radial stress of 350 Kpa, the mean errors in M/A implied a typical error in the eccentricity of 0.5 mm. This discrepancy probably results from the difficulties of surveying the exact point of action for radial force, as much as imperfections in the load cell characteristics.

9.5.11 The above calibrations provide the constants necessary for the insitu measurements of σ_r, τ_{rz} and M/A. The expected errors include allowance for non-linearity, hysterisis, and stability over a period of 24 hours. No attempt was made to evaluate the effects of temperature change.

9.6 ASSESSMENTS OF LOAD CELL ACTION EFFECTS

9.6.1 When determining total stresses insitu, it is important to evaluate the way in which the load cell modifies the forces it is attempting to measure. With the load cells described, this can be estimated by considering the compliances in relation to the load-deformation characteristics of the London clay mass. A simple calculation of the working compression of the axial cells shows a displacement of 0.4 μm per KN applied force. During penetration or load testing, the cells will thus act as springs which separate the otherwise practically rigid pile elements. With a force of 5 tonnes the compliance will give 20 μm displacement over a length of 20 mm, which amounts to an axial strain of 0.1%. Reference to the test data presented in Chapter 7 suggests that such strains will significantly modify the soil stresses near the cells. This influence will be relatively unimportant during steady penetration, or pull-out, but could modify the residual stresses after installation.

9.6.2 The effects of gauge action for the load cells can be estimated by reference to linear elastic solutions for load-displacement relationships for the following cases; (i) stress changes developed at the boundary of a cylindrical cavity formed in soil and (ii) loading a rigid square plate resting on a semi-infinite soil mass.

9.6.3 If a sudden change in radial stress, $\Delta\sigma_r$, is imposed, the load cell face would displace radially by an amount, $\delta_r = \Delta\sigma_r.10^{-5}$. (Where σ_r is in Kpa and δ_r is expressed in millimetres). The solution of Gibson and Anderson (1961) for cylindrical cavity expansion shows that a radial movement of the boundary by δ_r would invoke a change in the boundary radial stress, $\Delta\sigma^*_r$ where;

$$\Delta\sigma^*_r = G.2\frac{\delta_r}{r} \qquad \text{Eq. 9.5}$$

$\Delta\sigma^*_r$ can then be seen as the error in the measurement of $\Delta\sigma_r$. Although the load cells are of finite length and occupy less than the full circumference, Equation 9.5 can be combined with the compliance to estimate the ratio $\Delta\sigma^*_r/\Delta\delta_r$. Substituting r = 50 mm, and taking a value of G equivalent to the typical triaxial measurement of $E_{u(0.01)}$, 1 x 10^5 Kpa, shows that $\Delta\sigma^*_r$ is less than 1.3% of $\Delta\sigma_r$.

9.6.4 In the alternative analysis, the load cell is considered as a rigid square plate, of width B, acting against a soil mass with G again taken as 3.3 x 10^4 Kpa. Applying the solution given by Poulos and Davis (1974) with v = 0.5, the error in measurement can be written as;

$$\Delta\sigma^*_r = 4.5.\frac{\delta_r}{B}.G. \qquad \text{Eq. 9.6}$$

Substituting as before, and noting that B is 77 mm, Equation 9.6 gives $\Delta\sigma^*_r$ as 1.9% of $\Delta\delta_r$. It is thus possible to neglect any cell action effect in the radial stress measurements.

9.6.5 Similar analyses may be applied to the shear action by noting that, for a sudden change in shear stress $\Delta\tau$, the compliance of the

cells gives a shear displacement, δ_s, equal to 2 x 10^{-4} mm per Kpa. The effect of this load cell displacement on the local shear stresses can be estimated by applying a shearing displacement, δ_s, to an axially symmetric soil mass and calculating the shear stress change $\Delta\tau^*$ so induced. Cooke (1974) derived an approximate equation for the shear stress transfer from a pile, and this can be rearranged to consider the case of a displacing cylindrical element within an elastic disc of soil;

$$\Delta\tau^* = \frac{\delta_s . G}{r_o} / . [\log_e(r_m/r_o)] \qquad \text{Eq. 9.7}$$

substituting for into the equation allows the ratio $\Delta\tau^*/\Delta\tau$ to be found; δ_s and G are specified above, r_o is 50 mm and r_m/r_o can be taken as 100, following from Randolph and Wroth (1978). The calculated ratio predicts the under reading of shear stress to be less than 3%.

9.6.6 Poulos and Davis (1974) provide an elastic solution for the case of a rigid square area undergoing shear loading. This expression leads to an alternative estimate of the shear cell action effect with;

$$\Delta\tau^* = \frac{\delta_s . E_u . 0.94.}{B} \qquad \text{Eq. 9.8}$$

Evaluating Equation 9.8 as before indicates a more significant under-reading of around 20%. This particular calculation ignores the axial symmetry of the problem and probably overestimates the cell effect.

9.6.7 In the four simple calculations given above, the soil was considered as a very stiff linearly elastic material. The non-linear boundary value analyses presented in Chapter 8 demonstrated that more realistic stress-strain assumptions lead to load-displacement ratios that fall rapidly with increasing load factor. Whilst the use of $E_{u(0.01)}$ was appropriate for the first three elastic solutions, a lower stiffness should probably be substituted into Equation 9.8, and this would predict a smaller shear cell action effect.

9.6.8 The simple analyses considered the response of the instruments when subjected to sudden stress changes, and allow the conclusion that the errors in the measurements made with the local cells are unlikely to be large. During steady pile movement the errors can be expected to become smaller. Only the shear force element appears to be sensitive to soil stiffness, and cross checks can be made between the directly measured stresses and those deduced from the axial force distributions.

9.6.9 A further difficulty exists with the radial stress cells in the alignment and profiling of the load cell facing plates. Despite the efforts of the manufacturers, in some cases the plates lay behind, or stood proud of, the cell body by as much as 1.0 mm. However, on installation, these peaks and troughs would be smoothed by soil infilling, and the errors could be estimated by comparing outputs from opposing pairs of load cells.

9.7 PORE PRESSSURE MEASUREMENTS

9.7.1 Johnston's experiments were carried out without any measurements of the pore pressures acting on the pile face, and this omission makes his data difficult to interpret in terms of effective stress. It was considered essential to develop piezometer cells for the Canons Park study, and the most expedient solution was to extend the laboratory piezometer probe experience reported by Hight (1982).

9.7.2 Four cells were constructed to the design shown in Figure 9.18. The cell bodies were made to connect into the existing instrument assembly, and provided screw fitted carriers for the miniature transducer blocks. The detachable blocks were machined to give a cylindrical profile, and sealed into the cell bodies with 'O' rings. Into each block was permanently set a Druck PDCR 81 miniature electrical piezometer, similar to those described for laboratory use in Chapter 5. The devices were of the 1,550 Kpa range and were sealed into the blocks with loctite. A small space, in the shape of a shallow truncated cone, was left between the piezometer stone and the outer cell profile, which was approximately 1.0 mm in maximum depth.

During testing this space was to be filled, just before installation, with soft deaired London clay. This filling assisted in maintaining full saturation, gave a high air entry pressure and avoided the soil-stone adherence problems experienced by Morrison (1984).

9.7.3 The cables to the transducers were terminated approximately 150 mm from the devices with special miniature plugs made by casting gold connectors in resin retainers. The transducer blocks were deaired and calibrated remotely from the pile, using purpose made portable perspex cells, and were mounted into their final positions just before pile installation. Saturation was achieved by prolonged immersion in pressurised deaired water, with frequent applications of suctions to draw air from the piezometers. Calibration was carried out using the procedure described in Chapter 5 for pressure transducers and good accuracy was found; in each case the expected error from the best fitting straight line was less than 0.5 Kpa. Table 9.1 gives calibration constants and shows the high sensitivity of the devices to pressure change. The manufacturers state a temperature sensitivity for the instruments that amounts to a change in zero offset of 0.8 Kpa/°C.

9.7.4 Hight (1982) describes checks on the response times for probes mounted on triaxial samples. Tests were carried out with the probes mounted against samples of London clay, and the full equalisation of an instantaneous change in all round pressure was generally achieved in less than one second. In the pile application, the assessment of response is more complex. When the piezometer first penetrates a particular layer, the total stress change could cause an undrained response in pore water pressure in the soil infilling. This would only equalise to the ambient pore pressure after consolidating of the 1 mm thick London clay pad. Considering a time factor, $T_v = 1$, as representing full consolidation, the time for the process can be calculated for a C_v value of 0.3 m^2/yr, as 1.8 minutes. However, once the pad of clay has been consolidated, the compressibility will fall and the response time become smaller. An outer saturated porous stone would in principle give a better response, but might be more difficult to saturate before testing. The compliance of the silicon sensing

diaphragm within the Druck devices is negligible and, with an internal water-filled volume of 1 mm^3, no cell action effect need be considered for the transducers.

9.8 INSTALLATION OF THE INSTRUMENTED PILE AT CANONS PARK

9.8.1 It is notoriously difficult to measure the pore water pressure and stress changes that accompany the installation of a displacement pile. Indeed, the consistency of most published data is generally poor, and is certainly less satisfactory than the standard achieved in routine laboratory tests on soils. Nevertheless, Johnston's previous successful use of the load cells, the performance tests and calculations described in the foregoing sections, gave confidence that the measurements at Canons Park would be of good quality. Whilst the transducers employed in the laboratory were typically accurate to ± 0.1%, the measurements made with the instrumented pile are probably reliable to within ± 5%. However, the instrumentation includes a degree of redundancy, and the overall accuracies are sufficient to allow a detailed study of the effects of installing a jacked pile in London clay.

9.8.2 The elements of the pile were assembled in the laboratory with the threaded junctions being sealed with 'Boss White' compound and epoxy coal tar caulking. The instrumented length, from the pile tip to Axial cell 4, was fitted into a single length with great care being taken over the loom of 16 cables. On site, the instrumented section was lowered into the 2 metre deep cased hole and the remaining lengths added on. At the end of this stage the pile head stood 3.5 metres above surface, and only the first two groups of load cells were positioned below ground level. As the pile was gently lowered into the casing, the transducer blocks for pore water cells 1 to 3 were fitted, and their cylindrical profiles made up with deaired London clay. A sheet of cling-film was slid over each assembly to prevent drying, and these detached as each cell reached water level. Lowering the pile displaced the 1 metre depth of ground water that had accumulated in the casing, and the free water level remained at the surface for the full three month duration of the pile test.

9.8.3 The instrument cables were terminated in a ground level junction box, which connected the 33 transducers to the power supply and data logger located in the nearby instrument cabin. Readings of all channels were taken every 5 minutes and were automatically stored on magnetic tape.

9.8.4 When the main section of the pile was resting on the base of the cased hole, the two metre blank section was added, and the top axial load cell and jacking head were fitted. The loading cross head was then installed, and the electrically powered hydraulic pump and ram system set up. Markings had been made at 50 mm intervals along the pile length and, when all was ready, jacking commenced with the data logger automatically recording all channels at 120 second intervals. The rate of penetration was recorded manually during installation.

9.8.5 A lightweight 30 tonne jack was used, and this could be manhandled when the crosshead position was lowered. The jack acted through a steel ball to reduce shear loading, and a system of steel spacing pieces was used to reduce the number of crosshead repositionings. The stroke of the jack was 75 mm and, allowing for seating play and the bending of the reaction beams, the effective pile displacement was around 35 mm per push. In each push the load was built up rapidly giving a typical penetration rate of 500 mm/minute. This is considerably faster than the jacking rates employed by Price and Wardle (1982) or Kitching (1983). The jacking operation lasted a total of 70 minutes, including a pause period when a faulty hydraulic seal was repaced on the jack. The transducer block for pore water cell 4 was finally installed as the pile tip approached a depth of three metres.

9.8.6 As jacking continued the pile head load gradually increased with depth. After the tip had exceeded 4.5 metres depth, the jacking force began to approach the capacity of axial cell No. 4, and at 5.2 metres the upper two axial cells buckled. Jacking was stopped, and the data logger reprogrammed to continue reading with a reducing frequency. The failure was surprising, as Johnston's tests had also been carried out in London clay, and extrapolating previous tests at Canons Park suggested a reasonable margin of safety. The two suggested

reasons for the overload were the faster rates of penetration and claystones.

9.8.7 The pile was left in place from the 5th July until 22nd October, 1984, and during this period continuous readings were taken from all channels. At the end of the interval a controlled load tension test to failure was carried out, and the pile was finally extracted on the 23rd October, 1984. In the following sections the measurements made during jacking, equilibration and load testing will be separately discussed.

9.9 MEASUREMENTS MADE DURING JACKING

9.9.1 The pile installation was achieved by approximately eighty 35 mm pushes. In each push the full capacity of the pile was mobilised for a period of around 15 seconds, and the proportion of the installation time during which the pile was rapidly penetrating the ground was perhaps only $^{1}/_{15}$th of the total. Most of the 70 minute long installation phase was spent manipulating the jacking and reaction system between pushes. The data logger recorded information at two minute intervals, and few of the records therefore coincide with the peak conditions achieved during a push. Two peak data sets will be discussed, but it is worth noting that the calculated response time of the pore water pressure cells is greater than the 15 second push time. Most of the presented measurements should therefore be regarded as representing conditions shortly after jacking to a particular depth.

9.9.2 As the pile was first lowered into the casing it was noticed that the transducer zero readings shifted. This was thought to result from a temperature sensitivity of the instruments, and the ground water was assumed to cool the load cells and piezometers. No temperature checks had been made for the load cells, but the piezometer manufacturers quote a sensitivity which amounts to a 4 Kpa shift for the expected 5°C change in temperature. To account for this temperature effect the various calibration zero's were reassessed from the outputs given by each transducer when positioned in the casing 1.5 metres below ground level. This allowed a period of at least 30 minutes for cooling

in the ground water, and corresponded to a condition when the radial stress and pore water pressure were known to equal 15 Kpa.

9.9.3 The measurements of total radial stress are shown in Figure 9.20 which traces the average for each pair of local load cells against depth of penetration. The trends are consistent, and show step increases at 2.4 and 4.5 metres depth; these levels correspond to the junctions of the Head/Disturbed London clay, and the Disturbed London clay/Intact Brown London Clay. Considering the deepest instruments, 0.23 metres above the tip, the values of σ_r amount to approximately 4 Cu. In a given stratum the instruments further up the shaft tend to show progressively lower stresses, with σ_r falling closer to 3 Cu, 2.2 metres above the tip. The values of σ_r recorded by each opposing pair of local load cells typically differed from their means by less than 15 Kpa, or 5%.

9.9.4 Figure 9.21 shows the corresponding diagram for the piezometer cell measurements. In the Head and Upper Disturbed London clay layers only small pore water pressures were measured, and these corresponded to almost hydrostatic conditions. Below 2.8 metres depth the pressures increased sharply and rose above their pre-installation values to a maximum of 350 Kpa, or 4 Cu, at 3.5 metres depth. After attaining this maximum the pressures fell rapidly and considerable dilation was indicated in the Lower Disturbed London Clay, and laminated Intact Brown London Clay strata. The traces could be compared to the results of a large piezocone sounding, as they sensitively pick out the changes of the soil response to shearing. The pattern of piezometric changes over the length of the shaft is more complex than that seen for the radial stresses. For the layers above 3.5 metres, the pore pressures appear to fall with increasing penetration; whilst in the lower layers the reverse trend is apparent.

9.9.5 An alternative method of presentation is to sketch the distributions of σ_r and u developed on the pile at particular time instants. This is shown on Figure 9.22, and the trends discussed above appear more clearly; at a given level σ_r declines with time and pore pressures either rise or fall, depending on the depth relative to the 3.5 metre level. It is, however, difficult to construct such summary

plots when the pressures are only known at three points. Figure 9.22 c) shows three profiles of σ'_r deduced from the measurements of σ_r and u. The trend of the data is for σ'_r to increase with penetration, although there is an anomalous dip at around 3.5 metres. This corresponds solely to the pore pressure peaks observed at this depth, and could result from a lag time effect in the piezometer cells; the clay infilling could simply be showing an undrained responses to a rapid total stress change. The radial effective stresses developed in a given layer appear to fall slightly as the tip passes, but the measurements vary too rapidly for this to be certain. At the end of installation the general pattern is for σ'_r to exceed its initial K_o profile by a factor between 2 and 3.

9.9.6 Over the jacking period the data logger reading cycle coincided with the peak installation loads during four of the 80 pushes. Figure 9.23 plots the distributions of axial force with depth noted on two such occasions. The diagrams show the pile head load to be approaching 11 tonnes at 5 metres penetration, with a base resistance greater than 2,000 Kpa. Assuming an end bearing coefficient, N_c, equal to 9 gives a mobilised undrained shear strength almost double that measured in the triaxial tests. The gradients of the axial force-depth curves correspond to the average shear stress transfer between pile and soil, and both plots show $118 > \tau_{rz} > 114$ Kpa below 3 metres depth, with an average τ_{rz} of around 60 Kpa between 1.8 and 3 metres.

9.9.7 The τ_{rz} distributions inferred from the axial load cell data are plotted on Figure 9.24, which also shows the average of the shear stresses measured with the local load cells. The agreement is excellent, and both systems show skin friction stresses which vary less with depth, and are far higher than those noted by Price and Wardle (1982) or Kitching (1983). Simultaneous measurements were made with the radial stress and piezometer cells, and the tendency is for the total radial stresses to briefly rise during each push. The piezometers suggest that loading causes a decrease in pore pressure at the four metre level, gives an increase at 5 metres, and has little effect near the surface. Discounting the apparently low effective stresses recorded at 3.5 metres, and assuming no errors to exist in the other

measurements, the deduced values of $\tau rz/\sigma'_r$ fall between 0.23 and 0.4. Such ratios imply mobilised angles, δ', between 13 and 22 degrees. It is clear that the fast rates of penetration greatly increased the frictional strength, but loading may also have induced transient changes in pore water pressure that were imperfectly recorded with the piezometers.

9.9.8 In summary, the observations made during installation point to three important conclusions;

(i) The pore pressures generated on installation vary with the stratification and the distance behind the tip. The maximum recorded values are close to the predictions of $\Delta u/Cu_{(o)} = 3.5$ given Wroth et al. (1979), whose analyses were discussed in Section 3.6. However, at most levels the piezometers indicated far smaller pore water pressures than expected from cavity expansion theory. In the most dilatant layers negative pore water changes were found.

(ii) The total stress changes also vary with depth and position relative to the pile tip. The radial stresses acting on the shaft are smaller than those predicted Wroth et al. (1979) for installation in overconsolidated London clay. The effective stress distributions show ratios of $\sigma'_r/Cu_{(o)}$ which vary from 3 to 4 near the tip, to between 1.5 and 2 higher on the shaft; the cavity expansion theory predicted a value of 2.6 for $\sigma'_r/Cu_{(o)}$.

(iii) The skin frictions and end bearing developed during jacking are highly sensitive to the rate of penetration, and the deduced effective shaft friction angles are compatible with fast ring shear tests in the laboratory.

9.9.9 During installation the instrumentation performance was generally satisfactory; the records are consistent and good agreement is found when systems are cross-checked. The failure of the uppermost two axial cells made little difference to the monitoring of installation or equilibration processes. A replacement pile head load cell was manufactured for the subsequent stages of the tension load test and pile extraction.

9.10 MEASUREMENTS OF THE EQUILIBRATION PROCESS

9.10.1 Following jacking, the pile was left in place for $3^1/_2$ months, with the transducers being monitored continuously for the full period of 2,600 hours. After approximately 100 hours the first radial stress circuit of a local load cell started to become unstable, and with increasing time more radial stress channels showed faulty readings. Given the degree of redundancy in the instrumentation scheme, it was possible to continue monitoring the radial stresses for a further 600 hours, by which time 40% of the circuits were functioning incorrectly. The pore pressure cells and the three undamaged axial load cells operated perfectly for the full duration of the experiment.

9.10.2 Figure 9.25 plots the mean radial total stresses monitored at three levels for 600 hours; the uppermost set of cells hardly protruded from the casing and their measurements are of little value. The curves show a rapid fall in σ_r from the maximum values noted during jacking. Over the first 24 hours the reductions amounted to approximately 100 Kpa for the lower two levels, with a change of approximately 55 Kpa for the highest station. The rate of decline slowed continuously, but further reductions of between 50 and 70 Kpa developed over the following 600 hours. It is interesting that the data suggest small increases in σ_r after 400 hours for the upper two sets of load cells.

9.10.3 The pore water pressure trends are more complex, and it is useful to study the first 24 hours in some detail. The records are plotted in Figure 9.26 against time from the end of jacking, and three stages can be identified in each trace;

(i) A maximum pore water pressure is observed within 1 to 10 minutes. This could reflect a lag in the piezometer response, or some short term equilibration effect.

(ii) After reaching their maximum, the pore water pressures decline and show minimum values between 50 and 300 minutes after jacking.

(iii) The three instruments then show a slow climb in pore water pressure which is still continuing after 24 hours.

9.10.4 The longer term trends in the pore pressures are indicated in Figure 9.27. The increases in piezometric pressure persist until a second maximum develops after 50 to 100 hours. At this stage the excess pore water pressures amount to 310 Kpa near the tip, 75 Kpa at the uppermost level, and 50 Kpa at the middle position. The deepest instrument shows a rapid period of dissipation for 50 hours, but all three gauges indicate that the subsequent consolidation proceeds slowly. The records point to an acceleration in pore water dissipation during the second month, and after 2,600 hours the excess pressures range between 70 Kpa for the deepest cell, to 10 Kpa for the middle piezometer.

9.10.5 The observed variations in pore water pressure are complex, and represent a marked departure from the simple dissipation curves expected by Wroth et al. (1979). To help understand the processes, Figure 9.28 extrapolates the contours presented by Baligh and Levadoux (1982) for the octahedral shear strains induced by steady penetration of a round tipped pile. The K_o consolidated triaxial test characteristics reported in Figure 7.30, are used to define approximate limits to the zones where the strains should be sufficient to produce either; (i) full dilation at the critical state condition, or (ii) a smaller degree of dilation between yield and the critical state.

9.10.6 The figure clearly involves considerable extrapolations of an ideal analysis and the results of slow experiments, but it shows that a dilatant region, around 1 metre in diameter, could be expected to envelope the pile. Between the critical state limit and the γ_{oct} = 0.25% contour, the shear induced pore pressures would show radial gradients as steep as 400 Kpa per metre. Combined with these shear induced changes, would be an unknown distribution of pore pressures developed due to changes in mean total stress. Adding together these two components might easily produce a maximum in post-jacking pore pressure at some radius from the pile. Equilibration could then commence with both inward flow towards the pile and outward flow away from the radius of maximum pore pressure. Combining this mechanism with reductions in total stresses could explain the medium term increases in pore water pressure, and would imply relatively flat pore pressure distributions after the maximum had developed. Such small hydraulic gradients are compatible with the observed slow rates of long-term dissipation.

