DEVELOPMENT OF SOUNDING EQUIPMENT FOR THE ASSESSMENT OF THE TIME-SETTLEMENT CHARACTERISTICS OF RECENT ALLUVIAL DEPOSITS WHEN SUBJECTED TO EMBANKMENT LOADS

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ABSTRACT

Many embankments on the soft, highly variable, recent alluvial deposits along the South African coast have suffered large settlements necessitating ongoing costly repairs.

Due to the soft variable soils, borehole sampling is difficult and laboratory testing requires to be extensive for adequate subsoil modelling; cone penetration testing was considered to be a potential means to overcome these problems. Twenty five years ago in South Africa, as elsewhere, cone penetration testing equipment was relatively crude and the methods of interpretation were simplistic. The application of cone penetration testing to recent alluvial deposits therefore required improvements to both the equipment and the derivation of soil parameters.

The equipment was upgraded by introducing strain gauge load cells capable of measuring cone pressures in soft clays with adequate accuracy. Hence, correlations of cone pressures with compressibility and shear strength became possible.

Predictions of settlement times and magnitudes are of equal importance and a consolidometer-cone system was developed to assess both of these.
A piezometer was incorporated into a cone to ascertain whether the settlements were due to consolidation. The piezometer cone performed so well that it superseded the consolidometer-cone and by 1977 a field piezometer cone was in regular use.
Developments in piezocone interpretation have taken place concurrently with those in equipment; coefficients of consolidation are evaluated from pore pressure dissipations, and soils identified from the ratio of pore and cone pressures.

These developments have been validated in two recent research projects, by comparing measured and predicted settlements at eleven embankments monitored for up to fifteen years. The data shows that for embankments on the recent alluvial deposits the constrained modulus coefficient, α_m is :

$$\alpha_m = 2,6 \pm 0,6$$

The data also shows that coefficients of consolidation from piezometer cone dissipation tests are correlated with those from laboratory tests and back analysed embankment performance as follows :

$$\text{Embankment } c_v = 3 \text{ CPTU } c_v = 6 \text{ Lab } c_v$$

It is concluded that piezometer cone penetration testing is particularly suitable for the geotechnical investigation and the subsequent design of embankments on recent alluvial deposits and should be considered as complementary to boreholes with sampling and laboratory testing. The existing database of embankment performance should be expanded with particular emphasis on long term measurements and on thorough initial determination of basic soil parameters.

DEDICATION

This work is dedicated to

God

Wendy

My family

PREFACE

The whole of this thesis is my work unless specifically indicated to the contrary in the text, and has not been submitted in part or in whole to any other University.

Some thirty years ago the author operated a deep sounding machine, one of the first in the country, on a misty lake in Ireland and marvelled at the way subsoil information could be garnered. The magic of the moment never entirely passed and when the opportunity arose to use the technique in Natal the die was cast.
The development of the national road system surged in the early 1970's and since many of these roads on the Natal coastal routes crossed extensive recent alluvial deposits, the geotechnical problems of instability and settlement became major factors in the road design. Traditional methods of investigation consisted of boreholes with sampling and laboratory testing. Whilst these were satisfactory, provided they were of adequate quality, they were relatively expensive if sufficiently detailed models of the subsoil were to be obtained for design purposes.

Cone penetration testing provided a potential a solution and this led to research work conducted over a period of twenty five years which continues today. The initial development of ideas for improvements to the mechanical equipment took place whilst the author was carrying out preliminary investigations for freeway routes over the coastal alluvial deposits. This was followed by a period devoted largely to cone penetration testing research and development and to embankment design methods at the National Institute for Transport and Road Research, and to the initial registration for a Master's degree under the supervision of Professor K Knight in 1975. This research programme was completed as originally envisaged, but not submitted because during its course the author conceived the idea of the piezometer cone. This proved to be such an exciting prospect that the research and development continued for a number of years until piezometer cone testing has now become almost routine for geotechnical investigations on alluvial deposits. In 1983, due to Professor Knight's retirement from the University, Mr Phillip Everitt was appointed as the supervisor.