9.10.7 To explain the first peaks in pore water pressure, and the rapid decline to the short term minimum is more difficult. If it is postulated that the principal displacement surface for the pile-soil system exists some finite distance from the pile face, then a dead zone of material that moves with the pile could exist in a less sheared state than the surrounding soil. The piezometer cells would first register the pore pressures within this narrow zone, and these would exceed those beyond the displacement surface. In this case, equalisation would initially involve reductions in pore pressures at the pile surface.

9.10.8 The changes of piezometric pressure with time are further explored on Figure 9.29. The diagrams follow the development of the maximum pore pressure distributions around 50 hours after installation, and illustrate the slow subsequent dissipation. The plots also illustrate the tendency of the profiles to conform vertically from their highly non-uniform initial conditions.

9.10.9 The pore water pressure and radial stress data can be combined to calculate changes in radial effective stress up to 600 hours

after installation. Figure 9.30 shows profiles of σ'_r with depth for the medium and long-term cases. For the time span considered, the highest radial effective stresses are developed at the end of installation. The following 50 hours correspond to a period of rapid reduction in radial effective stress. This process slows after 100 hours, and briefly reverses at the deepest level. Only small reductions occur after this time, and it is possible that the 600 hour profile represents the minimum. The rates of dissipation accelerate after 1,000 hours, and the subsequent period probably involves a simpler pattern of radial consolidation. All the analyses reviewed in Chapter 3 predicted increases in σ'_r for dissipation under such conditions.

9.10.10 In summary, the observations during equilibration show phenomena which are more complex than the simple consolidation process described in the cavity expansion analyses. Immediately after jacking the radial effective stresses show a maximum which reduces over 2 to 4 days through a swelling process. The maximum pore water pressures observed after the swelling phase dissipate slowly without any measureable increases in σ'_r developing for the first 4 weeks after installation. The 600 hour profile shows a distribution of σ'_r which falls below the initial K_o profile at the uppermost instrumented position, but exceeds the pre-installation effective radial stresses at the level of the pile tip. The pore-pressure time graphs indicate significant consolidation over the following $2^1/_2$ months, and increases in σ'_r probably developed before the tension test of 22nd October, 1984.

9.11 RESIDUAL SHEAR STRESSES AFTER INSTALLATION

9.11.1 It is well known that residual shear stresses persist over the shafts of displacement piles long after installation. Cooke, Price and Tarr (1979) describe detailed measurements of these stresses for tubular closed end piles jacked into London clay, and Figure 9.31 illustrates the pattern observed for a solitary installation and the mid-pile of a small group. In the case of the single pile, a certain proportion of the base resistance is locked in as the relaxation of the steel shaft is resisted by shear stresses at the pile-soil interface.

9.11.2 The axial load, and local shear stress measurements made with the fully instrumented pile are used in Figure 9.32 to plot the residual forces acting on the instrumented pile at two stages; (a) just after installation, and (b) 735 hours later[1]. The arrows shown on Figure 9.32 plot the local gradients of axial stress change inferred by the shear cells measurements.

9.11.3 The plots are at first perplexing, until it is recalled that the axial compliance of the pile is almost completely due to the thin walled axial load cells. When the jacking force is released, the cells attempt to expand, and the soil resists this action by developing a local system of opposing shear stresses. As a consequence, the axial force diagram after jacking shows a cusp at each load cell level.

9.11.4 In the longer term this pattern is influenced by the stress and strain changes that accompany equilibration. Contractions in the lower strata appear to further compress the axial load cells, and balancing tensions are developed in the upper layers.

9.12 TENSION LOAD TEST

9.12.1 As a result of the difficulties with the local load cells it was decided to extract the pile before the piezometers had reached final equilibrium. The pile head load cell was replaced, but the remaining damaged cell could not be reached before the pile was extracted. A compression test was not feasible, but tension could be applied as the buckled cell would straighten under load. However, the pile head displacements would be influenced by the distortion of the cell.

9.12.2 The loading test was carried out on 22nd October, 1984, with the data logger recording information at two minute intervals. A

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1 At this stage the shear cells on all local load cells were still functioning.

tension coupling connected the pile head to a similar jacking system to that employed during installation. The pile was loaded to failure in increments of 6 KN, applied at the rate of one per minute. A pair of RME[1] displacement transducers and a dial gauge were mounted between the pile head and the 3.5 metre long box section reference beam. Continuous manual checks were also made of the load-displacement behaviour.

9.12.3 The test took approximately 32 minutes to reach failure, and Figure 9.33 shows the overall load-displacement curve. The characteristic is considerably softer than that observed by either Price and Wardle (1982) or Kitching (1983). This probably reflects the distortion of the damaged load cell. As the test was load controlled the displacement rates varied continuously but 0.15 mm/ minute can be considered typical before failure.

9.12.4 Figure 9.34 shows the traces of piezometric pressure and total pile load with time. It is interesting that changes in pore water pressure only start to occur as the tension approaches 85% of the pile capacity. The shallowest gauge is the first to respond, and reduces by approximately 10 Kpa before peak. The deepest cell only deviates after the last two load increments, and shows a 15 Kpa increase in pressure at peak capacity. No significant changes are seen at the intermediate level, but the piezometers show the pressures above and below to continue changing for several minutes after failure. These trends might reflect the large post-peak displacements, or could be the result of equalisation between the cells and an adjacent discrete shear surface. After reaching post failure maxima, the pore pressures were slow to return to their former values, and had not equalised 19 hours later.

9.12.5 The axial loads monitored during the tension test are summarized on Figure 9.35. Part a) shows the force-depth diagrams recorded at load factors of 0.5, 0.81 and 1.0. Part b) gives the deduced shear stress-depth plots. A clear progressive failure

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1 These devices are described in Section 5.

mechanism is indicated for the upper clay layers, where shaft friction appears to reduce by up to 50% in the second half of the loading test. It is interesting that this feature was not noted by Price and Wardle (1982) in their tests at the same site. An important difference between the two sets of experiments is the relatively high compressibility of the axial load cells used in the Imperial College tests, and this might explain the occurrence of progressive failure.

9.12.6 The earlier faults in the radial stress measuring cells were disappointing, as they preclude any discussion of the radial effective stresses or pile-soil friction angles developed during tension loading. Nevertheless, the tests produced valuable information and allow some preliminary conclusions to be drawn;

(i) The flexible pile system induces progressive failure.

(ii) The degree of drainage during the pile test was probably small, but large pore water pressure changes were only observed post-peak.

(iii) Relatively high shaft adhesions were achieved $3^1/_2$ months after installation, and the maximum values approached the initial undrained shear strength measured in slow triaxial tests. As a result of progressive failure the average ratio of $\tau_{rz}/Cu(o)$ at peak was slightly reduced to 0.87.

(iv) The end bearing pressure produced in the tension test was considerable, and amounted to an N_c value of 11.5 when divided by the initial undrained shear strength.

9.12.7 The effect on shaft resistance of the long equilibration period is more clearly illustrated in Figure 9.36, which combines the test measurements at three levels with the maximum skin frictions noted by Price and Wardle. It will be recalled that the earlier tests were conducted on steel piles jacked in at Canons Park, which were sub-

sequently tested using a slow constant rate of penetration. The data are directly comparable, and the results obtained by Kitching (1983) may be used to justify the assumption of no immediate gains in adhesion after installation. Similar trends are found for each level, and these reflect the slow pace of the dissipation curves plotted on Figure 9.27.

9.13 EXTRACTION OF THE PILE AND MODIFICATION OF THE INSTRUMENTS

9.13.1 Following tension testing, the pile was jacked out from the ground using the reverse of the installation procedure, and the shaft emerged from the water filled casing with a coating of clay that was approximately 3 mm thick. It was possible to check the functions of some of the transducers before dismantling and these indicated small drifts in the piezometers and axial load cells, with average respective errors of ± 5 Kpa and ± 1KN. Inspection of the local cells immediately showed that the failures had been associated with water ingress, as the sealing had been disrupted on many of the instruments. The interior of the pile showed signs that condensation had also caused some corrosion of the steel surface.

9.13.2 Measures have been taken to improve the instrumentation and prepare for further studies. Recognising the long periods required for equilibraiton, two new piles are being made so that up to four tests will be feasible per year. The instrumentation is being divided to provide four local cells, four axial cells and two pore water pressure cells per pile. Conical tips have been made, and these will ease installation through claystone layers.

9.13.3 To improve the waterproofing of the local load cells, special insitu rubber mouldings are being made which provide exact compression seatings for the outer facing plates. A soft rubber is being used which gives a relatively large area of sealing contact without impeding the cell action. New strain gauges and cabling are being provided for each cell, and tolerances are being improved for the profiling of the facing plates in relation to the piles.

9.13.4 Stiffer axial load cells are being made so that the vertical compressibility of the pile can be varied, and higher capacity cells are to be used to avoid buckling failures. Experiments are also underway to assess the potential improvements in piezometer cell response time that might be gained by replacing the soil infill with a facing stone.

9.13.5 Improvements in the data acquisition and energisation system are being made by separating the power supplies to the 40 transducer channels, and logging the supply voltages independently. Reductions in power, and underwater quality cabling are being used to improve long-term stability. The interior space of the piles will be kept vapour-free by the use of a trickle feed, dry nitrogen, system.

9.13.6 Calibrations of the various transducers will now include an evaluation of temperature sensitivity, and checks will be made of the operating temperatures of the devices under their working conditions.

9.14 DISCUSSION

9.14.1 The field work described in this chapter provides many insights into the characteristics of displacement piles. Combining the results of earlier BRS tests with specially instrumented experiments has helped to understand the processes of installation, equilibration and loading. Following the pilot work with the 100 mm diameter pile, the instrumentation is being reconfigured to give improved performance. The continuing programme of tests should further clarify the development of radial effective stresses during equilibration and pile loading.

9.14.2 Monitoring the jacking process showed how the developed stresses and pore water pressures varied rapidly with depth. Large increases in radial effective stresses were noted, and clear evidence was seen of strongly dilatant behaviour in the soil surrounding the pile. The relatively fast rate of penetration resulted in high values of skin friction and base resistance, and it is interesting to correlate these data with laboratory measurements and previous tests at the same site.

9.14.3 The London clay ring shear data summarized in Figures 7.42 to 7.44 demonstrate how fast movement on a pre-existing smooth shear surface can produce a high initial strength that falls with continuing displacement. Figure 9.37 a) plots the initial ring shear maxima against shearing velocity. Recalling that the local measurements indicated δ' angles between 13 and 22 degrees, compatibility between the laboratory and field angles of effective friction can be seen at the appropriate rates of displacement.

9.14.4 The second part of Figure 9.37 plots average installation shaft resistance, measured between 2 and 5 metres depth, against rate of penetration. Three cases are considered; the slow jacking reported by Price and Wardle (1982), the rapid installation of the I.C. instrumented pile, and the intermediate sampler pile experiment reported by Kitching (1983). This diagram summarizes the steep changes in skin friction found when penetration rates exceed 10 mm per minute. At slower rates, Price and Wardle estimate only a 5% change in capacity per tenfold change in rate.

9.14.5 The local measurements of stress and pore pressure allowed the variations in the radial effective stresses acting on the shaft to be evaluated. The recorded stresses were somewhat different from those predicted by cylindrical cavity expansion theory; at the end of installation, $\sigma'_r/Cu(o)$ measured at the pile shaft typically amounted to 65% of the theoretical value. It is useful to further examine the results by comparing the end of jacking σ'_r profile to the expected variation of p'_{cs} with depth. The octahedral shear strain levels near the pile shaft were sufficiently high to produce critical state conditions over the entire shaft length, so it is reasonable to assume that the mean effective stresses[1] equal the critical stage value p'_{cs}. The K_o consolidated triaxial tests on London clay showed p'_{cs} to be related to p'_o and OCR, and the observed relationship is plotted in

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1 As a first approximation it is assumed that any shear stress reversals imposed after reaching the critical state induce no pore water pressure response.

Figure 9.38. The initial mean effective stress p'_o, was obtained from the profiles given in Figure 7.19 and combined with the curve given in Figure 9.38 to construct the plot of p'_{cs} against depth shown in Figure 9.39. The laboratory tests were carried out with reconstituted clay, and it is likely that the response of the laminated ground below 4.2 metres would be to dilate more strongly, and to produce a higher angle ϕ', than the samples mixed from slurry. The constructed line could be seen as a lower bound for p'_{cs}, particularly at depths greater than 4 metres.

9.14.6 The ratio of measured σ'_r to calculated p'_{cs} at the end of jacking is plotted against depth in Figure 9.40. It is worth re-calling here that σ'_r may have been over-recorded, as the measured pore water pressures rapidly equalised to higher values, minutes after the end of jacking. Thus, the calculated ratios of σ'_r/p'_{cs} can be considered an upper bound, particularly below the 4 metre level. Nevertheless, three limiting ratios have been marked on Figure 9.40 which all correspond to failure conditions with $\sigma'_r = \sigma'_1$, and $\phi' = 23.5°$, but with different permutations of σ'_2 and σ'_3. Near the pile tip, σ'_r/p'_{cs} approaches and exceeds limit No. 3, which corresponds to the largest possible ratio for these assumptions. Above 3.8 metres depth, σ'_r/p'_{cs} falls below limit No. 2, which corresponds to plane strain cavity expansion, and for the entire upper half of the shaft the plot suggests that it is not possible for σ'_r to be the major principal effective stress. This simple treatment is approximate, but suggests several points of agreement with the strain path solutions for Boston Blue clay at low OCR's, and the analysis made in Chapter 3 of the results obtained by Francescon (1983) in his model pile experiments.

9.14.7 With time, the divergence between the measured stresses and the predictions of the cavity expansion analyses become more pronounced. In place of the expected rapid increases in σ'_r the experimental data consistently show reductions in the radial effective stresses, which continue for at least 600 hours.

9.14.8 Complex patterns of pore water pressure migration are observed after installation, with a short term swelling phase being followed by slow consolidation. It is probable that the continuing dissipation led to some recovery in σ'_r before the tension load test, but these changes can only be guessed.

9.14.9 The duration of the equilibration period was far longer than expected, and this phenomenon is worthy of further discussion. The London clay oedometer tests reported in Chapter 7 gave coefficients of consolidation for compression and swelling, and for OCR's between 3 and 7 the vertical coefficients could be bracketed between 0.2 and 0.8 m^2/yr. As the pile installation would have severely disrupted any horizontal layering close to the piles, the median of these values might be considered appropriate for a first analysis of the pore water dissipation. The consolidation analyses discussed in Chapter 3 showed that the times required for large degrees of consolidation are relatively insensitive to the initial conditions. It is therefore reasonable to use the curves given on Figure 3.27 to estimate t_{90}, even though they were derived from cavity expansion theory. Adopting the initial Au/Cu as 4, a time factor of 50 can be scaled from the figure, which gives t_{90} as 50 r^2_o/C_v, or 0.25 years. The calculation therefore predicts that after 3 months the average excess pore pressures at the pile face should equal 0.4 Cu(o). In fact, the measured excess pressures after $3^1/_2$ months ranged between 0.05 and 0.57 of the respective initial undrained shear strengths.

9.14.10 It is useful here to refer to the comprehensive studies of Walbanke (1976) who carefully installed hydraulic piezometers at several London clay sites. Her measurements showed that augering a 100 mm diameter bore hole radically alters the local pore water pressures, and that a period of months or even years can be required to reach 95% equalisation. Similarly, Tedd and Charles (1981) report slow decays in total radial stress after installing push in spade cells at Canons Park, and these also took several weeks to study. In summary, it appears that equalisation times for overconsolidated clays may be far longer than is commonly appreciated.

9.14.11 The effective stress conditions prior to loading are difficult to discuss, as the radial total stresses are not known. Noting the relatively small values of σ'_r acting after 600 hours, and the limited degree of subsequent dissipation before load testing, it is reasonable to conclude that the soil remained overconsolidated throughout. For the case of elastic swelling and recompression, Randolph et al. (1979) show that σ_r will remain constant at the pile face with $\Delta\sigma'_r = -\Delta u$. The assumption of elastic behaviour might be appropriate here, and would indicate respective increases in σ'_r of 115, 25 and 20 Kpa at the 5, 4 and 3 metre levels.

9.14.12 The restricted data available from the tension test show only small changes in pore water pressure before failure, perhaps as a result of a time lag between the shearing soil and sensing transducers. The data are consistent with the loading analyses of Potts and Martins (1982), but are difficult to consider quantitatively. For the three instrumented levels to satisfy the failure criterion $\tau_{rz} = \sigma'_r \tan \delta'$, certain values σ'_r would be required. Adopting $\delta' = 20°$ from the ring shear tests, Table 9.2 lists the values of σ'_r and compares these with stresses projected from the 600 hour stage, by assuming elastic behaviour during dissipation.

9.14.13 There are clearly substantial differences and, accounting for the observed changes in pore water pressure during loading, increases in σ_r of between 131 and 58 Kpa are required to balance the equation. It is feasible that such total stress changes occurred during loading or consolidation. Elevated values of δ' are also plausible for overconsolidated soil with a pre-existing shear surface, provided it has been sheared in a turbulent manner. Unfortunately, the absence of measurements reduces this discussion to speculation, and the hypotheses can neither be proved or discounted.

9.14.14 In contrast, the axial load data are unequivocal, and show larger shaft frictions than the previous tests of Price and Wardle, or Kitching. The results show the effects of the prolonged dissipation and reinforce the conclusion that equilibrium is established slowly after installation. The load transfer curves indicate a progressive

failure mechanism that probably originates in the relatively high compliance of the axial load cells. This observation suggests a way of studying the development of progressive failure in future tests by substituting cells of varying stiffness into the instrumented pile assemblies.

CHAPTER 10

PREDICTIVE ANALYSES OF THE MAGNUS FOUNDATIONS

10.1 INTRODUCTION

10.1.1 Previous chapters have discussed the elements required for realistic analyses of the foundations of piled offshore structures. It is the purpose of this chapter to synthesise these components in order to make predictions for the Magnus Production platform. The first part of the chapter describes the idealisations made for the finite element calculations regarding soil stratification, initial stresses, soil models and interface conditions. The second main section reports the results of the analyses and discusses their implications. In the final part predictions are compared with both full-scale field measurements and the results of a conventional analysis reported by Rigden and Semple (1983).

10.2 SOIL CONSTITUTIVE LAWS

10.2.1 Chapter 3 reported parametric studies of the effective stress changes that accompany pile loading, and a strong dependency was noted on the assumed constitutive relationships. Martins (1983) considered that models of the Modified Cam Clay (MCC) type were most appropriate for pile-soil analysis, provided that a variant was chosen that gives consistent results under plane strain conditions (see Appendix A2).

10.2.2 However, the studies of load-displacement behaviour set out in Chapter 8 emphasized the requirement of considering non-linear stress-strain characteristics within the stress space considered as elastic in Modified Cam Clay. It was shown that the assumption of a linear shear modulus contradicts the large body of field and laboratory data, and is likely to confuse the prediction of load-settlement curves.

10.2.3 The soil model employed for the Magnus analysis combines the well-known features of modified Cam Clay with the type of empirical pre-yield formulation discussed in Chapter 8. The virgin consolidation

line, yield locus, angle of friction and flow rules are governed by the equations of Roscoe and Burland (1968) and a Hvorslev surface is incorporated on the dry side. Within the 'elastic' region, non-linear shear and bulk moduli are specified by two similar secant equations;

$$\frac{G}{p'} = \frac{E_u}{3P'} = A + B\cos\alpha \quad \log_{10}\left[\frac{E}{\sqrt{3}.C}\right]^{\gamma} \qquad \text{Eq. 10.1}$$

$$\frac{K'}{p'} = A' + B'\cos\alpha' \quad \log_{10}\left[\frac{|\varepsilon_v|}{C'}\right]^{\gamma'} \qquad \text{Eq. 10.2}$$

(where $|\varepsilon_v|$ is the absolute value of ε_v)

10.2.4 As described in Chapter 8, the various constants in Equations 10.1 and 10.2 can be readily found from experimental curves relating stiffness to log strain, and the derived secant equations may be re-expressed without difficulty as tangential equivalents. It will be recalled that tangent formulations are more suitable for use in non-linear finite element calculations. Minimum and maximum strains also need to be specified to give limits beyond which fixed elastic stiffnesses are specified.

10.2.5 In principle, the curve fitting routine set out in Appendix A5 could be used to derive shear stiffness parameters that vary with depth and radius from the pile, with account being taken of changes in soil composition, OCR and the effects of pile installation. However, the results presented in Chapters 6 and 8 suggest that the problem may be simplified considerably. Regarding the parameter describing the scale of stiffness, $(E_u/p'_o)_{0.01}$, it was shown that K_o consolidated samples tested in compression gave ratios that fall within a surprisingly narrow band, and that a value of 2,000 was typical, except for unaged normally consolidated samples. For this special case a relatively low stiffness ratio was noted although the modulus in extension was unusually high. The results of recompression tests on pre-extended samples, the tests on sampled specimens, and experiments on isotropically consolidated clay further

suggest that the effects of installation and reconsolidation will not greatly alter the typical value of $(E_u/p'_o)_{0.01}$.

10.2.6 Triaxial compression tests impose a particular stress path, and there is no a priori reason why the deduced stiffness data should be exactly compatible with the shear stiffness characteristics of anisotropic soil deforming around a vertically loaded pile. However, curves of isotropic shear modulus against invariant shear strain deduced from undrained K_o compression tests were found to give good agreement with soil characteristics interpreted in a similar way from pile test data, at least for London clay and low plasticity glacial tills.

10.2.7 The triaxial data reported in Chapter 6 showed the degree of non-linearity to vary with many factors, but the characteristic provided by an undrained K_o compression test from OCR 2 was found to give a good median to the observed range. Indeed, the results justify the use of a single set of parameters to describe the pre-yield variations in G/p'_o with octahedral shear strain, for all the clay layers in the Magnus profile.

10.2.8 In a similar way, it was decided to use a single set of parameters, derived from the data presented in Figure 6.60, to describe the normalised variation of bulk Modulus with volume strain. In fact, the Magnus analyses all assume undrained conditions with $\varepsilon_v = 0$ and Equation 10.2 is almost superfluous.

10.2.9 It must be emphasised that the non-linear stiffness expressions are a simple empirical way of reproducing the response of the soil to monotonic loading starting from particular initial conditions. The formulations take no account of the kinematic nature of the small strain zones, which can be translated and modified when the shear or volume strain increments exceed limits of around 0.05%. The described model therefore needs to be used carefully in cases where the stress path directions can reverse after attaining large strains. Indeed the simple non-linear stiffness relationships could become invalid in certain circumstances.

10.2.10 In order to check the ability of the composite "Magnus clay" model to match the experimentally observed behaviour, a series of finite element analyses was carried out to reproduce the following laboratory element tests; continuous one dimensional loading of σ'_v from 100 to 400 Kpa, K_o unloading over the same range and undrained triaxial compression tests from K_o conditions at OCR's 1.1,2 and 8. The various parameters for the model are listed in Table 10.1.

10.2.11 Considering the oedometer simulations first, the K_o loading was applied by specifying increments of σ'_v with $\varepsilon_r = 0$ from initially normally consolidated conditions with $K_o = 0.5$. Figure 10.1 plots the computed 1-D loading stress path in $(\sigma'_A - \sigma'_r)/2 - (\sigma'_A + \sigma'_r)/2$ stress space, and the diagram shows how the path deviates from the assumed stress ratio and rapidly conforms to a flatter line, with $K_o \simeq 0.68$. This result is a consequence of the flow rule and yield function adopted for Modified Cam Clay, and reference has already been made in Chapter 3 to the number of ways in which MCC does not match the detailed behaviour of natural soils consolidated under K_o conditions. The e - log σ'_v diagram presented in Figure 10.2 confirms that the addition of the pre-yield non-linearity does not effect the one dimensional virgin consolidation line.

10.2.12 The swelling phase of the oedometer test was simulated by resetting the strain terms in Equations 10.1 and 10.2 to zero, adopting the initial stresses calculated for the end of the loading stage, and reducing σ'_v without permitting any radial straining. The resulting plots for stress-path and stress-strain are shown on Figures 10.1 and 10.2. The first figure also shows axial strains at appropriate levels, and may be compared with the path and strains of the swelling stage of a typical K_o consolidated test on Magnus clay. It is encouraging to note that the directions of the swelling stress paths are almost identical, although the initial stresses differ as a result of the K_o ratio predicted by Modified Cam Clay. The curvature of the computed path shows that the ratio of K'/G at any particular point is realistic, and that the overall 'isotropic' Poissons ratio is varying in an appropriate manner. The second point to note is the remarkably similar spacings of strain increments on the experimental and

theoretical stress paths. The swelling e-log p' line indicated on Figure 10.2 also compares well with the experimental data plotted on Figure 6.13. Although it is not clear to the eye, the plotted e - σ'_v relationship is far from log - linear. This reflects the steep changes in bulk modulus with volume strain, and realistically models the patterns of the oedometer data presented in Chapter 6.

10.2.13 The undrained compression tests were predicted by specifying initial stresses similar to those adopted for the experiments. Loading was applied by axial strain control with 50 gradually increasing increments; the first step was to 0.001%, the last from 19 to 20%. As the experimental samples were allowed to rest at the maximum axial stress for 2 to 3 days, the smallest OCR to be considered was 1.1, and this gave some allowance for ageing and secondary compression.

10.2.14 The predicted stress paths are shown in Figure 10.3, and strain contours have been added to assist comparison with the experimental data plotted on Figure 6.17. Regarding these two sets of results, the following points are of interest;

(i) The general patterns of volumetric behaviour after yield are reproduced by the model, at low OCR the stress paths move sharply to the left and at high OCR's strong dilation is seen.

(ii) The predicted pre-yield strain contours give good agreement with those measured.

(iii) The model overpredicts ultimate strength in each case. For OCR's 1.1 and 2, the critical state values are almost exactly equal to the measured peak strengths, but for OCR 8 the predicted critical state C_u considerably exceeds the experimental value.

(iv) Equally, the model is unable to reproduce the observed undrained brittleness, and shows yield at an unrealistically small axial strain for OCR 1.1

10.2.15 The element tests described above show that the Magnus clay model reproduces most of the essential features observed in the triaxial tests. To improve the description to include undrained brittleness, kinematic relocation of the small strain zones, or stiffness anisotropy would require the derivation of a far more complex model. Whilst Kavvadas (1982) and Gens (1983) have shown how this might be attempted, it is beyond the scope of the present project to develop such constitutive laws. The overprediction of Cu for high OCR clay can be overcome by using an option within program ICFEP to specify profiles of Cu and appropriate initial stresses. The ICFEP calculation of compatible distributions of OCR is then carried out automatically.

10.2.16 In summary, the 'Magnus Clay' model gives a greatly improved description of the observed behaviour in comparison with linear elasticity, Modified Cam Clay, or the simpler non-linear model described in Chapter 8. The essential features seen in undrained compression tests are successfully reproduced and good predictions are found for the constrained case of K_o swelling. Great care must be taken in situations where stress paths reverse after attaining large strains; if the strain terms substituted into Equations 10.1 and 10.2 are not adjusted, the important changes in stiffness noted in experiments will not be reproduced.

10.3 STRATIFICATION

10.3.1 The earlier sections of Chapter 6 considered the sequence of soils encountered at the Magnus site, and divided the profile into five main geotechnical units. These layers form the basis of the idealised stratification used in the finite element analyses, and the values of v, λ, k and γ' given in Table 10.2 were assessed from the test data presented in Chapter 6. Within each stratum it was possible to specify variations of Cu, OCR or initial stresses with both depth and radius, and the values chosen will be discussed in the following section. It will be noted that the two thin sand layers within group I have been considered as low plasticity clays, and little error is likely to accrue from this simplification.

10.3.2 To ensure that the finite element calculations were not artificially constrained by a fictionally rigid base, the lower limit to layer V has been taken as 170 metres below sea bed; i.e. to approximately double the pile length. In fact the site investigations had proved quaternary clays to a depth of at least 240 metres.

10.4 INITIAL CONDITIONS WITHIN THE CLAY LAYERS

10.4.1 Three main analyses will be presented for the Magnus foundations. Two consider the pile groups when loaded six months after installation; the third assumes that loading takes place 10 years later, when the soils are near to complete equilibrium. The first pair of studies attempt to reproduce the characteristics developed towards the end of the construction period, when the installation effects were likely to have been incompletely equalised. The two 'early' analyses differ only in the assumption of conventional critical state behaviour at large strains in the first case, and the specification of residual fabric at the pile-soil interface for the second.

10.4.2 Data reviewed in Chapters 3, 6 and 9 were used to estimate bulk values for the consolidation coefficients governing the equilibration process, with account being taken of the likely variations with depth of permeability and swelling modulus. The values of C_{vs} are listed in Table 10.3, and time factors, $T_v = t.c_v/r^2_o$, are evaluated for the cases of six months and 10 years after installation. As a guide to the degree of equalisation, percentage dissipations at the pile face have also been computed from the cavity expansion, linear consolidation, solutions reviewed in Chapter 3. The estimates suggest a slow rate of equalisation in the strong upper layers, and incomplete dissipation in all strata at the six month stage. It is encouraging to note that the inferred rate of consolidation for the two low OCR groups is similar to that measured by Sutton et al. (1979) over a short interval at the Forties FD Structure.

10.4.3 The step from these approximate estimates for rates of equalisation to make an evaluation of the effective stresses developed around the piles is more difficult. The reviews presented in Chapter

3 and the Canons Park experiments can be used to help bracket these stresses, but the results can only be considered approximate. The clearest guidance exists for the assessment of the pile face radial effective stresses in the lightly overconsolidated layers. More limited data exist to help select values of σ'_r in the high OCR strata, and Table 10.4 summarizes the ratios of σ'_r to initial radial effective stress adopted for the two stages considered.

10.4.4 In the case of low OCR clays, Kavvadas and Baligh (1982) give predictions of the complex variations of σ'_r, σ'_θ and σ'_z with radius at various times after installation. Their work predicts overall reductions in p' near the pile face after full equalisation and the data reviewed in Chapter 3 suggested that it would be unwise to simply adopt these distributions as the initial conditions for the full pile analysis. However, a starting point might be to accept the theoretical finding that $\sigma'_z > \sigma'_r > \sigma'_\theta$ for clay at low OCR. With higher degrees of overconsolidation this order might change so that σ'_r could remain the major principal effective stress.

10.4.5 Further guidelines may be established by limiting the extent to which the final σ'_z value can exceed the effective overburden pressure. Near to the shaft, increases in vertical effective stresses can be balanced by shear stresses on vertical planes and reductions in σ'_z in intermediate areas further from the pile; indeed both the cavity expansion and strain path solutions indicate large gains in σ'_z at the pile face. Randolph and Wroth (1982) later assumed that σ'_z returns to the effective overburden pressure when the pile is loaded, but this arbitrary reduction seems somewhat severe. For the following analyses it will be assumed that $\sigma'_z \leq 2.0\ \sigma'_{zo}$ at the pile face, this helps reconcile the experimentally inferred increases in p' and the published measurements of σ'_r.

10.4.6 The available data regarding changes of water content and strength close to driven piles were discussed in Chapter 3. It was suggested that, near to the shaft, p' could rise to 1.5 times its initial value for piles installed in normally consolidated clay; even larger margins were implied for heavily overconsolidated samples. In

order to span the difference between the implied gains in mean stress, and the reductions predicted by Kavvadas and Baligh (1982), limits to p'_e/p'_o were selected as 1.25 for OCR 1, grading to a higher ratio of 1.8 for the layers with greatest initial OCR. In addition to the constraints on σ'_r, p' and σ'_z it is also necessary to ensure that the assumed ratios of σ'_3/σ'_1, do not violate the Mohr-Coulomb failure criterion. As both the strain path and cavity expansion analyses suggest post-consolidation stress ratios close to 1 - sin ϕ', it was further decided to limit σ'_3/σ'_1 to between 0.9 and 0.45.

10.4.7 Applying these four constraints gave surprisingly little latitude when estimating the normalised values of σ'_z, σ'_r and σ'_θ listed in Tables 10.5 and 10.6.

10.4.8 Potts and Martins (1982) demonstrated that the orientation and relative magnitude of the initial stresses at the shaft face can have an important influence on the effective stresses developed when the pile is loaded to failure. Further from the pile these details are less significant, as the stress state remains within the elastic region. In the bulk of the soil mass, the moduli depend on p' and strain alone, and it thus is only necessary to ensure appropriate distributions of mean effective stress. The various analyses of pile penetration suggest that the effects of installation are contained within a region of radius 20 r_o, and it was therefore decided to adopt log-linear radial variations of σ'_r, σ'_z and σ'_θ from the pile face out to K_o ground stresses 20 radii from the shaft centre. No initial shear stresses were imposed on vertical or horizontal planes, and a further simplification was made by retaining the undisturbed K_o stresses for all soil below the level of the pile tip.

10.4.9 The final parameter to be specified was the degree of overconsolidation. Within the program ICFEP, this could be done by either imposing a fixed OCR, or by stating the undrained shear strength for each particular layer. For the analysis of the pile 6 months after installation the shear strength-depth variation given in Figure 10.4 was specified. The profile was derived from the data presented on Figure 6.73 and only differs from the undisturbed profile between 50

and 76 metres depth, where an increase of approximately 10% was assessed at the pile face. The OCR-depth profile calculated by ICFEP for soil at the pile shaft and under the pile tip is plotted on Figure 10.5. Normal consolidation was assumed over only the lower third of the pile, and for this section the undrained shear strengths showed slight variations with radius.

10.4.10 To carry out an analysis of the foundations 10 years after installation, it was necessary to modify the profiles to account for further consolidation. The normally consolidated zone was assumed to extend from 40 to 76 metres, and the shear strengths over the remaining sections of the pile to have increased by 10%. Combined with the increases in assumed soil stresses, these changes give rise to the Cu distribution and OCR-profile plotted on Figure 10.6.

10.4.11 The finite element mesh designed for the study is shown on Figure 10.7. A special feature of the mesh is the fineness of the elements close to the pile wall. The tubular steel pile was considered as a cylindrical solid body with an equivalent Young's modulus of 24 x 10^3 Mpa. This value was calculated from the pile radius and wall thickness given by Semple and Rigden (1983); the Poissons ratio was taken as 0.3.

10.5 DESCRIPTION OF RESIDUAL STRENGTH CHARACTERISTICS AT THE PILE-SOIL INTERFACE

10.5.1 Three full-scale analyses were made of the Magnus foundations. In the first, Run A, no attempt was made to simulate residual fabric developed at the pile soil interface. For Runs B and C, ammendments were made to the coding of program ICFEP to allow limiting values of the ratio tan $\delta' = \tau_{rz}/\sigma'_r$ to be specified in the finite elements close to the pile. The criterion was particular to these two components of stress and permitted greater ratios of shear to normal effective stress on other planes, provided that the overall continuum value of ϕ' was not violated. The program was altered so that if any increment of pile displacement resulted in the limiting ratio being exceeded at a given Gauss point, the local value of τ_{rz} was

reduced to match the current value of $\sigma'_r \tan \delta'$. Any excess in τ_{rz} was then redistributed in further iterations of the analysis. The procedure is analagous to that used in "no tension" finite element analyses, Potts (1985).

10.5.2 The adopted procedure results in a redistribution of shaft friction that allows equilibrium to be restored without fixing the absolute values of σ'_r or τ_{rz}. Indeed, both quantities are free to change as the global stress regime adjusts to continuing loading, only the maximum ratio between the two stresses is specified. In order to reproduce the ring-shear behaviour as realistically as possible the interface values of tan δ' were described as functions of relative displacement[1], as shown in Figure 10.8. The criteria for the four soil layers were assessed from the ring shear data presented in Figures 6.84 to 6.88, and were intended to simulate shearing at a rate typical of pile load testing.

10.6 PREDICTION OF FOUNDATION RESPONSE SIX MONTHS AFTER INSTALLATION

10.6.1 In this section the results obtained in Runs A and B will be discussed. Each computation produced very large quantities of information and only summary graphs and plots for representative points can feasibly be presented. The data obtained for the axial loading of single piles will be extended to cover group behaviour using the approximate procedure outlined in Chapter 4. It is worth repeating here that Run B only differs from A in the provision of interface residual strength characteristics.

10.6.2 The results of the two simulations are first summarized in Figure 10.9. In this diagram the full load-displacement curves

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[1] Within program ICFEP it was necessary to express the displacements as equivalent shear strains γ_{rz}. The conversion was made by dividing by the half element width.

corresponding to slow, undrained, testing are reproduced, and in each case the data were obtained by imposing around 100 increments of pile head settlement. At the left hand edge of the figure two conventional estimates of the ultimate single pile capacity are indicated for reference. The smaller capacity is that calculated by Rigden and Semple (1983) in accordance with the API rules, and the larger total was obtained by integrating the shear strength profile given in Figure 10.5, and assuming respective values of 1.0 and 9.0 for α and N_c.

10.6.3 Regarding the finite element load displacement predictions two points are immediately apparent;

(i) In the medium term the capacity of the pile is critically dependent on the existence, or otherwise, of a residual shear surface. Neglecting the effect of installation on interface fabric leads, in this case, to the prediction of a capacity three times greater than that calculated by the API procedure[1]. The specification of laboratory interface ring shear characteristics at the interface results in a calculated axial capacity that is only slightly greater than that estimated by the routine, $\alpha = 0.5$, method.

(ii) From an early stage the load-displacement relations are influenced by the assumptions made for the interface strengths. The curve for Run A continues to rise steadily until a maximum load is obtained after 300 mm settlement. In contrast, Run B shows a near maximum capacity when the displacement reaches 40 mm, and shows a distinctly softer response than Run A for loads greater than 20 MN (i.e. around 30% of the API capacity).

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1 In principle, Run A should be compatible with $\alpha = 1.0$, but as Cu varies with e in the employed variant of the Cam-Clay model, shear strengths specified for the initial stress conditions may vary from those ultimately developed at the pile face.

10.6.4 Comparisons between the results obtained in the two medium term analyses can be usefully made in a number of further plots. Traces are given on Figures 10.10 and 10.11 to indicate the profiles of shaft adhesion for particular values of pile head settlement. The distributions of τ_{rz} at 2 mm displacement are virtually identical, but at larger displacements divergences become clear. Considering the 30 and 35 mm settlement plots, Run B showed a more even distribution of τ_{rz}, whilst Run A indicated a much greater degree of mobilisation in the upper layers. At larger displacements Run B showed considerable strain softening below the 40 metre level, but Run A showed the upper hard overconsolidated layers to provide continuing resistance to increments of displacement.

10.6.5 To examine the trends in more detail it is helpful to consider the development of interface stresses at four Gauss points approximately positioned at the mid-depths of layers 1, 2, 3 and 4. These points are indicated on Figures 10.10 and 10.11, and graphs showing the development of the local values of τ_{rz} are presented on Figure 10.12. For the first, non-residual strength, analysis failure first occurs in layer 2, but only after a pile head settlement of 55 mm has developed. The failure propagates rapidly and layers 3 and 4 follow relatively quickly, but much larger displacements are required to fail the uppermost layers. This system is quite different to the mechanism of failure propagating from the pile head that has been assumed in simplified analyses of progressive failure by Murff (1980) and other workers.

10.6.6 The main cause of the effect is apparent when the stress paths plotted in Figure 10.3 are reviewed; large local shear strains are required to fail the heavily overconsolidated clay as a continuum, and so peak strengths are first mobilised in the lower OCR strata. The residual strength characteristics assumed for Run B restore the more familiar pattern of failure, with a progressive mobilisation of shaft capacity from pile head to base tip. The combination of effective stress changes and reductions of δ' with displacement produce marked post peak reductions in τ_{rz} for Layers 3 and 4. Layers 1 and 2 show slight changes in τ_{rz} after reaching peak values, with small gains for Layer 2 and reductions for Layer 1.

10.6.7 The observed variations in τ_{rz} are associated with changes in (i) the radial effective stresses acting over the pile surface and (ii) the frictional strength at the interface. Figure 10.13 plots the curves of σ'_r against pile head settlement obtained at the four representative points during Run A. It is clear that the assumption of continuum behaviour leads to the prediction of large changes in σ'_r for most of the strata. In each case the normal effective stress falls by around 10% to reach a minimum, and then climbs until an ultimate condition is reached. Dilation in the upper layers causes a many fold increase in σ'_r, but for the contractant Layer 3 the total gain in effective stress is less 10%.

10.6.8 The inclusion of a residual characteristic in Run B appears to suppress the above described variations in radial effective stress. Figure 10.14 shows the calculated traces for σ'_r, and only small reductions are observed at any of the representative points. However, it is interesting that the radial effective stresses show slight changes after peak conditions have been mobilised; in most cases small reductions are noted, but for Layer 2 significant gains are apparent.

10.6.9 A more complete picture of the stresses developed over the pile face is provided by the effective stress paths and pore water pressure plots for the layer mid-depth points. These data are presented in Figures 10.15 to 10.18, and the effective stress paths for Run A will be discussed first. The invariant co-ordinates J and p' are used for convenience, but it should be noted that a fixed Mohr-Coulomb angle ϕ' does not imply a single limiting ratio of J/p'. In fact, this slope varies according to the current value of the lode angle, θ, or the intermediate principal stress parameter, b. (This point is further discussed in Appendix A2). Regarding the stress paths it is evident that, without a residual surface, strong dilation occurs in all layers except the normally consolidated stratum, layer 3. The stress paths climb onto Hvorslev surfaces, but as θ and b change the traces plot as curves, rather than straight lines. The same dependence of J/p' on θ causes the contractant behaviour of Layer 3 to resemble strain softening, even though ϕ' and Cu do not fall with

increasing displacement. All four stress paths ultimately reach critical state conditions on a line corresponding to $\phi' = 29°$ for $\theta = 0°$ (i.e. b = 0.5).

10.6.10 Marking the values of pile head displacement on the stress paths gives further insights into the progressive mobilisation of shaft friction. Layer 2 is the first to yield, followed by Layer 3, and peak capacity is reached for these two strata before either of the two remaining layers starts to dilate. The increases in shaft resistance after 60 millimetres displacement are mainly due to the tenacity of the hard upper layers. The stress paths followed at the same points in Run B (Fig. 10.16) show quite different patterns. The dilatant behaviour of Layers 1, 2 and 4 is largely restricted by the simulated residual surface, although Layer 3 shows a notable degree of contraction. Layer 3 is now the first to yield, followed by 1, 2 and finally 4.

10.6.11 Figures 10.17 and 10.18 plot the pore water pressure changes anticipated for settlements up to 180 mm at the typical points. For Run A large variations are calculated in each layer. Considerable reductions are found to accompany the attainment of peak local shaft capacity in all cases except Layer 3, which does not fail until the settlement reaches 300 mm. In the other layers continuing post peak displacement induces even greater reductions in the pile face pore water pressures, and the final values are shown on the right hand side of Figure 10.17. As the effective stresses are constant in Layers 2 to 4 after 85 mm displacement, the graphs infer that the local total stresses fall rapidly as the stress regime accommodates the developing failure mechanism. With Run B the pore pressure changes up to peak local capacity are far smaller and, except for Layer 4, remain positive. After reaching peak conditions the lowest layer shows rapid reductions in pore water pressure, possibly as a result of the punching failure developing at the tip. Conversely, the normally consolidated soil produces considerable positive pore pressures.

10.6.12 It was shown in Chapters 4 and 8 that the surface settlement profiles deduced from single pile analyses can be useful in considering

pile group interaction. Figures 10.19 and 10.20 plot pairs of profiles at appropriate settlements for Runs A and B, and the graphs show the expected pattern of surface settlement being concentrated close to the pile shaft with the trend becoming progressively more accentuated as failure is approached. At small settlements the two analyses yield similar profiles, but the tendency at higher displacements is for Run B to indicate steeper local displacement gradients near the pile face and smaller surface interaction factors for points a few radii from the pile.

10.6.13 The use of a simplified procedure allowed the group axial load displacement relationships to be estimated from the single pile analyses. The first step was to idealise the Leg A4 foundations as an equally spaced circular group of 9 piles. Taking the circle radius as 6.88 metres, inter-pile distances were calculated and, by assuming superposition, group settlements assessed as the sum of the interacting components of displacement. The results of calculating the group displacements for Runs A and B at several loads are shown in Figure 10.21. The curves show total group displacements which are initially around 1.5 times those for single piles loaded to the same average load. As failure is approached the group displacements become proportionately closer to the single pile settlements.

10.7 PREDICTION OF FOUNDATION RESPONSE TEN YEARS AFTER INSTALLATION

10.7.1 The rough calculations set out in Section 10.4 suggested that a period of several years would be required for the soil mass to reach final equilibrium after pile installation. The study outlined in this section predicts the performance of the piles from the fully equalised state, and assumes the stress conditions set out in Tables 10.4 and 10.6. Again, undisturbed K_o stresses were specified below the pile tip, and at radial distances greater than 21 metres from the pile shaft. A discussion of the adopted initial conditions was included in Section 10.4, and need not be repeated here. The importance of considering residual strength properties at the pile-soil junction is demonstrated by the foregoing analyses, and it is therefore

appropriate that the analysis made for fully equalised conditions should retain the residual interface characteristics specified for Run B.