At that stage piezometer testing was becoming accepted internationally and new aspects and information frequently appeared. It was apparent, however, that the essential proof of the system for the prediction of embankment performance was to use it at embankments where the performance had been monitored. Eventually grants were provided by the Department of Transport for this, which enabled two research projects to be conducted during 1989 - 1990 and 1991 - 1992. After completion of the first of these a presentation of the author's work on cone penetration testing since the mid 1960's was made to the Faculty of Engineering at the University of Natal. The Executive Committee of the University Senate subsequently approved, in August 1991, that the registration be upgraded to doctoral status.

Mr Everitt's encouragement during this extended period has been a vital factor in ensuring an outcome for this task and the author wishes to express his gratitude for this.

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PART A : INTRODUCTION, PROBLEM DEFINITION AND GEOLOGY

A1 INTRODUCTION

This submission describes a programme of research carried out by the author since 1965 on the development of cone penetration testing. The development includes improvements to the then existing mechanical systems in South Africa and to the interpretation and application of CPT results. Emphasis is placed on the development and incorporation by the author of a pore pressure measuring system into a cone. This enabled much more geotechnical information to be obtained in soft alluvial deposits than was possible with the previously available mechanical standard cone penetration test equipment. The primary reason for requiring the geotechnical information in these materials was that the road design and construction programme in South Africa was rapidly expanded during the 1960's resulting in about 300 kms of the Natal north and south coast freeways from Richards Bay to Port Shepstone being investigated. The numerous rivers along the coast meant that the proposed roads incorporated many river crossings with extensive flood plains. The stringent geometrical standards resulted in bridge approach embankments often of considerable length and significant height. Since the underlying alluvial deposits generally contained thick soft silty clay layers, as well as loose sands, stability and settlement problems were to be expected. In numerous instances the actuality has even exceeded the more pessimistic expectations and settlements of up to 50% of embankment heights, of typically 7 m, have occurred giving rise to severe problems both during and after construction.

Conventional geotechnical investigation in the 1960's, as now, comprised boreholes so that the strata could be defined and samples obtained for laboratory testing. Cone penetration testing was practically unknown other than for investigating potential pile founding depth conditions in sands. The cone testing equipment was crude and although satisfactory for piling investigations, little if any useful information could be obtained for any other purpose. Nevertheless cone penetration testing was a rapid and economical method, compared to boreholes with sampling and laboratory testing, and the potential benefits of cone testing were seen to be significant, the proviso being that it could be developed to give satisfactory results in the soft alluvial deposits.

During the past two decades cone penetration testing has become much more widely used on an international basis with the result that correlations have become available between cone pressures and soil parameters representing shear strength and compressibility. These together with similar locally obtained correlations have been used for design purposes.

It is presently recognized, that until a full theoretical understanding of the mechanics of cone penetration has been achieved, these correlations must remain semi empirical and necessarily require local validation.

Experience world wide and locally has shown that the CPT approach to settlement prediction generally gives satisfactory results for sandy materials, but less so for clays. In South Africa, it has been found that some embankments on soft clays have suffered settlements significantly exceeding those predicted using the international correlations of compressibility with cone pressures. It was not known whether the unexpectedly high settlements were due to unsatisfactory compressibility assessment from the cone testing or for other reasons.

It was decided that investigation into this problem was necessary and therefore two research projects (1989 - 1990 and 1991 - 1992) were undertaken. In these the time-settlement characteristics of a series of embankments along the Natal and Cape coasts, which had been constructed over ten years previously, were back analysed and correlated with piezometer cone penetration test data. This showed that satisfactory predictions of both settlements and time for settlements could be derived from piezometer cone penetration testing.

This thesis is divided into five main parts:

A is introductory and sets out the definition of the engineering problems, and describes the methods of geotechnical investigation available at the time the work began, and also the geological and climatic conditions along the Natal coast.

B is a historical review of cone penetration testing, which also describes the current international methods of interpretation with the emphasis on the parameters required for embankment design on alluvial deposits.

C describes South African developments in CPT; this is prefaced by descriptions of the mechanical cone systems and the author's contribution in introducing and developing improved systems including prototype laboratory and field consolidometer-cones. It also details the development by the author, of first a laboratory piezometer cone and subsequently a field piezometer cone.

D describes the South African application of piezometer cone testing at various embankment sites and in particular to two research projects, funded by the Research Development Advisory Committee of the South African Roads Board, during which piezometer cone testing was carried out on embankments which had been monitored for many years. From these, correlations are derived between cone test data and compressibility and consolidation parameters.

E brings together the research and development programme for cone penetration testing, described in C and D, draws conclusions and makes recommendations for the application of the results to practice.

Appendix I Lists the author's references relevant to cone penetration testing and embankment design and contains copies of those which are in the form of papers; documents and research reports are not included.

Appendix II Contains laboratory consolidometer-cone test results.

It is noted that the original intention when this thesis was registered in 1975 for a master's degree was that the work would be taken to the development of the consolidometer-cone described in Part C. At that stage cone penetration testing had not achieved the prominence it now occupies and Part B would have been much less comprehensive.

The development of the piezometer cone by the author in the mid to late 1970's, described in the last section of Part C, gave greater impetus to cone penetration testing.

This led to many field investigations and to the two research projects described in Part D from which conclusions are drawn regarding the prediction of embankment performance from cone penetration testing. The greatly enhanced capability of cone penetration testing and its validation by experience resulted in the thesis being upgraded in 1991.

A2 PROBLEM DEFINITION

Road development throughout South Africa and particularly in Natal experienced a dramatic increase in the late 1960's and early 1970's. Due to favourable economic conditions and increased traffic growth major roads and freeways were designed and constructed including sections of the N3, Durban-Johannesburg; N2, along Natal south coast; Durban Outer Ring Road, N2, Natal north coast, the Southern Freeway within Durban and other provincial roads. In addition railway and harbour development at Durban and Richards Bay was undertaken.

The new roads demanded high geometric standards. Previously, road designers had been able to bypass many of the worst geotechnical problems caused by adverse geological conditions by utilising lower geometric standards, but the higher horizontal design standards required forced roads through areas of poor subsoils. In addition, the higher vertical alignment standards resulted in embankments being much higher than would previously have been necessary. Another factor which had a bearing was that both the public's and the road designer's expectations for the performance of a road had been considerably heightened largely because of a generally increased demand for performance in all facets of life and because of a public awareness of the high cost of the new roads.

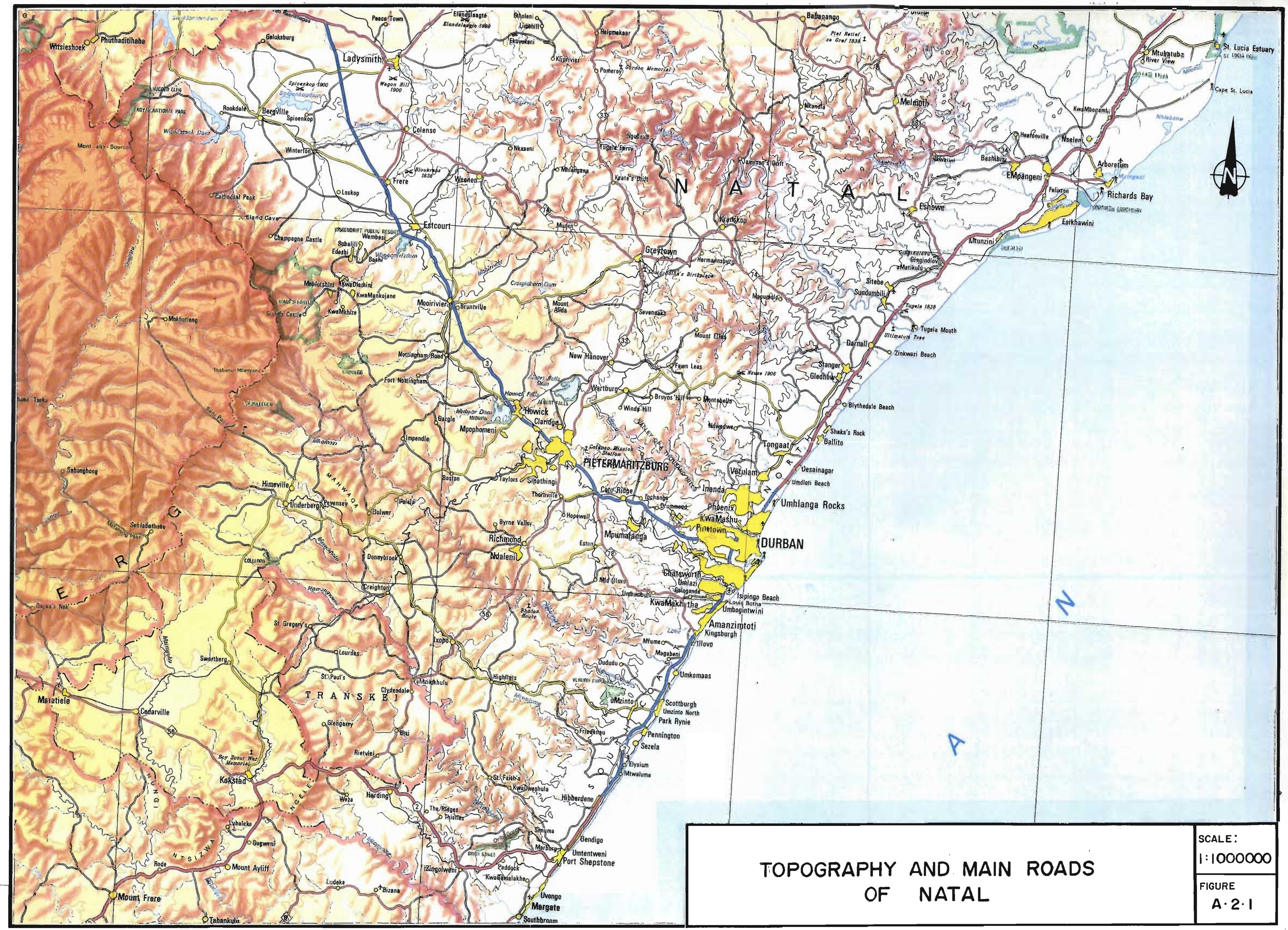
During planning of the Natal freeways considerable discussion took place on the advantages and disadvantages of following a coastal alignment as opposed to a more inland route. Access to the latter from the population centres along the south coast would have been costly, and although the coastal route was known to involve major difficulties with expropriation in the limited corridor available and multiple geotechnical problems at the numerous bridges and flood plains, the inland route, due to the steep topography, would also have resulted in high costs. The coastal route was selected, partly as a result of the conclusion based on preliminary geotechnical investigations conducted by the author, that the geotechnical problems, although formidable, were not excessive.