10.7.2 The results obtained are first summarized in Figures 10.9 and 10.21, and these show the single pile and group load-displacement curves. In comparison with Run B, it can be seen that full equalization leads to an increase in capacity by almost 50%, with a significant stiffening of the load deflection characteristics. However, the residual shaft friction angles ensure that the computed peak capacity remains far below that indicated by Run A.

10.7.3 Further insight is given by the skin-friction profiles plotted on Figure 10.11. Run C produces a τ_{rz} distribution that can be seen as a scaled equivalent of Run B, and is also quite different to that calculated for Run A (see Figure 10.10). An interesting point is that Run C virtually reaches its maximum capacity after a displacement of around 60 mm, whilst Run B showed full mobilisation after around 40 mm settlement. This feature allows a revealing contrast to be made between the two τ_{rz} distributions noted at 35 mm displacement. At this stage Run B was showing failure over almost the entire pile length, but in Run C the shaft friction was only fully developed above point 3. Failure in the lower layers develops as an advancing front, below which there are steep gradients in τ_{rz}.

10.7.4 Plots of shear stress against settlement for the 'mid-layer' points are given in Figure 10.12. These emphasise the parallels between Runs B and C, and help to illustrate the slight disparities in rates of skin friction mobilisation in the various layers. Subtle differences are also apparent in the traces of σ'_r with displacement shown on Figure 10.14, although Run C similarly predicts relatively minor changes in σ'_r during pile loading.

10.7.5 The effective stress paths followed by the four representative points are plotted using J, P' co-ordinates in Figure 10.22. As might be expected, the trends are reminiscent of those found for Run B (see Figure 10.16). The only significant difference

is the apparent lack of contraction calculated for Layer 3. The variations in pore water pressure for Run C are illustrated in Figure 10.23; close compatibility with Run B is again apparent. (See Figure 10.18).

10.7.6 Finally, the distributions of surface settlement from Run C for two values of pile head displacement are reproduced on Figure 10.24. The computed radial profiles show ground movements to be closely concentrated round the pile shaft, although in a slightly less pronounced way than those for Run B at the same pile head settlements. These data were used to generate the group load displacement curves of Figure 10.21.

10.8 CONCLUSIONS

10.8.1 The finite element analysis of the Magnus pile groups considered the slow undrained loading of single piles six months and ten years after installation. These predictions give many insights into the development of the platform's overall foundation response. This section lists the conclusions made possible by the studies and makes comparisons between the numerical predictions, results obtained by conventional design procedures and some limited data from field instrumentation.

10.8.2 The most important conclusions may be summarized as follows;

(i) When realistic soil models are used in numerical continuum solutions, but no allowance is made for residual fabric at the pile-soil interface, very large displacements are required to fail the overconsolidated upper layers. This prediction is at variance with the results of onshore pile tests on glacial tills (see Chapter 8).

(ii) If the interface characteristics seen in laboratory ring shear tests are modelled in the computations, more appropriate displacements to failure are predicted.

(iii) The "residual strength" analyses show capacity to be governed by the initial radial effective stress distributions and the permissible range for the skin friction angle, δ'. The estimates made of these quantities could be considered to be conservative; relatively low σ'_r values were assumed near the surface and no allowance was made for the possible effects on δ' of high speed installation or post driving overconsolidation in the upper layers[1].

(iv) In the medium term the overall computed capacity from Run B was slightly larger than the 'API design' summed resistance assessed by Rigden and Semple (1983). The computations showed that a far larger capacity could be expected to develop over a period of years. If faster rates of dissipation were to occur in reality, then these gains would be available at an earlier stage.

(v) The "residual strength" analysis showed that the initial shear strength profile had little influence, per se, on the maximum available skin frictions. Indeed, the local values of alpha calculated in the upper hard clays were far smaller than those for the deeper, more normally consolidated, sections of the profile. This mainly resulted from the assumed initial distribution of σ'_r on the pile shaft, as the inferred ratios of σ'_r/Cu generally increased with depth. As discussed above, the parameters selected for the upper clays may have been overconservative.

(vi) The inclusion of realistic residual strength characteristics drastically altered the pattern of stresses developed at the pile face. Strain softening was noted in the skin friction characteristics and the radial

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[1] It was argued in Chapter 6 that these two processes might lead to higher values of δ'.

effective stress and pore water pressure changes were seen to be considerably muted.

(vii) The pore water pressures acting over the shaft were found to vary with the degree of shaft friction mobilisation and pile settlement in a complex way, and do not give easily interpreted information on the effective stress conditions at the pile-soil interface; this conclusion is reinforced by the field test data described in Chapter 9.

(viii) The single pile analyses indicated pre-failure load-displacement curves and surface settlement profiles that differed greatly from those predicted by linear elasticity. These two features lead to group load-settlement curves which are much steeper than predicted by more conventional calculations; see Figure 10.21.

(ix) The initial portions of the load settlement curves were weakly influenced by the pile-soil failure criteria. Under design loading conditions, however, the overall displacement response is highly dependent on the assumptions made at the interface.

10.8.3 The accuracy of some of the numerical predictions can be assessed by making comparisons with the field measurements of the Magnus Foundation Monitoring Project (F.M.P.). Areas of interest include settlement gauge readings, vertical accelerometer data and shear stress distributions obtained from pile strain gauge stations. However, the writer is only aware of the early settlement gauge[1] data, which was reviewed in a report to British Petroleum by Jardine (1982). This document described the performance of the devices at the Magnus site.

10.8.4 The interpretation of the field measurements was made difficult by a series of mishaps on site, and there is evidence that

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[1] The author's work in developing the offshore instrumentation is fully reported in Appendix A6.

the instrumentation suffered from the timing and method of installation. It is also likely that the equipment suffered from the extreme energy of pile driving. Nevertheless, the recordings were interpreted as showing a settlement increment of approximately 2 mm occurring as top side modules with a total weight of 26,000 t were placed. Noting that the pile loads were unlikely to have been large before the modules were placed, comparisons should be made with the initial part of the plots given in Figure 10.21. The finite element computations can thus be used to assess the group compliance as 0.31 mm per MN average pile load. Assuming all 36 piles to be equally loaded, the additional topside weight gives an expected group displacement of 2.2 mm. The agreement between measurement and prediction is certainly encouraging, and confirmation from the storm loading accelerometer data would be even more rewarding.

10.8.5 The numerical results emphasize the significance of the key issues in the analysis of large piled foundations; initial conditions, effects of installation, the possible existence of residual fabric and the necessity of using realistic soil models to simulate pre-yield behaviour.

CHAPTER 11

PREDICTIONS AND MEASUREMENTS FOR THE FOUNDATION RESPONSE OF THE HUTTON TLP

11.1 INTRODUCTION

11.1.1 The second North Sea Platform for which predictions and measurements may be compared is the Hutton TLP, and the research programmes associated with this platform and the Magnus structure can be seen as complementary. Extensive laboratory investigations were made of the Magnus soils, but only limited comparative field data are currently available from the foundation monitoring project. Conversely, no laboratory work has been undertaken by the author with the Hutton soils, but high quality field information has been gathered using a suite of special foundation settlement gauges.

11.1.2 Chapter 11 is concerned with the predictions made for the TLP foundations, and also summarizes the load-displacement data gathered during platform installation. The first part of the chapter reports a numerical study which is similar to the long-term analysis made for the Magnus site in Chapter 10. Use is again made of the arguments set out in earlier chapters and the laboratory data obtained from the Magnus and London clay test programmes. Commercial site investigation data are re-interpreted and synthesized with the reported experimental findings to develop appropriate soil models, profiles of soil properties and distributions of initial stresses.

11.1.3 The second half of the chapter considers the observed full-scale behaviour of the foundations of the Hutton Tension Leg Platform. A brief account is given of the development by the author of high accuracy underwater settlement instrumentation and its installation on the foundation pile templates. The measurements obtained are then reported and comparisons are made with the finite element predictions. It is believed that these measurements represent the most comprehensive study yet of the load-displacement behaviour of a large piled offshore structure.

PART I

11.2 DESCRIPTION OF THE SOIL CONDITIONS AT THE HUTTON SITE

11.2.1 The soil conditions at the Hutton site were investigated by Fugro in the summer of 1979. A number of reports were issued detailing the field work and laboratory testing of soil samples; it may be recalled that Section 4.4 outlined a parametric study which started from a simplified soil profile presented in Fugro's summary report, UO 638. However, the conditions given in that document represented a conservative interpretation of the profile taken from the borehole with the lowest average shear resistances. For a predictive study it is more appropriate to consider typical conditions, and that is the aim of this section.

11.2.2 No geological interpretation of the sequence of strata was given in the Fugro report. The author was not able to make an inspection of the sample fabric and is not aware of any published supplementary information. However, given the platform's location, which is shown on Figure 1.1, and noting the results of the site investigations, it is probable that the proven layers of clay and sand were deposited as tills over the past 10,000 to 100,000 years. The frequent laminations in the clays, the presence of chalk, shell and other erratics, and the interbedding noted at the boundaries of many of the layers all point to a glaciomarine depositional environment. Indeed, there are many similarities between the Hutton borehole logs and those interpreted for the Magnus site. The principal differences are associated with the greater thicknesses of the sand layers at Hutton, and the higher plasticity indices noted for the clay strata between 14 and 60 metres depth.

11.2.3 The conditions at the Tension Leg Platform site are generally not uniform in lateral extent. Figures 11.1 to 11.3 show simplified bore hole logs, as interpreted by the writer, and it is clear that the sandy strata vary in thickness and number from one position to another. Figure 11.4 summarizes the grading distributions found from borehole

No. 1, and Figures 11.5 to 11.8 present the aggregated data concerning the variations of PI, γ_{bulk}, 1-D consolidation properties and undrained shear strength with depth. These plots were used to construct the summarized "typical" profile given in Figure 11.9. Account was taken in the interpretation of the data of the probably severe effects of sampling disturbance on the laminated clay layers. The conditions in the sands were mainly estimated from the insitu cone penetrometer testing.

11.3 DEVELOPMENT OF PARAMETERS REQUIRED FOR FINITE ELEMENT ANALYSES

11.3.1 It will be recalled from Chapter 10 that many parameters are required for the prediction of even solitary pile behaviour. It is necessary to specify the initial stress distributions in the soil layers and various strength-deformation parameters, including the yielding description and non-linear 'pre-yield' stiffness coefficients. It is also vital to consider the pile-soil interface residual strength characteristics and to be able to fix the equivalent elastic constants for the pseudo-solid cylindrical pile. Many of these parameters have not been directly measured or previously evaluated for the Hutton foundations, and the information has to be estimated by reference to the arguments developed and data assembled for the Magnus studies.

11.3.2 With such a reliance on the Magnus investigations, it is important to establish how closely the profiles at the two sites correspond. It has already been noted that the middle clay layer at Hutton is of typically greater P.I. (and indeed clay content) than the Magnus clays over the same depths. The Hutton sand layers are also thicker and more numerous than those proven at Magnus. Figure 11.10 compares the summarized 'average' shear strength profiles and these at least show encouraging agreement. Recalling that apparent OCR's for each depth may be deduced from the Cu/σ'_r variations, it is clear that the similar plots for strength imply comparable profiles of apparent overconsolidation ratio in the clay layers.

11.3.3 The arguments presented in Chapters 6 and 10 could therefore be repeated to justify an undisturbed K_o-depth distribution similar to

that estimated for the Magnus site. With the sand layers this assumption needs some further explanation. Daramola (1978) established that dense sands prepared in the laboratory show low K_o values when normally consolidated, but that σ'_h/σ'_v increases steeply when the soils are swelled back under conditions of zero lateral strain. Assuming the Magnus apparent OCR-depth plot to be applicable, Daramola's K_o data for overconsolidated dense Ham River sand plot significantly above the interpreted Magnus K_o profile given in Figure 6.66. In Chapter 6 it was recalled that Hight (1983) had argued that much of the apparent overconsolidation in the upper clay layers at North Sea sites was due to wave compaction, rather than unloading, and that this process would produce lower horizontal stresses than might be expected for literally overconsolidated sediments. It is notoriously difficult to measure insitu horizontal stress in sands, and the writer is unaware of any good data that give guidance in this instance. Thus, in the absence of any further information, it is reasonable to conclude that the Magnus interpreted K_o profile may be assumed to apply for both the clay and sand layers at Hutton. A minor variation to this scheme is the adoption of slightly higher values of K_o for the mid-depth strata; this reflects the higher plasticity indices measured for these layers.

11.3.4 Naturally, the estimated undisturbed K_o conditions can only be acting at some distance from the piles just before loading commences, and account must be taken of the effects of installation at points closer to the shaft. Before detailing the stress distributions adopted, it is useful to note that the following differences between the Hutton and Magnus cases;

(i) The piles are shorter and of smaller diameter.

(ii) The clay layers are thinner, more frequently laminated and are probably more freely draining.

(iii) A considerable proportion of the soil profile is granular.

(iv) The Hutton foundations were installed at least 12 months before the main platform installation, which took place in July 1984.

11.3.5 These four factors combine to suggest that a far greater degree of equalisation had taken place at the time of platform loading than was the case at Magnus. To simplify the analysis, only the fully equalised condition will be considered. This may be slightly non-conservative for the July 1984 date, but is likely to be appropriate for current and future conditions at the TLP site.

11.3.6 Following similar arguments to those set out in Chapter 10, estimates have been made of σ'_r, σ'_z and σ'_θ acting over the shaft length. Undisturbed conditions are assumed at radii greater than 20 r_o, and at levels below the pile tip. A log-linear variation is specified between the stated limits, and the stresses are listed in non-dimensional form in Table 11.1.

11.3.7 Regarding the soil property parameters required for the analysis, Table 11.2 lists values of γ', λ, k, e_1 and ϕ' interpreted from the site investigation data. The voids ratios at unit pressure and the slopes of the virgin consolidation lines were established from the final 1,600 to 3,200 kpa loading stages of the Fugro oedometer tests, and k was estimated from the λ/k - OCR relationships summarized in Figure 6.15. The values of ϕ' were judged from the plasticity indices, the Imperial College test data and the effective stress tests reported by Fugro.

11.3.8 For the first analyses it was decided to use the same expressions describing pre-yield stiffness variations that were set out in Chapter 10. Jardine, Symes and Burland (1984) found, in a limited set of triaxial tests, that medium dense Ham River sand gave similar variations of E_u/p'_o with strain to reconstituted Magnus clay. More recent work at Imperial College, and in other laboratories, has demonstrated that the Magnus test data are in fact typical of a wide range of low to medium plasticity clays, Jardine and Hight (1985) and

Hight (1985). Although it is possible that the mid-depth clay layers would show softer characteristics than those assumed for the Magnus analyses, it is equally likely that the very dense sand layers would show stiffer behaviour. From the present collection of laboratory data, it was therefore appropriate to employ the previously established expressions for $G/p'_o = f(E)$ and $K'/p'_o = g(\varepsilon_v)$ in all layers, and to substitute the same values for the coefficients that were assessed in Chapter 10.

11.3.9 For the cohesive soils the Modified Cam-Clay model was retained to describe the post yield behaviour. A different kind of soil model is appropriate for the granular strata, even though the non-linear stiffness expressions may be retained, and it was assumed that a Mohr-Coulomb description of yielding would be suitable. An option within program ICFEP allows this choice to be made, and it is only necessary to evaluate the friction angle, ϕ', and to select a dilation angle V'. In this case the latter parameter was set equal to $\phi'/2$.

11.3.10 Finally, limits to the interface stress ratio, τ_{rz}/σ'_r, had to be assessed. For the upper clay layers comparisons between index properties suggested that the characteristics developed for the Magnus[1] layers I and II would be applicable, whilst the parameters selected for the Magnus[1] layers III and IV were adopted for the lower Hutton clay stratum.

11.3.11 It was also considered important to include reduced values of δ' over the pile's interface with the sand layers. Lemos (1985) had carried out direct shear tests between Leighton-Buzzard sand and steel interfaces, in which he systematically varied roughness, density and normal stress. Following from these results it was decided to limit the maximum ratio τ_{rz}/σ'_r to tan 24°. For the sand layers δ' was specified as a constant, and did not vary with displacement.

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[1] These characteristics are summarized in Figure 10.8.

11.4 FINITE ELEMENT ANALYSIS

11.4.1 The idealised strata and finite element mesh used for the study are shown in Figure 11.11. The mesh was designed to provide thin elements close to the pile, and assumed a full quaternary thickness of 180 metres, or three times the pile depth. As discussed previously, it was only necessary to study the long-term case, and this was simulated by applying upward increments of pile head displacement. The response of the sand was considered to be drained, but no drainage was assumed for the clay layers. The pile was assumed to behave as an equivalent solid elastic cylinder of 1.83 m diameter, with Young's modulus equal to 28 x 10^3 MN/m^2.

11.4.2 The results may be discussed in a similar way to the Magnus studies, and the first aspect to consider is the single-pile load-displacement plot shown in Figure 11.12. It is useful to contrast the graph with that from the long-term Magnus analysis given on Figure 10.9, and such a comparison shows (i) the Hutton capacity is around 45% smaller, and (ii) the shape of the curves is generally similar, although the Hutton pile reaches peak capacity after 27 rather than 60 mm displacement. Considering the relative dimensions of the two piles, these results are consistent and it is interesting that the computed capacity exceeds the 'API' design limit by around 70%.

11.4.3 Greater insights into the development of the pile capacity can be gained from the plots of Figures 11.13 to 11.18. The first diagram shows distributions of τ_{rz} with depth for three points on the load-displacement curve. The profile for 1.1 mm displacement corresponds to the sustained tension (calm weather) condition, the second and third traces predict profiles with the overall capacity fully and half mobilised. The patterns are familiar from the Magnus runs B and C, and show the alternating layers of sand to have a relatively minor effect on the distribution of shaft friction. Progressive failure propagates from the pile head, and partial mobilisation of the ultimate capacity is again characterised by sharp gradients of τ_{rz} with depth.

11.4.4 Three 'typical' points on the shaft are identified on Figure 11.13, and the local variations of τ_{rz}, σ'_r and pore water pressure with displacement are graphed on Figure 11.14 a), b) and c). Plot a) illustrates the orderly progression of shaft resistance mobilisation, with point 1 failing, and strain softening, long before points 2 and 3. The changes in σ'_r are shown in b), and in each case the radial effective stresses remain within 8% of their initial values. For clay layer C the changes are slightly positive. In the mid-depth sand stratum, D, σ'_r falls by around 7%. In layer E a counterbalancing increase in radial effective stress is observed. It is interesting that for the three points considered the effects exactly cancel and the mean change in σ'_r is negligible. Part c) of Figure 11.14 shows the interface pore pressure changes in the two clay layers considered, and these prove to be similar with falling pressures that gradually tend to minima around 60 Kpa below the initial hydro-static values.

11.4.5 The developments in the pore water regime are further explored in the contour diagrams of Figure 11.15 a) and b). Only reductions in piezometric pressure are predicted for pile head displacements of 1.1 and 9.9 mm, and of course these are confined to the clay layers. The greatest changes occur at the pile-soil interface, but tension loading induces significant pore pressure effects many metres from the centre line. No analysis has been made of the time required for pore pressure equalisation, but the process is unlikely to be very rapid.

11.4.6 Figure 11.16 presents the stress-paths calculated for points 1, 2 and 3. The two clay layers show the 'Magnus' pattern of J rising and falling with little change in p'. The sand stratum, represented by point 2, follows a different path; under drained conditions tension loading leads to reductions in p' as J increases.

11.4.7 The patterns of soil straining around a single tension pile are illustrated by the contours of E shown[1] in Figure 11.17 a) and

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[1] E is the shear strain invariant defined as

$$E = \frac{\sqrt{2}}{3}\left[(\varepsilon_1 - \varepsilon_2)^2 + (\varepsilon_1 - \varepsilon_3)^2 + (\varepsilon_2 - \varepsilon_3)^2\right]^{1/2}$$

b). The plots emphasize two main points, (i) the shear strains in the soil mass are generally extremely small and E only approaches the 0.1% level[1] at the closest points to the pile, when high load factors are developed, (ii) the pattern of strains shows the importance of placing the base and side boundaries of the finite element mesh at sufficient distances from the pile.

11.4.8 The progressions in ground straining patterns are expressed at the surface as variations in the profiles of vertical displacement. These are plotted in non-dimensional form in Figure 11.18, and show the expected features of surface heave becoming increasingly concentrated close to the shaft as the pile is loaded. These profiles form the basis for the calculations of pile group displacements set out in the following sections.

11.5 PREDICTIONS OF PILE GROUP DISPLACEMENTS

11.5.1 The measurements of pile group movements made at the Hutton site provide a special opportunity to compare predictions with nature. To increase the amount of information gained from the site monitoring two kinds of measurements were made. Firstly, the global foundation vertical movements were observed by monitoring uplift with respect to a datum positioned on a radius 60 metres from the template centre[2]. Secondly, local gauges were deployed that measured vertical displacements of the group relative to a datum positioned only 2 to 3 metres from the edge of the pile template. This location is within the zone of influence of the eight piles in the group, and comparisons between local and remote gauge data allow the distribution of ground heave to be discussed in addition to the absolute values of group displacement.

11.5.2 A third feature of instrumentation scheme is the fixing of the active pots at positions on the templates that allow rotations

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1 For an undrained triaxial test E is equivalent to $\sqrt{3}\ \varepsilon_a$

2 An inspection of Figure 11.18 shows ground displacements at this distance to be negligible.

developed by changes in applied moment to be deduced, provided that the effects of total axial load can be accounted for. A principal aim of the numerical analyses was therefore to provide comparative predictions for the three quantities that could be measured on site.

11.5.3 Firstly, the global axial load displacement curve could be predicted from the single pile runs in the way discussed in Chapters 4 and 10. To do this, uniform loading and superposition were assumed, and the 'rigid inclusion' effect of neighbouring piles neglected. The treatment is thus not rigorous, but if the ratio of pile to soil stiffness is not large and the piles occupy only a small proportion of the group's plan area, the errors are unlikely to be large. Randolph and Wroth (1979) made similar assumptions in their simplified elastic analysis of pile group interaction and found close agreement with more complete Boundary Integral Equation calculations. Indeed, their results were obtained with relatively close group spacings and a far higher ratio of pile to soil stiffness than that considered here. The careful pile group field tests of Cooke et al. (1979) add further support to the use of this type of superposition.