The new route comprised about 100 kilometres of freeway along the Natal south coast and about 150 kilometres along the north coast. Although not all of this was intended to be initially constructed to complete dual carriageway limited access freeway standard, it nevertheless represented an enormous capital investment in the road infrastructure. During the same period Richards Bay harbour was being developed and with it the new railway line for coal transport from the Reef. The harbour works involved similar extreme geotechnical problems of structures and embankments on deep very soft alluvial deposits and much of the geotechnical data on the Natal coastal soils originates from this area.

The topography of the Natal coast is shown in Figure A2.1. In broad terms this comprises a narrow relatively low lying coastal strip about 2 to 3 km wide in the south from Port Shepstone up to about Mtunzini (Tugela) in the north. From there northwards the coastal strip widens and becomes noticeably flatter so that in the Richards Bay area the coastal plain is about 20 km wide. Although the well populated coastal zone is relatively low in elevation and flat compared with the hinterland, it is nevertheless moderately hilly to the extent that a modern freeway standard road restricted to maximum gradients of about 3% requires fills and cuttings of up to about 10 m to 15 m.

Figure A2.2 shows the numerous rivers along the Natal coast and each river shown is sufficiently large to necessitate a bridge, or a large culvert. There are 40 rivers in the 300 km stretch, ie one bridge every 7 kms. At the time when many of the embankments described were being designed the conventional spelling of the Zulu river names was Umgeni, Umhlatuze etc. More recently these have been modified to Mgeni, Mhlatuze etc. The convention used herein has been to retain the form which was current at the design stage for those embankments described, but to conform to recent usage for other river names. The geology and the nature of the river courses is discussed in the following section, but it may be noted here that not all of the river crossings involve flood plains and hence approach embankments to bridges over alluvial deposits. Nevertheless approximately 30 such potential problem sites are encountered. A typical site would include a piled bridge and an embankment say 6 m high and 100 m long over an alluvial deposit comprising loose silty sands and very soft silty clays about 20 m deep.

There are two aspects to the geotechnical problems of embankments on soft soils which need to be addressed. The first is the nature and effect of the problem, ie stability and settlement which are discussed in A2.1, and the second is selecting the appropriate investigation and analyses which are discussed in A2.2.



TOPOGRAPHY AND MAIN ROADS OF NATAL

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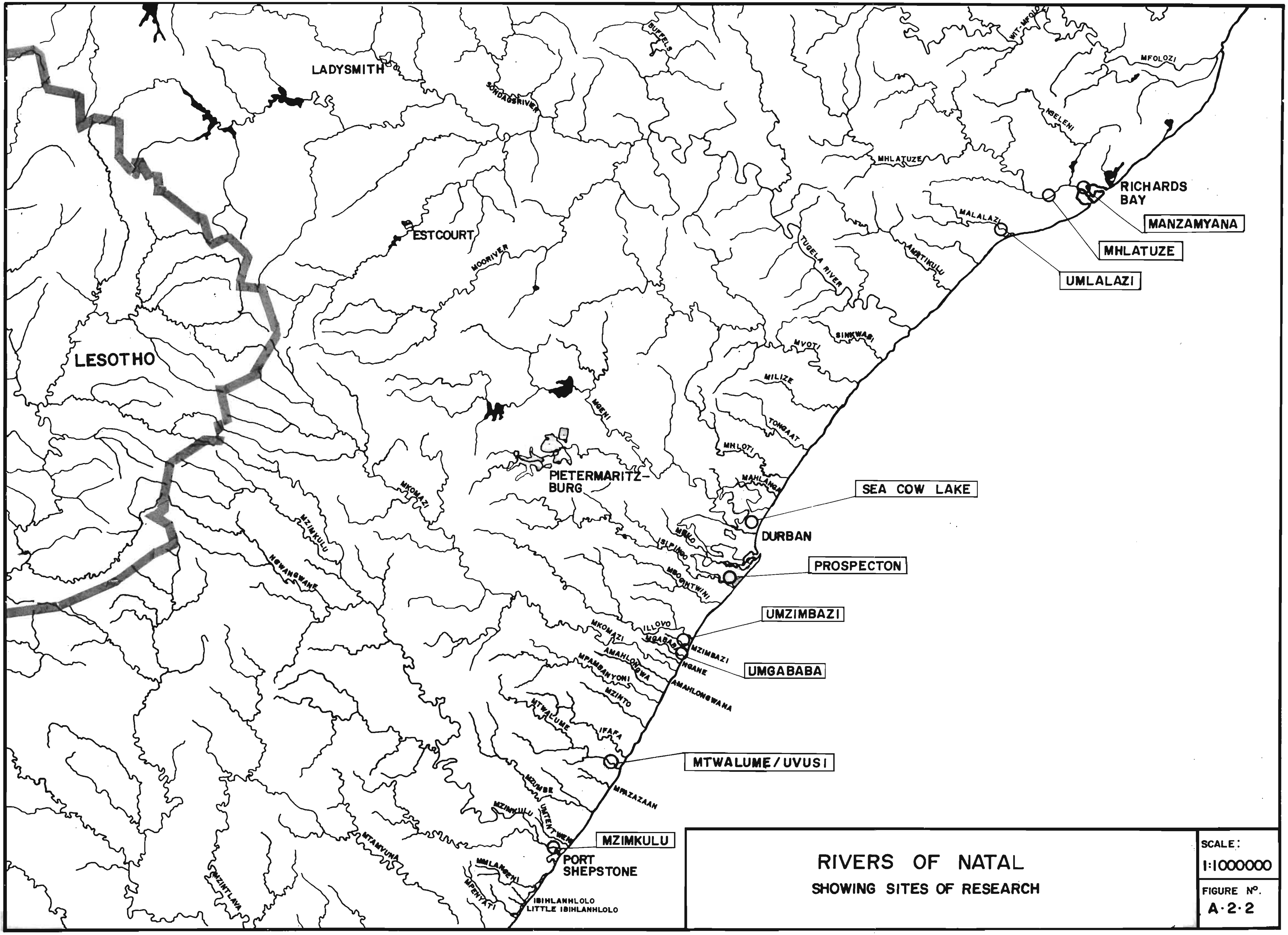
FIGURE A·2·1



RIVERS OF NATAL
SHOWING SITES OF RESEARCH

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FIGURE No. A·2·2

A2.1 Embankment Engineering Problems

It is convenient to separate the problems into the two aspects of stability and settlement and this division follows the traditional soil mechanics approach in which shear strength and compressibility were treated as essentially unrelated phenomena. The undrained shear strength of a clay was seen as a unique property of the soil and stability analyses postulated a failure state based on the limiting shear strength, viz a go or no go situation in which strain was not relevant. Similarly, compressibility was seen as concerned only with strains while the shear stresses were irrelevant.