11.5.4 To provide predictions of the local datum gauge readings, it was necessary to make one further set of calculations. Firstly, the global pile group displacements were obtained, and then the absolute displacements of the local datum position were assessed using a similar superposition procedure. Deducting the calculated heaves of the local datum position from the global template uplifts gave the relative displacements that would be measured with local displacement gauges. The results of these pile group analyses are indicated on Figure 11.19. Plots of finte element predictions for local and remote datums are shown, as is the initial group compliance estimated by the Platform designers. The latter prediction was based on conventional T-Z and linear elastic calculation routines, Barton (1984).

11.5.5 Regarding these plots, three points are noteworthy;

(i) the finite element analyses indicate a very stiff response, with overall displacements many times smaller than those routinely calculated.

(ii) The local and remote datum plots show similar characteristics; for small uplifts the remotely measured displacements are around 60% greater than the local ones, at larger displacements the two curves become progressively closer.

(iii) The local datum characteristic almost exactly recovers the single pile load-displacement curve.

11.5.6 In comparison with the less realistic linear elasto-plastic analyses discussed in Chapter 4, it is interesting to note that a comparable curve was predicted for the local datum displacements, but that far larger global movements were calculated in the original analysis. (The results obtained with the simpler soil models were summarized in Figure 4.19).

11.5.7 If a further approximation is accepted, it is possible to use the single pile analysis to estimate the group moment-rotation stiffness. The procedure is illustrated in Figure 11.20 for a moment M applied about an axis of symmetry, with the pile-template fixings considered as pin joints. A distribution of axial pile loads must first be assumed, and the increments associated with a moment applied at relatively low load factors can be summarized by the symmetrical forces $\pm p_1$ and $\pm p_2$. A solution can be found immediately by assuming a relationship between p_1 and p_2, and as a reasonable first approximation we can write $p_1 = p_2.L_1/L_2$. This done, the displacements at any pile can be found by summing the contributions of vertical movement provided by the pile in question, and the other seven in the group. To simplify the procedure the set of interaction factors found for the single pile at 1.1 mm displacement were used,[1] see Figure 11.18. For the linear system described the displacements at piles 1 and 2 are equal and opposite to those at 7 and 8. Similarly, piles 4 and 3 can be paired with their opposites at 5 and 6. A test can then be applied to the assumed ratio of p_1/p_2, as for a rigid template δ_1/δ_2 must also equal L_1/L_2. If the ratio proves satisfactory, the rotation of the group can be calculated as $\theta = \delta_1/L_1 = \delta_2/L_2$.

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1 It was noted that for relatively small load factors the interaction factors were sensibly independent of loading level.

11.5.8 Such an exercise was carried out, assuming p_1 to equal 5 MN. A set of displacements was calculated with δ_1 = 1.230 mm and δ_2 = 0.492 mm, and the first assumption for p_1/p_2 lead to a ratio of δ_1/δ_2 which was within 5% of L_1/L_2; this was considered an acceptable approximation[1]. Correlating the applied moment to the calculated angular displacement gave an overall moment-rotation stiffness of 113 x 10^6 tonne m/radian. Using a similar procedure to that set out in 11.5.4, it is possible to calculate a moment-rotation stiffness with respect to a local datum for settlement observations, and a value of 163 x 10^6 tonne m/radian was found. The predicted global rotational stiffness is far greater than the value of 44 x 10^6 tonne m/radian assessed by the foundation designers.

11.5.9 In summary, Part 1 has reported the development of a finite element analysis of single piles installed at the Hutton site. This has been extended to make predictions for the group axial load-displacement, and moment-rotation, relationships of the Tension Leg Platform foundations. Axial stiffness curves have been given for both local and remote datum observations, and linear rotational stiffnesses have been calculated for applied moments in the range 0-20,000 tonne metres. In Part 2 of the chapter these predictions will be contrasted with field measurements. Firstly, the instrumentation system deployed at the Hutton site will be described, then the results obtained during platform installation will be reported, and finally comparisons made between predictions and field measurements.

PART 2

11.6 THE SETTLEMENT MONITORING SYSTEMS DEVELOPED FOR THE FOUNDATIONS OF THE HUTTON TENSION LEG PLATFORM

11.6.1 Conoco's Hutton Tension Leg Platform represents a major advance in platform design. The structure, which was installed in

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1 If required the value of p_2 could have been increased in small steps until δ_1/δ_2 exactly equalled L_1/l_2. This would, however, have made very little difference to the deduced rotational stiffness of the pile group.

July 1984, is illustrated in Figure 11.21. It can be seen that the platform exerts tensile forces on the four templates that vary with the draught of the floating hull. Wind, waves and tide cause fluctuations in the loads from a nominal sustained tension. Under the combined loadings of the 100 year storm the maximum upward force developed on an individual group is 9,100 tonnes. As experience of the behaviour of piles under these conditions is limited, Conoco (U.K.) Ltd. decided to monitor the performance of the foundations using underwater instrumentation.

11.6.2 It was decided that the integrity of the foundations could be assessed by studying any tendency for the pile groups to pull out under sustained load. This can be illustrated using the general curve sketched in Figure 11.22. Up to a certain group force, L_E, load-unload loops give rise to negligible permanent displacements. For loading paths such as OAB or BCD irrecoverable displacements develop. These movements become large as the maximum load approaches the ultimate capacity. A system which recorded the development of any permanent displacements with time could thus be used to assess the performance of the pile groups under the applied loads.

11.6.3 Conoco asked the Soil Mechanics section of the Civil Engineering Department at Imperial College to assist with the development of equipment and to carry out analysis for the monitoring exercise. Following this request, the author worked to enhance the offshore settlement gauge which had originally been designed for BP's Magnus Foundation Monitoring Project. (The previous development work is described in Appendix A6). The following sections describe the development of the settlement gauges and their successful installation and operation on the Hutton TLP. One of the more novel features of the equipment was the acoustic telemetry system which was required to transmit the data from the sea bed. The telemetry equipment for the project was designed and manufactured by Bell Electronics in association with Imperial College.

11.6.4 The settlement gauges operate using a hydraulic differential-pressure principle. As shown in Figure 11.23 the two primary

components of each system are the active and passive pots. The active pot is attached to the foundation template and the passive pot is fixed to either a local (L) or remote (R) datum position. The active and passive pots are interconnected by a dual hydraulic system, one limb of which is filled with mercury, the other with oil. If the active pot moves vertically in relation to the passive pot, a differential pressure is set up between the mercury and oil limbs at their terminations in the active pot. The oil and mercury pressures in the passive pot are maintained at ambient seabed pressure by means of two flexible membranes. Elements of the system are protected by reinforced concrete covers.

This particular arrangement offers a number of advantages;

(i) A differential pressure hydraulic system allows high resolution and stability.

(ii) Changes in seabed pressure do not affect the system.

(iii) The gauges can be installed by divers without special equipment.

11.6.5 The most important aspect of the gauge design was the choice of differential pressure transducer. Although cost, size, compliance and sensitivity were considered, accuracy, stability and overload protection were the vital requirements. Long-term tests were carried out to assess stability and accuracy for a range of devices before the final choice was made. After adopting a tandem arrangement with two large bonded strain-gauge diaphragm transducers, development work fell into three main areas;

(i) Designing the system components; metal work, signal conditioning, underwater cabling, hydraulics and antifouling systems.

(ii) Developing techniques for assembly and testing, including a vacuum filling arrangement for the hydraulic circuits and the hyperbaric calibration facility shown in Figure 11.24.

(iii) Developing an underwater deployment package to assist installation, and carrying out feasibility trials with divers.

The assembly and calibration of the gauges was carried out for Imperial College by Custom design Mouldings of Cwmbran, Gwent.

11.6.6 The two transducers in each gauge were of similar construction, but different ranges. A full-scale operation of ±0.07 bar was selected for the 'fine' devices and ±0.3 bar for the 'coarse'. These allowed corresponding measurements of displacement between ±60 mm and ±300 mm. Full overload protection was provided for the diaphragms up to a limit of ±30 bar. The signal conditioning circuits amplifed the wheatstone bridge voltages to give a full-scale output of ±2.5V, and could maintain constant bridge conditions in the event of large changes in power supply voltage.

11.6.7 The hyperbaric calibrations showed overall accuracies of between 0.05 to 0.1 mm over a 60 mm displacement loop for both fine and coarse transducers; a typical calibration is shown in Figure 11.25. Stability checks showed negligible drifts over a period of days. Cyclic reponse and temperature sensitivity tests were also carried out and showed the gauges to have the required characteristics.

11.6.8 With a conventional platform, a cable connection could be provided to link the foundation templates to a power supply and monitoring station at the surface, although there can be difficulties with cable pulling and the duct work is expensive. As the TLP configuration makes the running of cables between the hull and seabed impractical, Conoco decided to use acoustic telemetry links for the positioning systems required during the installation and operation of the platform. Bell Electronics were commissioned to extend their permanent positioning system to provide an acoustic data link with the settlement gauges and to work with Imperial College to develop a seabed acoustic package for each settlement gauge. The principal functions of the package were to provide a long life power supply, to carry out data logging and data compression tasks and to code and decode

communications between platform and gauge. In summary the SBAP was required to;

(i) provide, when commanded, power to its dedicated settlement gauge for a warm-up period of two minutes.

(ii) Monitor, digitise and store sampled data at a rate of one complete set per second for three minutes.

(iii) Process and acoustically transmit coded parameters from the seabed to the Acoustic Positioning Systems (APS) hull transponders.

11.6.9 The gauges communicate with the surface via an acoustic telemetry link between the SBAP, with its own sonar equipment, and transducers mounted on the TLP hull. The use of the surface APS system is momentarily suspended during settlement gauge telemetry operations. The operator uses the command telemetry link to set the SBAP into one of its four operating modes :

(i) quiescent
(ii) gauge monitoring
(iii) standby
(iv) data transmission

If a command to commence gauge monitoring is received, the SBAP commences a two minute warm-up period for the settlement gauge. This completed, the outputs and 5 volt references for both coarse and fine transducers are sampled at a rate of one per second for 3 minutes. Whilst the data is being collected the mean, minima, maxima and standard deviations are calculated for both transducers, as are the mean values for the two 5 volt references. The 720 individual readings are thus summarized using 10 parameters. When the data is requested by an acoustic command from the surface, the summary parameters are transmitted as ten packets, each with a 12 bit data element and an identification code.

11.6.10 A microprocessor is included in each SBAP to manage the following functions;

(i) to direct the encoding and decoding of the acoustic signals;

(ii) to control system timing;

(iii) to process the sampled data;

(iv) to perform operational checks.

The principal design objectives for the SBAP circuits were to provide long uninterrupted service and high accuracy. The intended life was 10 years with 2,000 individual gauge reading cycles. The combined accuracy of analogue to digital conversion, data processing and coding is better than ±0.1% Full Scale Operation (F.S.O.) and resolution finer than ±0.025% F.S.O. The probability of correct data communication between the gauges and the TLP hull was designed to exceed 99%.

11.6.11 The surface processing equipment employs the same hardware as the platform's acoustic positioning system (APS). The software for commanding the SBAP and displaying the received data was merged with the positioning system and can be initiated using a single command. Following this request the operator is able to switch on a gauge, read data and switch off by using simple commands. If any data is corrupted during transmission re-reading can be requested. The data received at the surface is converted to engineering units using the internal calibration tables within the display processor. Displacements and voltages are shown on the console screen and hard copies may be taken from a dedicated line printer.

11.6.12 An auxiliary read-out unit was provided for use by divers during gauge installation and for occasions when direct readings may be required. The AROU is an underwater digital voltmeter which connects to the SBAP through an underwater mateable connector. Once connected the unit provides continuous power to the gauge and redirects output

signals from the acoustic telemetry unit. The AROU circuits sample and digitise both transducer output voltages at a rate of one reading per 5 seconds. The data are presented on two separate red LED displays. Use of the unit requires no intervention by the SBAP and causes no permanent effect to the gauge installation. The AROU has a battery pack which is sufficient for 150 hours of continuous operation. The accuracy of the analogue to digital conversion is better than ±0.1% F.S.O. and the resolution is finer than ±0.03% F.S.O. The AROU and SBAP systems are electrically compatible and, for the same transducer data, the two systems give digital outputs which are similar to within 0.2% F.S.O.

11.6.13 Each complete gauge set was packaged for diver deployment as shown in Figure 11.26. Saturation divers were used to mount three complete gauge sets on the foundation templates, this work being supervised by the author on board the Diving Support Vessels Stephaniturm and Deepwater I during May and June 1984. Figure 11.27 shows a typical foundation template, photographed before offshore installation. Figure 11.28 shows an active pot fixed to its template in a photograph taken by divers, and Figure 11.29 gives the general arrangements of the templates and gauges on the sea bed.

11.6.14 Some initial problems were experienced with the acoustic link, but the equipment was fully commissioned before tow-out and worked faultlessly throughout platform installation.

11.6.15 The TLP installation programme allowed a 24 hour period between the arrival of the platform on site and the start of the loading sequence. This pause was used to evaluate the gauge stabilities insitu and regular readings showed values of displacement which were constant to within ±0.03 mm for the fine transducers and ±0.15 mm for the coarse. These figures correspond to one least significant bit for each channel. The controlled slow increases in load that occurred as the platform legs were sequentially connected and tensioned provided an ideal opportunity to study the load-deflection characteristics of the piled foundations. Each of the 16 legs had been fully instrumented with load cells by Conoco, and over 200

simultaneous measurements of tension force and pile displacement were recorded.

11.7 DATA OBTAINED DURING THE INSTALLATION OF THE HUTTON TLP

11.7.1 The data obtained during the installation of the Tension Leg Platform are discussed in this section. Although the original aim of the underwater instrumentation was to provide a durable system to verify foundation performance, the installation procedure provided a unique opportunity to study the detailed stiffness characteristics of a large pile group at relatively low working loads.

11.7.2 The installation of the platform consisted of assembling and connecting the 16 tension legs in a series of complex operations. Once the first four legs were connected to their foundation templates, loads were applied to the foundations by tension motion compensators (TMC's). After a certain tension had been reached the load was transferred to load blocks and the tensions then varied as ballast was dumped and the mean sea level fluctuated with the tide. As further legs were installed loads were distributed between the anchoring points A, B, C and D on each template, see Figure 11.29. The total tension forces slowly increased to around 4,000 tonnes per template over the four day installation period, but with each tide there was a substantial variation in the loads. The Tension Leg installation procedure is more fully discussed by Tetlow and Ellis (1983).

11.7.3 During the four day tensioning period measurements of the forces acting on each leg were made using the TMC indicators initially, and then the load cell instrumentation installed on the load block assemblies. The measurements were of the tensions at the mooring compartment level of the platform and, to calculate the forces applied to the foundations, a deduction of 90 tonnes per assembled leg was made to account for dead weight. It is also important to note that the submerged dead weight of each template was approximately 900 tonnes.

11.7.4 Plots of the variations in combined tension on templates 1 and 2 are given in Figures 11.30 and 11.31. It can be seen from

Figure 11.29 that the first load increments applied to anchor point A were not central and would have given rise to turning moments about axes X-X' and Y-Y'. The active pots for all three gauges were located close to the axis X-X' so the main effects of eccentric loading would be caused by the moment about the Y-Y' axis. The eccentricity of loading about this axis varied from approximately 2.3 metres, when only leg A or legs A and D were connected, to 0.0 metres when all four legs were installed and equally loaded. Plots of the variation in eccentricity of loading about Y-Y' with time are also given on Figures 11.30 and 11.31. Plots of tide level are shown on the same graphs, and its effect on tension load is clear.

11.7.5 The settlement gauge systems were set in operation as soon as the platform was within 300 metres range of its final position. The gauges were interrogated via a single temporary transducer which was lowered from the platform's well bay area. During installation the permanent hull mounted transducer positions were dedicated to the Sonardyne Acoustic Positioning System[1]. Readings were taken at approximately 30 minute intervals over a 24 hour period before the TMC's entered their heave suppression mode to apply the first loads to the foundations. The initial readings were intended to confirm the stability of the gauges and their acoustic links. Once the platform was within 100 metres of the target position the acoustic system correctly transmitted and decoded more than 99% of the data sent from the settlement gauges. Occasionally communications were interrupted by interference from the Sonardyne systems.

11.7.6 The static readings from the local gauges SG4L were both stable to within one least significant bit (l.s.b.) over the 24 hour period before heave suppression. The readings for the remote gauge SG3R showed larger variations, of approximately 0.1% full-scale output, over this interval which were related to similar shifts in the two 5 Volt references. The variations in the reference voltages became progressively smaller with time. After 08.00 hours on 13th July, 1984

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1 This system was used to navigate the platform into the optimum position above the foundations.

their values were constant to within 1 l.s.b. This initial instability was thought to be related to the analogue to digital conversion circuit in the sea bed acoustic package, and to be a result of the relatively short "burn-in" period sustained by gauge SG3R before delivery. The static period allowed the initial instability to calibrated and corrections can be made in case of any further shifts in the 5 volt reference values.

11.7.7 The initial settlement gauge data appeared at the APS console as non-zero values of 'pull-out'. It was not generally feasible to set the gauges to give precisely zero values during diver-installation and so the displacements of the templates caused by loading were assessed as changes from nominal initial values of 'pullout'.

11.7.8 The data from the three settlement gauges are shown on Figures 11.32 to 11.34. These records should be viewed in conjunction with the load-time graphs given in Figures 11.30 and 11.31. Five main points are clear from an inspection of these recordings;

(i) The scatter in the settlement-time curves is very small, and amounts to around one 1.s.b. for both the fine and coarse transducers (i.e. ±0.15 mm and ±0.03 mm respectively).

(ii) There is very good agreement between coarse and fine transducers on all three gauges.

(iii) There is a close correspondence between changes in tension load and displacement.

(iv) The displacements are very small.

(v) As the eccentricity of load decreases, the displacements reduce.

11.7.9 The load-displacement relationships derived from the changes in tension and the outputs of the fine transducers over the TLP installation period are given in Figures 11.35 to 11.37. It is evident that the load displacement records can be split into segments depending on the eccentricity of loading. For simplicity, points where the legs of a given template were unequally loaded have been shown as dashed lines. Over the longest segment of the load settle- ment records, where the eccentricity of loading about Y-Y was 2.3 metres, the curves showed a gradual trend away from the initial straight-line plots. It is interesting to note that the tidal cycles of unloading and reloading appear as recoverable excersions up and down apparently unique loading curves.

11.7.10 Before considering the load displacement data in greater detail it is useful to briefly explore the probable effects of eccentric loading on the settlement gauge readings. Considering the pile template arrangements shown in Figure 11.29 and assuming a linear foundation response, it is clear that a pure turning moment applied about axis Y-Y' would give rise to a rotation of the template and a relative uplifting of the active pot as shown in Figure 11.38. The effects of combined axial loads and turning moments could thus be expressed as a fan of lines relating tension, upward displacement of the active pot and eccentricity of loading.

11.7.11 When the effects of eccentric loading are recognised, it then becomes possible to glean extra information by separating the rotations from the mean axial displacements. The changes in gauge readings that took place as the eccentricity of loading shifted can be used to calculate the rotation of the templates about Y-Y' due to the reduction in the moments about that axis. Rotational stiffnesses can then be determined, and Table 11.3 gives the values calculated for the shifts in eccentricity from 2.3 to 0.77 metres, and from 0.77 to 0.0 metres for a fixed total load of 3,000 tonnes per template.

11.7.12 Relationships between Tension Force and deflection for central, non-eccentric, loading have been extrapolated from the data and are given on Figure 11.39. It is encouraging that the local

displacements measured on templates F1 and F2 show close correspondence. It is also interesting that the differences between locally and remotely measured displacements are relatively small.

11.7.13 The monitoring of the foundation performance has continued since the installation of the platform. Evidence gathered over the first 70 days of the working life of the TLP showed that displacements continued to increase during an extended period of calm weather, albeit at a decreasing rate, even though the loads rarely exceeded their installation values. In a report to Conoco, dated November 1984, it was noted that increases in uplift of between 30 and 50% had occurred and it was considered that the main reason for these movements was time dependent soil stiffness in the soil mass. Following this calm weather period, the gauges have been used to assess the foundation response to more severe environmental loading.

11.7.14 The principal aim of the continuing monitoring exercise is the assessment of foundations' ability to sustain extreme storm loading. The first winter's data have now been gathered and guidelines for the interpretation of the gauge information have been formulated. Readers are referred to Jardine, Potts, Hight and Burland (1985) for an account of the strategy that has been developed and the type of analyses carried out to establish limits to the permissible range of permanent pile group displacements.

11.8 COMPARISONS BETWEEN PREDICTIONS AND FIELD MEASUREMENTS

11.8.1 The measurements made during installation may be compared with the group axial force-displacement predictions and moment-rotation stiffnesses discussed in Section 11.5. Qualitatively, the calculated axial load-uplift curves shown on Figure 11.19 are very similar to the measured data presented in Figure 11.38. The displacements are very small, the curves are gently non-linear and the remote datum measurements are less than double those determined with the local datum instruments. The comparison is made in a more quantitative way in Table 11.4, by listing the displacements for an average pile load of

4 MN and the various moment-rotation relationships. In discussing the rotational stiffnesses it is useful to note that the maximum moment applied to the foundations during installation was around 5,000 tonne metres, whilst the calculated stiffnesses covered the similar range of 0 to 20,000 tonne metres.

11.8.2 Inspecting the columns given in Table 11.4, the agreement found between the predictions and measurements is remarkable. It is worth stating that only one analysis was made of the Hutton TLP foundations, and that it is unlikely that a closer correspondence could be obtained by altering any of the many parameters assessed for this run. The results must be considered fortuitous to a certain degree, but the main trends cannot be contested. In comparison with routine design calculations, or even the more complex linear-elasto plastic analyses reported in Chapter 4, the treatment set out in Part 1 of this chapter allows unprecedented realism in the prediction of the foundation response.

11.8.3 It is also interesting to briefly consider the longer term displacements that were observed with the Hutton instrumentation. The slow upward movement of the pile groups could be accounted for by several processes, and these include : time dependent stiffness, the effects of low level cyclic loading, consolidation effects following initial tensioning, creep in the steel and pile-template grout, and the effects of seasonal changes in water temperature on the instruments.

11.8.4 The analysis set out in Part 1 of the chapter was therefore extended to include an assessment of the consolidation effect, which was simulated by allowing the pore pressures generated on loading to 4 MN to dissipate whilst the pile load was held constant. Considering the small zone of influence, it is likely that the equalisation process would be complete within the 70 day calm period. The finite element calculations showed that the pile head displacements could increase by around 10% as the pore pressures dissipate.

11.8.5 Concerning time dependent soil stiffness characteristics, it may be recalled from Chapters 6 and 7 that triaxial tests indicated a

sensitivity of stiffness to strain rate equivalent to 10 to 15% per log cycle. The installation procedure slowly built up the tension forces over a 2 day period, and the average duration of loading by 16/7/84 was around 36 hours. The following 70 day period therefore represents around 1.7 further log cycles of time, which might be expected to cause an overall stiffness reduction of between 17 and 26 percent. When combined, the loading duration and consolidation effects appear to be sufficient to explain the observed long-term template movements. However, it is possible that the other phenomena cited in 11.8.3 gave rise to part of the apparent increase in vertical displacement.

CHAPTER 12

SUMMARY AND CONCLUSIONS

12.1. INTRODUCTION

12.1.1 The final chapter of the thesis sets out a brief summary of the research and underlines some of the more important conclusions. The section commences with a short resume of the eleven preceding chapters, continues with a series of main points collected under convenient headings and ends with recommendations for further work.

12.2 REVIEW OF THE CONTENTS

12.2.1 The central argument of the dissertation has been developed in three parts. Chapters 1 to 3 comprise the first, and these critically reviewed current practice in the design of offshore pile groups, pointed to deficiencies in the empirically based codes, and considered how recent research might be applied to formulate a more fundamental approach to pile design.

12.2.2 Following from the introductory reviews, a series of experimental investigations was reported in Chapters 4 to 9. The work was intended to provide sufficient additional information to enable improved foundation analyses to be attempted for two important North Sea Production Platforms. These middle sections covered suites of numerical experiments, described programmes of laboratory work and also reported field scale instrumented pile studies. Accounts were given of the development of new equipment and techniques, and comparisons were made between the experimental results and relevant published field data. The combination of non-linear numerical techniques and soil models based on high quality laboratory data was seen to be powerful and produced encouraging agreement with full-scale behaviour.

12.2.3 The final part of the thesis reported the application of the improved descriptions of soil properties and initial conditions in Finite Element analyses of the Hutton TLP and Magnus foundations. Complex non-linear soil models were derived from the experimental work, and these were used to predict the behaviour of single piles. The results were then extended to consider group action, and predictions were made that could be compared with the field data that had been gathered with special underwater monitoring systems.

12.2.4 An important theme of the work was therefore the comparison of predictions and field measurements, and the final sections of the thesis showed that excellent agreement can be obtained between the two. Indeed, the results discussed in Chapters 8, 9, 10 and 11 allow the conclusion that the response of large pile groups can be accurately predicted from non-linear analyses, provided that the studies are based on a fundamental understanding of the soil mechanics involved. The significant improvements afforded by the described analyses emphasized the necessity of performing high quality tests to assess soil elemental stress-strain properties under appropriate conditions, the need to interpret the data carefully, and the benefits of formulating models that realistically predict soil behaviour, both at very small and large strains. The consideration of residual fabric formed near to, or at, the soil-pile interface was similarly shown to have a considerable effect on the predicted characteristics. Also apparent was the importance of making detailed assessments of the geological stratification and understanding the initial stress conditions that result from the installation of driven piles.

12.2.5 The material of the eleven preceding chapters has been presented in a somewhat compressed form, and it is difficult to provide a representative but concise summary of all the areas covered. Whilst the main results obtained from the analyses of the offshore foundations may be stated relatively briefly, the more general conclusions must be considerably condensed. The following sections can therefore only present thin precis' of the main topics covered in the thesis. These are followed by a short section dealing with recommendations for further research.

12.3 SOIL PROPERTIES

12.3.1 It was suggested in Chapter 3 that the starting point in any general effective stress theory of pile behaviour was the division of the problem into five areas and the first of these was the consideration of soil stress-strain and strength properties. A large part of the thesis has been associated with this topic, both in the reviews presented in Chapter 3 and the experimental work described in Chapters 5 to 8. The most important points can be summarized as follows;

(i) The studies involving reconstituted Magnus and London clay generally confirmed the themes developed by Gens (1982). Stark differences in behaviour were observed between isotropically and K_o consolidated soils and, whilst the concepts of critical state soil mechanics provided a useful starting point for understanding clay behaviour, the mathematically formulated models often gave oversimplified descriptions of soil strength and deformation characteristics.

(ii) The constitutive relations at very small strains were considered to be of particular interest, and special instrumentation was developed to study the stiffness properties of soils at strains as small as 10^{-3}%. The investigations showed that a wide range of soils, tested from a variety of initial conditions, display similar characteristics of very high initial stiffness and strong non-linearity. Index parameters were proposed to quantify these phenomena, and a significant body of data has been summarized in Chapters 6 and 7. Parallel tests have been run using the resonant column apparatus, and these have confirmed the trends highlighted by the triaxial experiments.

(iii) Comparisons between laboratory and full-scale measurements of soil stiffness show the triaxial data to be in generally good agreement with equivalent relations determined from instrumented field trials. Indeed, the results of K_o consolidated compression tests conducted at around 5% axial strain per day appear to be an appropriate basis for the accurate prediction of a wide range of undrained problems.

(iv) Tests involving load-unload loops, and the observation of yield points where sharp changes occur in undrained stress path direction, have been used to divide the available stress space into three distinct regions, namely;

(1) a zone where the shear response is very stiff, strains are recoverable, but the stress-strain relations are non-linear,

(2) a second zone where the shear behaviour is less stiff and plastic strains can develop, but where there is no indication of volumetric yield[1].

(3) an outer zone where volumetric yielding occurs, the shear response is soft and plastic straining predominates.

(v) Comparisons have been made between the response of intact and reconstituted specimens. Far less divergence was seen in the stiffness-strain relations than had been expected, although important differences were noted in the peak and ultimate strengths. The studies have helped to establish frameworks for the interpretation of laboratory experiments, and a recent paper by Hight, Gens and Jardine (1985) combines these results with other data

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1 i.e. sharp changes in undrained stress path direction, or drained bulk modulus.

in an attempt to provide practical guidance for offshore site investigations.

(vi) Summaries were given of relevant ring shear experiments conducted by Lemos (1985) and Lupini (1980). These demonstrated the importance of considering residual fabric and interface phenomena, particularly in the consideration of driven piles where soil can be sheared against the steel shaft. The results of interface tests with low plasticity clays gave surprising results, and large reductions in ϕ' were also noted for the case of dense sands shearing against a rough steel surface. The effects of shearing rate were seen to be significant in problems relating to pile installation and monotonic loading.

12.4 EVALUATION OF INITIAL GROUND CONDITIONS

12.4.1 The second essential area of study listed in Chapter 3 was the evaluation of initial ground conditions. Chapters 6, 7, 10 and 11 considered the problems of assessing, for particular sites, the stratification, the profiles of soil properties and the distributions of initial stresses. Of the results and arguments discussed in these chapters three main points are particularly noteworthy;

(i) When care was taken, remarkably consistent profiles of soil properties could be assessed for cohesive strata. With granular layers, the cone penetration test provided the most tangible way of assessing consistency, but very little other data was available from the commercial site investigations.

(ii) Whilst it was relatively simple to determine profiles of insitu vertical stress and pore water pressure, there were severe difficulties in establishing the horizontal effective stress distributions. High quality Self Boring Pressuremeter tests provided only crude estimates

of σ'_H at Canons Park, and no such data existed for the offshore locations. Sampling effects and the existence of relic gas bubble structures were seen to invalidate the use of laboratory suction measurements as a means of estimating K_o. Indeed, the insitu stresses could only be assessed indirectly by considering the geological history of the sites and their shear strength profiles.

(iii) It was useful to use a combination of insitu tests, experiments on reconstituted material and tests on intact samples to bracket the probable bands of insitu soil properties. However, great care was required when analysing the data, and the conventional interpretations of both laboratory and insitu tests were seen to often be unhelpful in obtaining representative stress-strain parameters for the prediction of foundation response.

12.5 ANALYSES OF DRIVEN PILE INSTALLATION AND SUBSEQUENT EQUILIBRATION PHENOMENA

12.5.1 A large section of Chapter 3 was devoted to reviewing the current state of knowledge regarding the effects of displacement pile installation, and the experiments of Chapter 9 highlighted the special case of jacked piles in overconsolidated clays. It was seen that treatments of the installation and equilibration processes are particularly sensitive to the assumptions made regarding the constitutive relations of the soil, and that complete analyses are not currently feasible. Attention was focussed on two simplified theories, which both assumed the soil strain paths to be material independent, and predictions were compared with the available body of field measurements. From the reviewed data it was possible to conclude;

(i) The cavity expansion solutions seriously oversimplify the patterns of ground straining caused by pile installation.

(ii) The fluid analogy made by Baligh (1975) in the strain-path method provides a better starting point than cylindrical cavity expansion.

(iii) The soil models employed in analysis need to be realistic.

(iv) Theoretical results obtained using a simple variant of modified cam clay in a cavity expansion solution greatly overestimate the radial effective stresses developed at the pile face immediately after installation, and this error is compounded during the subsequent pore pressure equalisation stage.

(v) For the case of lightly overconsolidated clay, reasonable estimates of the changes in σ'_r can be obtained by combining the strain path method with more realistic soil models.

(vi) Experimental evidence suggests that it is not reasonable to expect the ultimate radial effective stresses at the pile face to rise far above the initial geostatic values as a result of pile installation.

12.5.2 The results of the reviews and studies were used to develop best estimates of the ground stress distributions at various times after the installation of the Magnus and Hutton pile groups.

12.6 THE CONSIDERATION OF PILE LOADING

12.6.1 Chapter three included a summary of the various approaches to understanding the loading process of shear stress transfer from shaft to soil and the greatest controversy appeared to centre on the stress conditions developed near the interface. In most treatments, the soil is considered as adhering to the shaft and behaving as a continuum until peak capacity is reached. This assumption is probably justified with short sections of bored piles, but it was noted in Chapter 3 that accurate analyses could only be performed if realistic descriptions of soil continuum behaviour were considered. Indeed, Martins (1983) had used careful model tests to demonstrate that simple pseudo-elastic treatments were likely to produce unreasonable predictions of the stresses developed during loading. The same studies highlighted the

importance of residual fabric; this could either develop near the shaft during the loading of a bored pile, or result from the installation process of a displacement pile.

12.6.2 The possible formation of such residual fabric during installation was shown to have a major effect on the stresses developed during the subsequent loading of driven piles. In particular, far lower ultimate skin frictions could be mobilised, and the hypothesis of interface microfabric re-orientation was put forward to explain the relatively low shaft capacities found in typical pile tests. This argument was preferred to the "simple shear" analogy set out by Randolph and Wroth (1981). The full-scale numerical studies provided in Chapters 10 and 11 further explored the significance of possible microfabric effects near the soil-pile interface.

12.6.3 The analyses set out in Chapter 10 demonstrated that it was necessary to assume residual interface characteristics for the Magnus piles, if realistic capacities and displacements to failure were to be calculated. It was also shown that specifying limiting values of τ_{rz}/σ'_r considerably muted the variations of radial effective stress and pore water pressure predicted at the pile shaft. Moreover, a consequence of residual interface conditions appears to be that changes in σ'_r during pile loading may cancel sufficiently to be neglected.

12.6.4 The reviews of Chapters 2 and 3, the data presented in Chapter 9 and the calculations reported in Chapters 10 and 11 emphasize the importance of the pre-loading radial effective stress conditions at the pile shaft. It was noted that for large piles the effects of installation may equalise slowly, and that in some circumstances the capacity may slowly change for months or years after installation. This observation is significant when interpreting existing pile test data, and poses questions for the designer when matching loading conditions and ultimate capacities.

12.6.5 Whilst the pile capacity is controlled by the strengths mobilised near the shaft, the load-settlement characteristics reflect the combination of slip on discontinuous surfaces and shear straining

in the surrounding soil. The numerical studies showed the importance of considering both realistic interface behaviour and suitable non-linear characteristics for the soil mass, when attempting to model the response to load. This requirement became particularly clear when group behaviour was considered, as predictions based on linear elastic solutions appear to greatly overestimate the interactions of large piles. The studies showed this to result from intense radial gradients of shear strain; these imply relatively soft behaviour near the shaft at working load levels, but moduli which are many times higher operating only a few radii from the interface.

12.6.6 Approximate methods were set out which allow the non-linear single pile analyses to be used to assess group characteristics. Predictions for group axial displacements, ground surface profiles and moment-rotation stiffnesses could be produced. For the two offshore groups considered, excellent agreement was found between measurement and calculation.

12.7 RECOMMENDATIONS FOR FURTHER RESEARCH

12.7.1 The range of subjects touched upon in this thesis is wide, and it is intended to restrict the suggestions for further work to the area of improving predictions of the foundation response of large offshore structures. The main recommendations can then be listed as follows;

(i) There is a great need for well instrumented field scale model pile tests. These should be carried out to extend the data base of studies concerning the processes of installation, equilibration and loading. Measurements should include radial stress, shear stress and porewater pressure. The programmes of work should allow multiple tests to investigate the effects of installation rate, time and nature of loading to failure.

(ii) Tests to failure should also be carried out at prototype scale to check the predictions for capacity and load deformation characteristics. The experiments should simulate North Sea conditions as closely as possible, and should be instrumented to both gain vital data concerning failure conditions, and to assess the rates of dissipation after installation. The use of ground profile monitoring equipment is important, as this will help to assess ways of extrapolating single pile characteristics to consider group action.

(iii) At present there is little information to help designers estimate radial effective stresses at the pile face. It would be of great value to establish the insitu variations in horizontal effective stress at some typical North Sea sites, and to continue the use of instrumented working piles to study the changes in distributions of σ_r, u and τ_{rz} from installation through to storm loading.

(iv) Accurate theoretical predictions of the development of stresses and strains around displacement piles have become feasible for the case of low OCR clays. However, granular strata and layers of overconsolidated clays are frequently present at the sites of North sea platforms, and it is important that research is carried out to improve analyses of these more complex soil conditions.

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APPENDIX A1

Description of Program ICFEP
(Provided by Dr. D.M. Potts)

Imperial College Finite Element Program (ICFEP)

Introduction

The Imperial College Finite Element Program (ICFEP) has been developed over a period of years by Dr. D.M. Potts. The program has been written specifically for analysing geotechnical problems. Two dimensional, plane stress, plane strain and axi-symmetric geometries may be analysed with either linear or non-linear material behaviour. An option is also available to enable both large displacements and large strains (geometric non-linearity) to be dealt with. The program is built around a sophisticated non-linear solver which can deal with rapid changes in stiffnesses as well as non-symmetric stiffness matrices. A high degree of accuracy is maintained during the solution stage of the analyses and this enables the reliable prediction of failure conditions. ICFEP is currently mounted on the soil mechanics in house PRIME 750 computer, and, with the recent addition of versatile output data plotting routines, provides a powerful facility for geotechnical analysis.

Mesh Creation

In most finite element analyses the most tedious task is constructing the finite element mesh. This is also the stage at which errors can occur if large amounts of data are required to define the mesh. The sophisticated mesh generation facilities within ICFEP enable complicated meshes to be constructed with the minimum of data input. The comprehensive data checking routines also minimise errors and result in a consistent finite element mesh. The computer performs all of the repetitive tasks of mesh generation, leaving the analyst free to concentrate on the essential features of the problem under investigation.

Both four and/or eight noded isoparametric quadralateral elements may be used. There is no restriction on the different material types that may be used within any one analysis. Curved boundaries or material interfaces may also be employed.

Boundary Conditions

The following boundary condition options are currently available. These may be used sequentially or they may operate concurrently.

(i) Prescribed displacements

Displacements can be specified at a single internal or boundary node, or at a range of boundary nodes. Components of displacement in the local co-ordinate directions x_l, y_l may be prescribed. If no local axes are specified then the components refer to the global axes x_g, y_g.

(ii) Local axes

The orientation of local axes can be defined at a single internal or boundary node, or at a range of boundary nodes. It is possible to have different local axes defined at different nodes. This feature is very useful in conjunction with the other boundary condition options.

(iii) Tied freedoms

Degrees of freedom (the two displacement components at each node) can be tied at a single node, between two nodes or between a range of boundary nodes. This is achieved in the finite element analysis by giving the particular components of the nodal displacements common degree of freedom numbers.

(iv) Boundary stresses

Both normal and/or shear stresses can be applied to a section of the boundary.

(v) Point loads

Point loads can be defined with any orientation at a single internal or boundary node, or at a range of boundary nodes.

(vi) Body forces

Body forces can be defined as a field of force acting over the whole/or part of the body with any specified orientation. This option can be used to apply gravitational loading.

(vii) Pore pressures

Changes in pore pressures can be defined over the whole mesh, or over sections of elements within the mesh.

(viii) Dissipation

Pore pressure dissipation can be defined over the whole mesh, or over sections of elements within the mesh.

(ix) Excavation

This option can be used to simulate removal of material (elements) from the finite element mesh.

(x) Construction

This option may be used to simulate the adding of material (elements) to the finite element mesh.

(xi) Springs

Springs can be defined at a single internal or boundary node, or a range of boundary nodes.

(xii) Plane strain

Conventionally a plane strain analysis assumes zero strain in the out of plane direction. This is, however, an unnecessary restriction and it is possible (in the general case) to have a finite strain in this direction. This option allows such a strain to be prescribed.

(xiii) Release reactions

This option may be used to release reactions at a single internal or boundary node, or at a range of boundary nodes. This facility allows reactions created by prescribing displacements to be removed.

Material Models

The following constitutive models of material behaviour are currently available within ICFEP.

(i) Linear elastic

The material may be modelled as being isotropic or anisotropic elastic. The material properties may vary spatially and a facility for varying the properties linearly over an element is available.

(ii) 2D Mohr-Coulomb (linear elastic-perfectly plastic)

For the elastic part of the model, the options specified above in (i) are available. The plastic part is controlled by yield and plastic potential surfaces of the Mohr-Coulomb type. These are defined by the in plane stresses σ_x, σ_y and τ_{xy}. The out of plane stress, σ_z, does not affect the plasticity formulation. The angle of dilation, v, can be specified independently of the angle of shearing resistance ϕ and therefore both associated and non-

associated conditions can be considered.

(iii) Drucker-Prager (linear elastic-perfectly plastic)

For the elastic part of the model, the options specified above in (i) are available. The plastic part is controlled by yield and plastic potential surfaces of the Drucker-Prager type giving cones in principal stress space defined by strength parameters c', ϕ' and the angle of dilation, v. Both cones are circular in cross-section and the yield surface inscribes the equivalent Mohr-Coulomb hexagonal cone defined by the same strength parameters. The angle of dilation, v, is specified separately and non-associated ($v \neq \phi$) conditions may be defined.

(iv) Cam Clay (non-linear elastic, strain-hardening/softening plastic)

This is a form of the Cam Clay model presented by Schofield and Wroth (1968). Various options are available for the shapes of the yield and plastic potential surfaces in the deviatoric plane. It is possible to specify a Hvorslev surface on the 'dry' side. Elastic behaviour is governed by a non-linear bulk modulus derived from the swelling line in $e - \ln p'$ space and a constant shear modulus.

(v) Modified Cam Clay (non-linear elastic, strain-hardening/softening plastic)

This is a variant of the Modified Cam Clay model of Roscoe and Burland (1968). Various options are available for the shapes of the yield and plastic potential surfaces in the deviatoric plane. It is possible to specify a Hvorslev surface on the 'dry' side. Elastic behaviour is governed by a non-linear bulk modulus derived from the swelling lines and a constant shear modulus.

(vi) Drammen Clay (non-linear elastic, strain-hardening/softening plastic)

This model is described by van Eekelen and Potts (1979) and differs from Cam Clay and Modified Cam clay in that there is plastic behaviour below the state boundary surface. Various options are available for the shapes of the yield and plastic potential surfaces in the deviatoric plane. An option for dealing with cyclic loading is also available.

(vii) Nova-Wood (non-linear elastic, strain-hardening/softening plastic)

This model is described by Nova and Wood (1979) and was developed to describe the behaviour of sand. It is based on the concepts of critical state soil mechanics.

(viii) Non-linear Elastic (isotropic elastic)

In this model the elastic parameters are assumed to vary with stress level, mean stress and depth. A hyperbolic variation of Young's modulus can be specified and a different unloading-reloading modulus may be used. A strength cut off is also available.

(ix) Strain Hardening/Softening Mohr-Coulomb (linear elastic, strain-hardening/softening plastic)

For the elastic part of the model, the options specified above in (i) are available. Both the yield and plastic potential surfaces are of the Mohr Coulomb type and the strength parameters c' and ϕ' can vary with strain. It is also possible to vary the angle of dilation with strain and non-associated conditions ($\nu \neq \phi$) can be modelled.

Additional features of soil models

A "no tension" option is available with all the material models and this can be used to ensure that no parts of the soil continuum go into tension. Undrained analyses can be treated in terms of total stress with models i, ii, iii, ix and x. Alternatively, the models can be used to consider undrained behaviour in terms of effective stress as with the pore pressures being considered separately. This is effected by specifying a finite bulk modulus for the water phase.

Recent work has demonstrated the importance of considering soil characteristics that are initially very stiff, and also show rapid reductions of "elastic" moduli with strain. An option is available in which the elastic part of any of the above listed models can be replaced by appropriate non-linear variations of bulk and shear moduli using the forms given by Jardine (1985).

Solution

To solve the finite element equations ICFEP employs an accelerated form of the initial stress approach with the option of updating the stiffness matrix during the solution. Element integrations are performed numerically and 1 x 1, 2 x 2, 3 x 3 or 4 x 4 Gaussian integration schemes can be used. For non-linear analyses the boundary conditions are applied incrementally and facilities are available for stopping and restarting. It is also possible to save data at specified increments of the analyses for post processing.

Output

Printed output can either be requested during the finite element analysis or obtained from data stored on disc. Displacements, strains, stresses, pore water pressures, reactions and spring forces can all be listed. The analyst has complete freedom in selecting the

information to be printed, and it is possible to limit the output to that associated with various sections of the mesh.

ICFEP incorporates its own sophisticated plotting package which enables the following plots to be obtained :

(i) The displaced mesh.

(ii) Vectors of displacements.

(iii) Contours of all stress, strain and displacement components as well as principal values and invariants.

(iv) Vectors indicating principal stress and strain directions.

These can be obtained for the whole mesh or for selected sections of the mesh. The analyst has full control over the plotting scales and contour levels, although default values may be left for the program to determine.

A special option of the plotting package is the facility to plot graphs of the variation of either stress, strain or displacement with distance. For example, if an analysis of a retaining wall is to be carried out, graphs of the variation of stresses, strains and displacements over the vertical face of the wall can be requested. This option is extremely flexible and the variables may be plotted along a straight or curved line anywhere within the mesh. Again the analyst has control of the graph scales and size.

Plotting can either be requested during the solution stage of the analyses or be run on saved data after processing. The resulting plots can be viewed on the screens of the graphics terminals, and hard copies can either be obtained from these terminals or the higher quality HP plotter.

APPENDIX A2

The Modified Cam Clay Formulations available with Program ICFEP

A2.1 Introduction

A2.1.1 The modified Cam-Clay soil model was proposed by Roscoe and Burland (1968), and was a revised form of the Cam-Clay equations set out by Schofield and Wroth (1968). Both models are based on the ideas of critical state mechanics, which were formulated from the results of experiments performed on isotropically consolidated sediments. The modified Cam-Clay model was found to give improved predictions of the test data, particularly with regard to shear strains at low stress ratios.

A2.1.2 It is not intended to describe the derivation of the model in this appendix and, for further explanation readers are referred to Schofield and Wroth (1968) and Atkinson and Bransby (1978). Particular features of the model are discussed in more detail in Section 3.2.

A2.2 Essential features

A2.2.1 The model starts from the assumption that soil follows a linear path in e-log p' space when undergoing virgin consolidation with a fixed ratio of σ'_3/σ'_1. The slope of this path is assumed to be independent of σ'_3/σ'_1 and the line for isotropic consolidation is given in terms of specific volume as,

$$v = v_1 - \lambda \ln (p') \qquad \text{Eq. A2.1}$$

It is further assumed that swelling from any point on a virgin consolidation line takes place on a family of curves which can be written as

$$v = v_s - k \ln(p') \qquad \text{Eq. A2.2}$$

The behaviour in isotropic consolidation and swelling is indicated on Figure A2.1. Whilst virgin consolidation produces permanent volume changes, it is assumed that the straining during swelling is recoverable.

A2.2.2 The behaviour during shear is controlled by the position of the stress state in relation to a family of yield loci, which plot above the swelling lines in v, p', J space as shown in Figure A2.2. The soil is assumed to be elasto-plastic and its stress state can only exist under, or on, the generalised yield surface. In the original Cam-Clay Model this surface was "bullet-shaped", but in the modified version the locus plots as an ellipse in q-p', or J-p', space. The yield surface can be written as;

$$F(\sigma) = s^2 - (p'_o/p' - 1) = 0 \qquad \text{Eq. A2.3}$$

where $s = J/p', gy(\theta)$ Eq. A2.4

In this case p'_o represents the value of p' at the intersection of the current swelling line and the isotropic virgin consolidation line.

A2.2.3 The function $gy(\theta)$ can be selected to represent a surface of revolution in the deviatoric plane, or indeed to follow a Mohr-Coulomb hexagon. The choice of function can have an important influence on the results of boundary value analyses, as discussed by Potts and Martins (1982).

A2.2.4 The plastic strain increment vectors are normal to a plastic potential surface which need not necessarily coincide with the yield function. Potts and Gens (1983) discuss the application of a versatile family of plastic potential surfaces described with similar equations to the yield surface, but with different variations of $gp(\theta)$ with the lode angle. Thus the plastic potential is given by

$$Q = S^2 - \left(\frac{p'_Q}{p'} - 1\right) = 0 \qquad \text{Eq. A2.5}$$

$$S = J/(p', gp(\theta) \qquad \text{Eq. A2.6}$$

$$gp(\theta) = X/(1 + Y \sin 3\theta)^2 \qquad \text{Eq. A2.7}$$

The authors discuss suitable values for the coefficients X and Y.

A2.2.5 Beneath the yield locus the soil is assumed to behave elastically. As a consequence of Equation A2.2 the elastic drained bulk modulus K_b can be expressed as;

$$K_b = vp'/k \qquad \text{Eq. A2.8}$$

In the original formulation of Modified Cam Clay it was assumed that elastic shear strains could be neglected, but this leads to unrealistic predictions of stiffness for overconsolidated soil. The formulation employed for ICFEP allows a constant value of shear modulus G before yield, but it should be noted that when combined with Equation A2.8 this implies a variable Poisson's ratio for such cases as a drained triaxial compression test.

A2.2.6 The model so far described has been found to give good predictions of the behaviour of isotropically consolidated clays in a normally or lightly overconsolidated state. For soils on the 'dry' side of the critical state a different kind of yield function has been found to apply. A suitable surface was proposed by Hvorslev (1937) which can be expressed as a planar surface, inclined from the critical state line at angle α_h to the vertical. This is indicated in Figure A2.3, and its equation is given by;

$$F(\sigma) = s - \frac{P_{cs}}{P} - \left(1 - \frac{P_{cs}}{P}\right) \frac{\alpha_h}{gy(\theta)} \qquad \text{Eq. A2.9}$$

(s is defined in Equation A2.4)

The plastic potential can again be expressed by the Equations 2.5 and 2.6, and the function $gp(\theta)$ can be chosen to follow a Mohr-Coulomb hexagon if required.

A2.3 Variants referred to in the thesis

A2.3.1 From the previous paragraphs it will be seen that the term "Modified Cam Clay" covers a family of possible variants, rather than a single constitutive law. The use of a finite elastic shear modulus or

a Hvorslev yield surface on the dry side is optional, and choices can be made concerning the functions gy(θ) and gp(θ). These parameters can all be important.

A2.3.2 In the studies considered for this thesis two functions of gy(θ) have been employed. With the first, the yield loci form a surface of revolution about the space diagonal gy(θ) is constant with;

$$gy(\theta) = M/\sqrt{3} \qquad \text{Eq. A2.10}$$

M is usually taken as the critical state ratio of q/p' in the triaxial compression test where $\theta = -30°$. The equation implies that ϕ' varies considerably with the value of the intermediate principal stress, which contradicts most laboratory test data, and so gives inconsistencies in plane strain, Gens (1982).

A2.3.3 Alternatively, gy(θ) can be defined to follow a constant angle of friction. Martins (1983) demonstrated Equation A2.11 to be appropriate.

$$g(\theta) = \sin\phi'/(\cos\theta + \frac{1}{\sqrt{3}}\sin\theta.\sin\phi') \qquad \text{Eq. A2.11}$$

In either case g(θ) is fully defined by the familiar constant ϕ'.

A2.3.4 In the final sections of the thesis a further variant of the model is employed which gives more realistic behaviour within the 'elastic' region. Formulations are set out in Chapter 10 which present the shear and bulk moduli as functions of shear and volume strain invariants. The equations are empirically derived from laboratory test data, and sections are devoted in Chapters 8 and 10 to the choice of coefficients and the implications of their use in analysis.

APPENDIX A3

Push in Pressuremeter Tests at Magnus site

A3.1 During the period 14th April to 16th April, 1981, a geotechnical investigation was carried out at the Magnus site from the M.V. Ferder drill ship. Soil sampling and cone testing were performed by Fugro Ltd., and a programme of insitu push in pressuremeter tests was carried out by Stressprobe Limited. The work was reported in their document "Pressuremeter Testing in Magnus Field" dated June 1981.

A3.2 Figure A3.1 uses three diagrams to illustrate the mechanical arrangements of the 84 mm OD. device; Figures A3.2 to A3.6 reproduce the cavity strain-pressure curves determined in five successful tests. Interpretations were made by Stressprobe using the analysis of Gibson and Anderson (1961), and the deduced values of G and Cu are summarized in Table A3.1.

A3.3 Inspection of the unload-reload loops of the five tests shows the results to radically depart from ideal linear behaviour. The data are reinterpreted in Chapter 8, using the analysis of Palmer (1972) and the results of non-linear elastoplastic finite element studies.

Depth	G	Cu
metres	Mpa (2% reload)	Kpa
5.2	19	162
29.2	27	260
40.2	23	245
49.4	25	245
69.3	29	355

Table A3.1. Pressuremeter data interpreted by Stressprobe using Gibson and Anderson (1961) analysis

APPENDIX A4

Self Boring Pressuremeter (SBP) tests at Canons Park

A4.1 The SBP tests were carried out at Canons Park by P.M. Insitu, and are reported in their document, T76-R1. The work was performed between the 18th and 21st June, 1984 and consisted of 10 inflation tests, nine of which were carried out in a single hole located adjacent to the two boreholes indicated in Figure 9.1. The remaining test was carried out at shallow depth at a point nearer to the main BRS testing area.

A4.2 The pressure-volume curves are reproduced in drawings A4.1 to A4.10. These were analysed in accordance with the method of Gibson and Anderson (1961) and the interpreted values of G and Cu are given in Chapter 7. More detailed plots were supplied by the operators at a later stage and these form the basis of the comprehensive analysis given in Chapter 8.

A4.3 The procedures for the operation of the instrument were essentially the same as those described By Windle and Wroth (1977).

APPENDIX A5

Calculation of Non-Linear Parameters from Triaxial Test Data

A5.1 The finite element studies considered in Chapters 8, 10, and 11 employed an empirical non-linear, undrained model. In order to derive the parameters for this formulation, test data are plotted as normalised secant stiffness against $\log_{10}\varepsilon_A$ and comparisons are made with the periodic properties expected from Equation 8.1. The parameters A, B, C, α and γ can then be found from the following procedure;

1. First locate the observed, or projected, maximum stiffness point from the stiffness, log strain curve. Maxima for equations such as 8.1 occur when :

$$\cos \alpha(\log_{10} \frac{\varepsilon_a}{C})^{\gamma} = 0$$

i.e. when $\alpha(\log_{10} \frac{\varepsilon_a}{C})^{\gamma} = (2\pi)n$ Eq. A5.1

assuming n = 0 for the observed maximum

gives $\log_{10} \frac{\varepsilon_a}{C} = 0$ and C = A

2. Next locate the crossing point where the angular part of Equation (A5.1) must equal $\pi/2$ so

$$\alpha(\log_{10}\frac{D}{C})^{\gamma} = \frac{\pi}{2}$$ Eq. A5.2

then locate the minimum point where the angular part must equal π, so

$$\alpha(\log_{10}\frac{E}{C})^{\gamma} = \pi$$ Eq. A5.3

dividing it is possible to find γ as

$$\left[\frac{\log_{10} E/C}{\log_{10} D/C}\right]^{\gamma} = 2$$

$$\text{and } \gamma = \log_{10}2/\log_{10}\left[\log_{10} E/C \,/\log_{10} D/C\right] \quad \text{Eq. A5.4}$$

3. From the result of Equation A5.4, α can be obtained from Equation A5.2

$$\alpha = \frac{\pi}{2}/(\log_{10}\frac{D}{C})^{\gamma}$$

4. The two scaling parameters A and B can be taken directly from the stiffness values corresponding to the strains C, D and E.

5. Equation A5.1 should then be evaluated for a number of points to find the degree of departure from the test data near the upper and lower limits of strain. The limits should be selected to prevent negative tangent stiffnesses from being predicted, and the lower limit should not usually be less than 0.001%.

6. The maximum, minimum and crossing points can be reselected if the degree of fit is unsatisfactory.

7. If it is required to evaluate the expressions for strains less than C, problems will arise when raising the logarithmic terms to a fractional power. In this case pre and post multiplication by -1 will be required.

A5.2 The general form of Equation 8.1 is also used in the more complex non-linear models described in Chapter 10, and for these cases the same procedure is retained for obtaining the requisite parameters.

APPENDIX 6

Development of an Offshore Settlement Gauge for the Magnus Foundation Monitoring Project

A6.1 INTRODUCTION

A6.1.1 In late 1980 Imperial College was invited to design and construct an offshore settlement gauge (OSG) for the Magnus Foundation monitoring project. It was intended to install the equipment in protective packages attached to the base of Leg A4, shortly before float out. The gauges were to be deployed by divers after all piling operations had finished, but before the larger topside modules had been placed. The main requirements for the gauges were; remote operation ability, good sensitivity and long-term reliability. The target accuracy set by the FMP managers was ± 1 mm, this limit was derived from the expected settlements of 25 to 30 mm for static loading and 50 to 60 mm under the worst anticipated storm.

A6.1.2 The principal design problems were associated with the harsh environment for which the gauges were intended. The operating depth was to be 186 metres below sea level, maintenance or replacement was not feasible, and the equipment had to withstand exposure to long-term corrosion and marine fouling. The equipment could not be sensitive to fluctuations in sea-bed pressure, but should be capable of sensing wave-induced cyclic movements of the platforms piled foundations. The equipment had to be sensitive, but also robust and suitable for diver deployment.

A6.1.3 A barometric working principle was chosen for the new gauges. The system comprises two main parts, as outlined in Figure A6.1, with an "active pot" and a "passive pot". The passive pot is fixed to the sea bed some distance from the structure (40 metres in the case of Magnus) and transmits the ambient sea pressure through separate flexible membranes to two parallel hydraulic systems. One is mercury filled, the other contains oil, but both run through stiff lines from the passive to the active pot, which is fixed to the structure. Inside the active pot the hydraulic lines divide through stainless steel manifolds so that one or more differential pressure transducers can be used to sense the relative pressure between the oil and mercury lines. Any changes in the level of the active pot will result in a corresponding change in differential pressure, Δp^*, which is equal to the

change in level multiplied by the difference in unit weights of mercury and oil. Calibrations can be made to relate changes in electrical output to relative heave or settlement. It will be seen that Δp^*, and hence the settlement measurements, are independent of the ambient pressure at the passive pot. The recordings are thus unaffected by tidal or other variations in mean sea level.

A6.1.4 In order to produce an operational gauge system a series of design studies and laboratory evaluations were required. These included;

(i) A parametric study to evaluate the potential of such a device, and its sensitivity to a number of variables.

(ii) A search for suitable differential pressure transducers, and bench tests to assess the performance of selected instruments.

(iii) A study carried out in co-operation with B.P. to assess methods of deployment, and the best means of interfacing with the FMP power supply, cabling and data logging arrangements.

(iv) Trials with professional divers to assess methods of datum pillar installation.

Following from these studies two prototype gauges were constructed, tested under simulated field conditions, and installed on the structure.

A6.2 FEASIBILITY AND PARAMETRIC STUDIES

A6.2.1 The dimensions and weights of the components were severely limited by the requirements of diver deployment. The hydraulic cables had to be of such a size and stiffness that a 55 metre coil could be payed out by hand, and the active pots could weigh no more than 25 kg submerged, each.

A6.2.2 Parametric studies were also required to assess the most suitable range and sensitivity for the gauges, the equipment's ability to respond to rapidly changing settlement and the effects of cyclically varying ambient pressure. The accuracy and span of the system is firstly limited by the differential pressure transducers selected. Differential pressure devices are essential; the high ambient pressures discount the use of even the most sensitive matched pair of total pressure instruments, although such devices would offer a very large span of measurable settlement. At an early stage, the FMP managers decided that the active pot should be hung from a ladder attached to Leg A4, and the spacings of the rings defined a first minimum to the working range. The use of diver deployment meant that changes in relative level of up to 1.5 metres might occur during installation, and this defined a minimum overload capacity for the transducers.

A6.2.3 It was decided that ± 0.35 bar transducers would be given the most suitable range. The target for the combined repeatibility of transducers, signal conditioning and data logging was ± 0.3% of full-scale operation (F.S.O.) as this gave a gauge accuracy of ± 0.8 mm. In a similar way, the maximum acceptable drift was 0.3%, and the overall working range for the settlement gauges would be ± 270 mm. The minimum overload protection was thus six times the F.S.O.

A6.2.4 The ability of the equipment to measure rapidly changing settlement depends on the hydraulic compliance of the system, and the viscous resistance to flow. The most simple model is the case of twin stiff, rough, pipes connecting two fixed infinite reservoirs through a piston retained by springs, as shown in Figure A6.2. Instantaneous settlement can then be considered equivalent to a rapid rise in the pressure at point x, by $\Delta P_x(t = o)$. Flow will then commence between points x and y at a rate q where;

$$q = K\Delta P_x(t) \qquad \text{Eq. A6.1}$$

(in this case K is the mean resistance to flow of the oil and mercury lines).

The flow arriving at point y displaces the piston, and the pressure increases at point y according the expression;

$$\Delta P_y(t) = \frac{\Delta V}{R} \qquad \text{Eq. A6.2}$$

where R is the volume-pressure compliance of the piston-spring system and $\Delta V = \int_o^t q\, dt$, combining these expressions gives a differential equation whose solution is;

$$\Delta P_x(t) - \Delta P_y(t) = A\, e^{\frac{-Kt}{R}} + B \qquad \text{Eq. A6.3}$$

For the given boundary conditions the degree of equalisation can be written as :

$$E(t) = 1 - \frac{\Delta P_x(t) - \Delta P_y(t)}{\Delta P_x(t - o)} = 1 - e^{\frac{-Kt}{R}} \qquad \text{Eq. A6.4}$$

A6.2.5 The response characteristics of the gauge can then be evaluated by substituting values of K and R into Eq. A6.4. Laboratory tests were carried out to evaluate K for various tubing types, and with the selected 55 metre lengths of twin $^3/_{16}$" nylon tubing encased in polythene, K was found to be 0.087×10^{-8} $m^3/s/Kpa$. The stiffness of the cable was then evaluated by installing a length in a large triaxial cell and applying hydrostatic pressure, with the saturated lines connected through a volume gauge to a back pressure line. The volume stiffness of a 65 metre length was thus assessed as 5.5×10^{-10} m^3/Kpa. From manufacturers' specifications, the compliance of a large strain-gauged beam differential-pressure transducer was assessed as 0.57×10^{-8} m^3/Kpa, and that of a semi-conductor diaphragm device as 3.4×10^{-13} m^3/Kpa. Trapped air might possibly give a further system compliance, which can be calculated as V_o/P_T with V_o as the volume of air at atmospheric pressure and P_T the working pressure in bar.

A6.2.6 The study of the responses to sudden settlement can be extended to cover sinusoidally varying settlement, by applying the

analagous solutions for piezometer response given by Hvorslev (1949). The ratio R/K exactly corresponds to Hvorslev's basic time lag parameter T, and can be substituted into his equations for amplitude factor (A.F.) and phase shift;

$$\text{A.F.} = \frac{\text{observed settlement amplitude}}{\text{actual settlement amplitude}} = 1/\sqrt{[1 + (2\pi T/T_\omega)^2]} \qquad \text{Eq. A6.5}$$

$$\text{Phase Shift} = \frac{1}{2\pi} \arctan 2\pi \frac{T}{T_\omega} \qquad \text{Eq. A6.6}$$

In this case $T = R/K$ and T_ω is the period of the settlement wave form.

A6.2.7 A range of configurations are evaluated in Table A6.1. It can be seen that a semi-conductor device gives an excellent performance, and that penalties are incurred in selecting a more compliant bonded strain gauge transducer. Shortening the cables, thorough deairing and increasing the bore size all give improvements in the characteristics.

A6.3 THE EFFECTS OF SEA BED PRESSURE VARIATIONS

A6.3.1 The use of double hydraulic lines and differential pressure transducers ensures that the slow tidal changes in sea pressure do not effect the operation of the gauge. Indeed if the two lines could have identical dynamic behaviour then the gauge could operate under the most rapidly fluctuating conditions. However, there generally will be differences between the lines and these may lead to measurement errors during severe storms.

A6.3.2 The most simple model for analysis is that shown on Figure A6.2. No interaction of the lines is permitted through the transducer, which is considered as two separate, infinitely stiff, total pressure gauges. For a sinusoidally varying sea pressure any difference in the velocities of wave propagation in the two lines will generate a false "settlement wave" at the transducers. Considering an ambient pressure variation P(t), then;

$$P = A\sin\omega t \quad \text{and} \quad \omega = \frac{2\pi}{T_\omega} \qquad \text{Eq. A6.7}$$

$$\text{hence} \quad \frac{dp}{dt} = \omega A \cos\omega t \qquad \text{Eq. A6.8}$$

If the velocities in the twin lines are V_{Hg} and V_{oil} respectively, then a time lag is predicted for a length L of;

$$\delta t = \left(\frac{1}{V_{Hg}} - \frac{1}{V_{oil}}\right)L \qquad \text{Eq. A6.9}$$

Thus the differential pressure wave can be expressed as

$$\Delta P^* = \delta t \omega A \cos \omega t \qquad \text{Eq. A6.10}$$

The proportion of the sea bed pressure which appears as a false differential pressure ripple can thus be written as;

$$\frac{\Delta P^*}{\Delta P_{max}} = \frac{2\pi\delta t}{T_\omega} \qquad \text{Eq. A6.11}$$

A6.3.3 Reference to tables of physical constants shows the speed of sound in oil and mercury to be comparable, and only small values of δt might be expected. However the velocity of wave propagation in a tube is governed by the equation

$$V_s = \sqrt{1/C\gamma}\ , \text{ Francis (1969)} \qquad \text{Eq. A6.12}$$

where C is the overall compressibility and γ the bulk density. If the contribution of the tubing and trapped air are large compared to the compressibility of either fluid, then significant differences can be expected between V_{Hg} and V_{oil}. In fact, the previously described tests showed the tubing to be relatively compressible and Equation A6.12 gave values of 1,500 m/s and 500 m/s for water and mercury filled lines respectively. Calculations can also be made to consider oil filled tubing and the effect of undissolved air.

A6.3.4 An experimental check was run using the scheme shown in Figure A6.4. The time taken for a shock wave to pass down a carefully deaired 50 metre length of twin $^{3}/_{16}$" O.D. hydraulic cable was measured for both mercury and water. Figures A6.5 and A6.6 show the oscilloscope traces and indicate speeds of around 500 ms^{-1} and 1,500 ms^{-1} for the mercury and water lines respectively. These velocities agree closely with those expected from calculation. Substituting values of compressibility for the transformer oil used in the settlement gauge lines indicates a time lag, δt, of 0.09 seconds for 65 metre long cables.

A6.3.5 To evaluate ΔP^{*} from Equation A6.11 it is necessary to know ΔP_{max}, the pressure variation induced by waves at the sea bed. Weigel (1964) gives linear wave theory equations for the amplitudes of such effects at the sea bed. For surface periodic waves of amplitude B, the theory predicts that similar period pressure waves will act at the sea bed with amplitude A, where A = B.K and,

$$K = {}^{1}/\cosh 2\pi\, d/L_{\omega} \qquad \text{Eq. A6.13}$$

In this case L_{ω} is the surface wave length, and d is the depth of water.

Wave length is related to period by the wave velocity c, with $L_{\omega} = cT_{\omega}$. The length can be found for a given period by iteratively solving a second equation relating C and L_{ω};

$$c^{2} = g \frac{L_{\omega}}{2\pi} \tanh 2\pi \frac{d}{L_{\omega}} \qquad \text{Eq. A6.14}$$

A6.3.6 Table A6.2 sets out values of K and L_{ω} for a range of wave periods and bottom depths. At 186 metres depth, the Magnus sea bed would only experience significant pressure variations when the wave period exceeds 16 seconds, and the wave length approaches 0.5 Km. These conditions correspond to the most severe storm considered by the platform designers. Data from the Institution of Oceanographic Sciences was used to estimate the values of T_{ω} and B for the 50, 10 and 1 year return periods at Magnus. Values of K were calculated, and

used to predict the settlement ripple effects developed during the considered storms. The results are tabulated in Table A6.3 and show that significant errors will only occur under the most extreme conditions.

A6.4 DATUM INSTALLATION METHODS

A6.4.1 The efficiency of the system depends on the passive pots remaining at the same level throughout the life of the gauge. It was considered that a light, unfixed, datum package might be unable to resist current action or scour effects. Similarly, a heavy unfixed package might generate its own settlement displacements, as the properties of the upper most sea bed soils had been shown by site investigations to vary from soft to stiff clay. It was therefore decided to fix the datum to the sea floor by means of miniature piled foundations which should allow good stability with negligible settlement.

A6.4.2 Various methods of producing such foundations were considered including; placing mini-piles using site investigation sea bed jacking units, jacking medium sized piles from reaction frames attached to a brace of the structure, employing specially designed small diver actuated suction piles and finally mini-piles installed by divers using hand-held hydraulic tools. The last listed method was selected as being the most suitable for the Magnus project, and a trial was held using Shiers Diving Contractors to evaluate a range of pile types and underwater tools.

A6.4.3 The trials were carried out on land at the Building Research Station's Hendon pile testing site. The London clay profile had been extensively investigated and the upper two metres classified as firm to stiff clay. The installation of screw auger and pipe piles by hydraulic hammer drills, impact wrenches and other tools was assessed. Overall, 38 mm diameter two metre long pipe piles, installed by impact wrenches, gave the best combination of ease of penetration and resistance to vertical and horizontal force. A single pile gave a good foundation at Hendon but, to cater for the possibility

of worse conditions at Magnus site, a group of 3 such piles was selected as the most appropriate datum foundation for use offshore.

A6.5 EVALUATION OF TRANSDUCER AND SIGNAL CONDITIONING PERFORMANCE

A6.5.1 The qualities sought in the selection of wet/wet differential pressure transducers can be summarised as; compactness, accuracy, high overload capability, chemical compatibility with oil and mercury, low compliance, good stability in the long-term and electrical compatibility with other FMP systems. After discussions with many manufacturers the choice was restricted to devices made by Bell and Howell, Sensotec and Druck. Samples of each type were purchased and extensive tests made to ascertain their performance. The Bell and Howell transducers were not finally used due to their low overload capacity, and it was decided to employ a single Druck and Sensotec gauge in each of the two prototype gauges. The manufacturers' specifications for the three considered devices are summarized in Table A6.4.

A6.5.2 To carry out suitable tests the calibration system sketched in Figure A6.7 was built. The transducers were carefully deaired by water filling, and then rigidly mounted inside a refrigerator, which was set at the expected field temperature. Water filled lines were taken from the transducers to a twin high pressure water supply source, which had been converted from a Bishop and Henkel (1962) standard triaxial pressure control system. The water-air manometer arrangement was assembled to measure the differential pressures, and a high pressure air supply was controlled through a Manostat valve to balance the water pressures. An Invar tape was used to measure the water levels in the three metre high manometer. The system allowed line pressures of 0 to 700 Kpa, and differential pressures of ± 0.3 bar, to be applied. The overall accuracy of measurement was ± 0.0001 bar, provided that a few drops of detergent were used to stabilise the menisci.

A6.5.3 Multiple calibrations were made of the transducers alone, and the transducers with their signal conditioning cards attached. The tests were continued for a three month period and the results may be summarized as follows;

(i) The performance of the Druck transducers improved with time. The mean errors from a best fitting straight line (BSL) decreased from initial values of ± 6 mm of water, to ± 3.5 mm with use. The Sensotec devices consistently gave mean errors in water head of ± 2.5 mm.

(ii) The Drucks showed an initial drift rate of around 0.01% FSO per day under sustained load, but this decreased with time. The Sensotecs showed no measurable drift.

(iii) The Sensotecs showed a less significant dependence on line pressure.

(iv) On initial energisation, the Druck devices did not become stable until 10 minutes had elapsed. The Sensotecs showed no warm-up effects.

(v) The Sensotec gauges showed far less scatter with time for sustained zero differential pressure.

(vi) With the signal conditioning cards connected, the combined systems showed no losses in accuracy or stability.

A6.6 CONCLUSIONS FOR GAUGE DESIGN

A6.6.1 The parametric studies showed that it was feasible to meet the FMP managers' specification using the differential pressure and dual-fluid system. The most difficult choice in the design was the selection of differential pressure transducers; calibrations showed the light, sensitive, Druck devices to have adequate accuracy and stability but to be less satisfactory than the Sensotec transducers. However, the use of twin Sensotecs would severely impair the response characteristics of the gauges and make the active pots unacceptably

bulky. Both devices appeared to have adequate overload capacity, but the near infinite protection of the Sensotec gauge offered a clear advantage. It was finally decided to deploy a transducer of each type in the two prototypes, and this offered some insurance against long-term problems with any single device.

A6.6.2 With this configuration, the gauges were expected to give an overall performance better than the specification. The dynamic properties should also have been adequate, and the diver trials had indicated that installation of the gauges and their datum foundations should be within the abilities of saturation divers.

A6.7 DETAILED DESIGN OF PROTOTYPE GAUGES

A6.7.1 The size and weight of the prototype gauge system was limited by the feasibility of diver deployment. It had been decided at an early stage that all electrical connections would be made onshore at waterproof junction boxes, with the two gauge systems being packed in protective boxes welded to the pile template cluster of Leg A4. It was planned that, after float out and platform installation at Magnus, divers would carry out the following tasks;

(i) Install datum foundation piles on the sea bed approximately 40 metres from Leg A4.

(ii) Open the protective boxes, take out the passive pots and hydraulic cables, move to the respective datum fixing point and place the datum.

(iii) Position the active pot, under guidance from a monitor on the surface, in its optimum position on a special mounting ladder fixed to the leg.

A6.7.2 The considerations of diver deployment, robustness and durability were uppermost when undertaking the detailed gauge design. The active pots were made from flanged mild steel tubing as shown in Figure A6.8. The lower flange pieces were welded onto the tubing

which was then turned down to the smallest wall thickness (6 mm) which would not give excessive distortion when the hydraulic cable and bosses were attached by welding. The upper flange plates consisted of 6 mm mild steel disks onto which electrical gland bosses had been welded. The lower flange plates were reduced in thickness over their central portion to produce a flexible membrane 1.5 mm thick. The diameter of the steel tubing was the smallest that could feasibly contain a Sensotec and a Druck transducer with their signal conditioning. (See Figure A6.9). Each flange was carefully machined flat with grooves for 'O' ring seals.

A6.7.3 The active pot flexible flange was designed to transmit sea pressure to the inside of the pot, which was to be oil filled, so that the risks of ingress of sea water and leakage from internal hydraulic fittings could be minimised. However, the possibility that air might be trapped inside the active pot during assembly meant that the construction should be able to stand large pressure differences without collapsing. Stops were provided to prevent a collapse in the case of an oil leak, with a maximum volume change of approximately 20 cc being permitted.

A6.7.4 The electrical and hydraulic cables penetrate the walls of the active pot through glanded terminations. Both types of cable are sleeved in polythene and specially designed mouldings were made which water stopped each cable component and presented an 'O' ring seal to the boss face. The upper and lower flanges of the active pot were sealed with a double 'O' ring arrangement, and all the welds were resin pressure impregnated to guard against porosity using the "Ultra Seal" proprietary process.

A6.7.5 Inside the active pot the hydraulic leads were separated and connected to two stainless steel manifolds, from which hydraulic connections were made to the transducers. All the metallic components in the hydraulic system (fittings, olives and transducer cavities) were made from stainless steel in order to avoid mercury corrosion problems. The hydraulic cable selected for the prototypes was a standard product used in hydraulic piezometer systems for use in Earth Dams.

A6.7.6 The selection of an electrical cable was more difficult. Ideally, the cable would be configured with eight cores. These should be arranged in twisted pairs of tinned stranded copper conductors sheathed in polypropalene with some armour protection and an overall waterproof sheath of polythene. An additional sheathing of polyurethane would also have been advantageous. The polythene would be the main waterproofing, the polypropalene would give the advantage of not melting during gland moulding operations, and the polyurethane gives the toughest outer protection. In fact, custom made cables would have been prohibitively expensive and a BICC standard product with 10 cores arranged into twisted pairs individually sheathed in polythene, with an outer polythene sheath was selected. The electrical and hydraulic glands were custom designed and manufactured. Special cooling arrangements were made to prevent fusion between the insulation of individual conductors.

A6.7.7 The passive pots were designed to be fluid reservoirs each with a highly flexible membrane interface with the sea pressures. This was achieved using the arrangement of two connected, open ended, stainless steel pots with "Bellofram" fabric reinforced elastomer rolling diaphragms; Figure A6.10 shows the general arrangement. The connecting stainless steel piece served as a boss for the hydraulic cable terminating gland, and was mounted on a steel plate. The base of the plate had fittings to allow a solid fixing to the datum pillar. Isolating nylon bushes and rubber gaskets were used to prevent corrosion at stainless/mild steel contact points.

A6.7.8 It was considered desirable that the mild steel parts of the settlement gauge should not corrode unduly over the working life of the device. A brief literature review suggested that unprotected steel lying in the sea water immersion zone, and away from heat sources such as riser pipes, should suffer an initially high rate of erosion followed by a steady rate of "thickness loss" of around 0.08 mm/yr. It was judged that the minimum steel membrane thickness should be 1.5 mm and that all mild steel parts should be protected by sand blasting followed by the application of two coats of Epoxy coal tar paint.

A6.7.9 The gauges were also designed to resist marine growth, as hydroid development, encrustation or sedimentation could render the flexible diaphragms inoperative, make any insitu inspection difficult and possibly hinder the divers in opening the protective boxes. Advice was sought from Aberdeen University Zoologists, and specialists at B.P's Environmental Control Centre. The consensus was that unprotected gauges could be fouled relatively quickly by a variety of organisms. It was therefore decided to take anti-fouling measures.

A6.7.10 All the mild steel components, including the protective covers, were coated with Takata LLL paint applied by airless spray. This compound is a tin copolymer self-polishing anti-fouling paint which leaches lethal ions for a period of at least three years. The Belloframs of the passive pots were covered with Cupro-Nickel/resin protective caps. These specially made components create a similarly deadly environment around the flexible covers, and this should persist for decades. Ports were positioned to allow pressure equalisation and a flushing action for the removal of detritus. The covers were manufactured by the United Wire Group of Edinburgh, and rubber 'O' rings were used to isolate the Cupro-Nickel resin from the stainless steel passive pot components.

A6.7.11 Signal conditioning was provided within the active pots by purpose made printed circuits. The cards regulated the transducer 10 volt supplies within tight limits and amplified the bridge outputs to provide highly stable signals in the range ± 3.5 volts. The circuits were designed for an external nominal supply voltage of 60 volts and could maintain a constant bridge supply in the event of supply variations as large as ± 20 volts. The overall power requirement was 9.6 watts per settlement gauge, and buffer circuits were provided to protect the gauges against power 'spikes' and other problems. The amplification circuits were designed to make the gauge outputs insensitive to changes in the resistance of the long cable runs to the surface. As a further water proofing precaution, the cards were potted solid in polysulphide epoxy resin blocks using the Berger PR 905 two-part mixing compound.

A6.7.12 Up to the point of diver deployment, the gauges were housed in two cylindrical boxes, each one metre in diameter and 0.5 m deep. These drums were welded to Leg A4 and were of simple design. A point of interest was the selection of foam materials for the boxes. The lids were heavy, and B.P. asked for permanent buoyancy aids to be provided which should not crush under the full sea bed pressure. Conversely, the interior of the drums required a pair of complex foam mouldings to retain and protect the gauge components. In the latter case the foam had to lose buoyancy when compressed, so that no threat was posed to divers opening the boxes, but to remain an effective cushion under all conditions.

A6.7.13 Experiments were carried out in the laboratory on various products and a rigid PVC foam, Permacell D150, was selected for the first application, and a brittle open cell rigid polyurethane foam for the second. The cell walls of the polyurethane foam crack with pressure and the air spaces flood without large overall strains developing. Polystyrene foam in contrast, was seen to compress virtually in accordance with Boyle's Law, and would have reduced to only a few per cent of its original volume at the maximum field pressure.

A6.7.14 The assembly of the gauges was straightforward, except for the deairing of the hydraulic lines. A limited head vacuum-draw arrangement was finally employed, but the manifold system inside the active pot resisted many efforts to extract the final bubbles of air. Two full weeks of deairing were required to satisfactorily saturate the hydraulic system.

A6.8 EVALUATION OF THE PROTOTYPE GAUGE PERFORMANCE

A6.8.1 Having assembled the gauge systems, it was important to carry out a series of proving and calibration tests. The programme included tests using less than the field line pressures in the laboratory, and tests carried out at working pressures using a large hyperbaric chamber. The main aims of the programme were to examine;

(i) The overall integrity of the system under simulated working conditions (i.e. structural design of pots, waterstopping, electrical performance, reversibility of flexible diaphragms etc.)

(ii) The response times for the gauge systems.

(iii) The linearity and hysterisis of the systems.

(iv) The calibration constants for each gauge.

(v) The effect of varying hydraulic line pressures.

A6.8.2 The various test series were carried out using the calibration frame sketched in Figures A6.11 and A6.12. A small lathe bed was converted to hold the passive pot in a fixed position whilst a screw thread was turned to raise or lower the active pot in a controlled manner. The frame could be either mounted in the laboratory or in the hyperbaric chamber. For the latter case the drive handle had to be taken through the wall of the pressure vessel with a glanded penetrator; to keep track of the changes in relative level between the pots, an electronic revolution counter was also employed. It was decided to carry out the hyperbaric tests first, and to use the laboratory work to complete any unfinished parts of the test programme.

A6.8.3 The hyperbaric tests were conducted at the National Environmental Research Council's (N.E.R.C.) oceanographic vessel support base at Barry, South Wales. A 2-metre high, 750 mm diameter chamber was hired, and Custom Design Mouldings of Cwmbran were contracted to assemble calibration equipment and fabricate cable penetrators for the tests. The Barry experiments demonstrated the overall integrity of both prototype gauges and showed a good overall performance for Gauge No. 1. However, Gauge No. 2 gave poor response and calibration characteristics; these malfunctions were thought to

have been caused by air trapped in the hydraulic system. After testing at Barry both gauges were reconstructed, with minor modifications being made to facilitate deairing. The devices were then recalibrated under line pressures of 700 Kpa in the laboratory at Imperial College.

A6.8.4 The performance of Gauge No. 1 was assessed with line pressures of 1,900 Kpa in three main test series using the pressure vessel. Firstly, a response time check was made by connecting the outputs from the Druck and Sensotec transducers to a dual beam storage oscilloscope. A rapid change in settlement was applied by winding the calibration frame down one revolution, and the response was monitored. Equilibration was effectively complete within 30 seconds and t_{95} could be estimated from the graduated screen as 20 seconds. This result is in accordance with the previously discussed theory.

A6.8.5 The second, calibration, series was carried out by raising and lowering the active pots through 320 mm travel in 20 mm stages. The data were analysed for the full load-unload loop and the mean error from the best straight line was ± 0.25 mm for both transducers; this corresponds to ± 0.06% F.S.O. It is interesting to note that when the errors from the best fitting straight lines for both transducers are plotted as a scatter diagram, as shown on Figure A6.13, they fall near to the 45° line. This infers that the errors were either in the hydraulic system or the measuring of the relative displacement. The transducers were performinig at least as well as they had in the laboratory.

A6.8.6 Finally, tests were carried out to investigate the effect of sinusoidally varying cell pressures. An electric motor was used to drive a crank through a gearbox, and this delivered a cyclic linear motion to a piston connected into the vessel pressure supply. A Bell and Howell total pressure transducer was used to monitor the cell pressure variation and the oscilloscope was connected to study the effect on the two transducers. As theoretically predicted, the outputs showed a periodic response to the cell pressure fluctuations, which was of the same frequency as the forcing wave, but with greatly

reduced pressure amplitudes. To assist the quantitative analysis, the outputs were also digitally recorded with a data logger, and the records showed a 1.7% differential pressure response for both transducers to a forcing wave of 20 second period. These results are also within the range expected from the parametric study.

A6.8.7 At the end of the Barry tests the gauges were modified and reassembled. Caps were placed over the passive pot reservoirs so that the system could be calibrated in the laboratory whilst subjected to an elevated line pressure of 700 Kpa. The response time checks were repeated, and Figure A6.14 shows the results for the reconstructed Gauge No. 2. The data show that both transducers were sensibly in equilibrium 20 to 30 seconds after winding had been completed.

A6.8.8 Calibrations were carried out using both continuous and intermittent powering. The latter case was investigated as the field operation power requirements of the gauges was such that only a 12.5 minute average energisation time could be given before each reading. The resuls were analysed by linear regression and the deviations assessed. In fact, the mean errors were almost double those observed in the pressure vessel tests. When plotted in the form shown in Figure A6.13 the points again group around the 45° line, but the mean error of ± 0.68 mm still amounts to less than 0.15% F.S.O. It is thought that the increased errors result from building vibrations, temperature changes and uncompressed air bubbles. If so, the performance of the gauges in the field should improve as the conditions are steady and elevated pressures reduce the volume of free gas present in the system. The final calibration factors are listed in Table A6.5.

A6.8.9 Tests were also made in the laboratory to simulate cyclic variations of ambient pressure and settlement. The pressure variation tests used a smaller version of the crank system deployed at Barry, but this showed rather larger response factors which did not vary greatly with wave period. The results were considered to result from the low line pressures, which would give far higher overall fluid compressibilities than the hyperbaric tests for similar quantities of trapped air. The cyclic settlement tests showed amplitude factors of

around 0.5 for T_ω = 20 seconds. This result is in fair agreement with the theoretical prediction, and an improved factor could be expected after prolonged immersion at elevated pressure.

A6.8.10 In summary, the calibration checks and other trials proved the gauge's performance to be generally satisfactory. The accuracy in the field should at least meet the ± 1 mm specification, and the device should be unaffected by all but the most severe storms. The dynamic performance of the gauges is such that cyclic motion of the platform foundations can be sensed, but the amplitude recorded will be between 50 and 100% of that taking place. The hyperbaric tests showed the overall design of the system to be suitable for prolonged immersion at high pressure, and the diver trials proved the feasibility of underwater deployment.

A6.9 DEPLOYMENT OF THE GAUGES

A6.9.1 The development, construction and calibration period of the settlement gauge systems was from February to September 1981. The equipment was taken from Imperial College in early October 1981, and transported to Highland Fabricator's Nigg Bay construction yard sited near Inverness. In March 1982 the author was called to site to supervise the installation of the gauges onto Leg A4. This was carried out without difficulty, except for the onsite moulding of the electrical cable into a potted junction box; this sensitive operation took several attempts. Tests at Nigg Bay showed the gauges to be functioning correctly with good power supply and signal return links. Figures A6.15 and A6.16 show the equipment being installed on Leg A4. Following these operations the equipment became the responsibility of British Petroleum, and the platform was floated out shortly after.

APPENDIX A7

Notation Employed

APPENDIX 7

NOTATION EMPLOYED

A to E	Constants
B	Width
B	Pore pressure Parameter
C_c	Compression Index
C_s	Swelling Index
C_u	Undrained shear strength (with sub and superscripts for plane strain, triaxial extension etc.)
C_{ub}	Undrained shear strength at pile base
C_v	Coefficient of consolidation
C_{vs}	Coefficient of swelling
D	Diameter
E	Young's modulus (E_p is pile Young's modulus)
E	Shear strain invariant $2[^1/_6((\varepsilon_1-\varepsilon_2)^2 + (\varepsilon_2-\varepsilon_3)^2 + (\varepsilon_1-\varepsilon_3)^2)]^{1/2}$ (E_p is plastic part of E)
E_u	Undrained Young's Modulus ($E_{u(0.01)}$ is modulus at 0.1% strain etc.
F	Intake Factor

G	Shear Modulus
H	Height, Hydraulic Head
J	Shear Stress Invariant $[^1/_6((\sigma'_1-\sigma'_2)^2 + (\sigma'_2-\sigma'_3)^2 + (\sigma'_1-\sigma'_3)^2)]^{1/2}$
K	Stress ratio σ'_3/σ'_1 [Special cases K_o, K_a, K_p]
K'	Bulk Modulus
L	Length, measure of linearity, L_w wave length
L(ε)	Linearity - strain function
OCR	Overconsolidation ratio [related to σ'_v]
P	Force, pressure
Q	Bearing capacity, Qb base resistance, plastic potential function
S	Proportion of mobilised shear stress, J/J_{max}
SBS	State Boundary Surface
T	Tension load, Time Factor, T_w wave length
V	Volume
X,Y,Z	Parameters describing the shape of the plastic potential
a to e	Constants
b	Intermediate principal stress parameter $(\sigma_2-\sigma_3)/(\sigma_1+\sigma_3)$
c_α	Coefficient of secondary compression

d	Depth
e	Voids ratio, e_1 voids ratio at unit pressure
$f(\varepsilon)$, $g(\varepsilon_v)$	Non-linear stiffness functions
$g(\theta)$	Functions describing shape of yield surface and plastic potentials in the deviatoric plane
k	Coefficient of permeability
i	Hydraulic gradient
l	Length
m_v	Coefficient of compressibility (m_s for swelling)
p'	$(\sigma'_1 + \sigma'_2 + \sigma'_3)/3$, special cases p'_o, p'_f, p'_{cs} etc.
q	Bearing pressure
q	$(\sigma_1 - \sigma_3)$
r	Radius, r_o pile radius
t	Time, thickness
u	Pore water pressure
v	Specific volume, v_1, specific volume at unit pressure
w/c	Water content
z	Depth below ground level
α to ω	Constants

α	Inclination of Hvorslev surface to horizontal
α	Henkel's pore water pressure coefficient, special case αf
δ	Displacement
δ'	Interface friction
Δ	Displacement, special cases Δr etc.
ε	Strain, subscripts; A axial, r radial etc.
ε	Eccentricity
γ	Bulk density, shear strain (subscripts define component)
θ	Angle, (special case lode angle)
k	Swelling line slope in v - lnp' space
λ	Virgin compression line slope in v - lnp' space
v, v'	Poisson's ratio, angle of dilation
σ	Normal stress (σ_1, σ_2, σ_3, σ_a, σ_r, σ_z etc.)
σ'_{vc}, σ'_p	Preconsolidation pressures
σ'	Effective normal stress
τ	Shear stress, subscripts define normal plane and direction of action
ϕ'	Effective angle of friction
ω	Angular velocity
Ω	Interaction factor