These simplified models remain adequate for many cases despite the theoretical limitations, since they permit reasonable calculations of stability and settlement to be made. Where the simplification may be misleading is that it artificially separates soil behaviour into two distinct categories and inhibits a fuller understanding of its real nature. Nevertheless it is convenient to describe the resulting problems of stability and settlement separately.

A2.1.1 Stability

Two approaches may be adopted for the stability of embankments on alluvial deposits, viz total stress or effective stress analyses. The undrained shear strength, c_u, is the appropriate soil parameter for the former and the drained parameters, c' and ϕ' for the latter.

For the initial stability analysis to determine whether an embankment on soft alluvial soils will be safe a total stress analysis is appropriate. However it is often the construction stage which is most critical and for this an effective stress analysis using the drained shear strength parameters is necessary. In practice the soft clays present in the alluvial deposits along the South African coast are frequently so soft that stability is a major problem and detailed stability analyses are necessary. Reliable measurements of both the undrained and drained strength parameters are therefore required, and since the soft clays are often variable in properties and distribution geotechnical investigation and testing needs to be comprehensive to have a high probability of determining the most critical strengths.

The determination of these parameters is discussed in A2.2.

A2.1.2 Settlement

As already indicated settlements of embankments on recent alluvial deposits in Natal have been as much as 50% of final embankment height. Not only are such large settlements a major problem, but the fact that they may take many years after the opening of the road to stabilize considerably exacerbates the difficulties. The direct costs, and the indirect costs of disruption to traffic necessary to rehabilitate settlements of embankments are high. Where embankments are contiguous with fixed structures relatively small differential settlements create a very obvious problem. The less severe problem of settlement along an embankment may not be so dramatic, but in the long term the resulting differential settlements may also cause unacceptable deterioration in riding quality.

The parameters representing consolidation are the coefficients of volume compressibility and consolidation, m_v and c_v. The influence of stress history on the behaviour of soils, including recent alluvial deposits, is marked and measurement of the overconsolidation ratio, OCR, is essential. In addition to the settlement due to primary consolidation, the local yield, the immediate or pseudo-elastic settlement and the secondary compression need to be addressed.

The latter, secondary compression, is that settlement which continues after complete dissipation of excess pore pressure. Although it is convenient to envisage that the secondary compression does not begin until primary consolidation is complete, this is no more than a calculation convenience and there is in fact no unequivocal way to separate the processes. Secondary compression is generally viewed as being a viscous creep phenomenon unrelated to drainage although the creep properties of a soil will be related to its moisture content. On recent alluvial clays, which often have a high organic content, secondary compression may be significant proportion of the total settlement.

For embankments on recent alluvial deposits the primary consolidation aspect dominates the settlement estimation. The reasons for this are, somewhat pragmatically, that local yield and the immediate or elastic settlements will occur during construction and will therefore be compensated for as construction proceeds. There will be a cost for this not only for the height "lost" but also for the necessity to start the embankment wider than the design width. This of course applies equally to the future height lost by consolidation settlement. The secondary compression is in many cases assumed to be negligible compared with the primary consolidation, not only because it is difficult to establish the appropriate soil parameters by laboratory testing, but also because the design life of roads is generally about 20 years during which time relatively small settlements will be accommodated by ongoing maintenance. It is, however necessary to establish whether the secondary compression will in fact be small or very slow, because if not, decisions will have to take account of the ongoing settlement.

A2.2 Investigation and Analysis

The foregoing two subsections discussed both embankment stability and settlement problems. The latter is the primary subject of this work but the two behaviour aspects are so closely interrelated that it would be inappropriate to ignore the former, particularly since the investigation and testing have many common factors.

It should be noted that a formal data base for embankment performance in South Africa, does not exist, and one of the considerable difficulties in the recent research projects was assembling a sufficient number of reliable performance records of embankments to justify detailed back analysis.

A2.2.1 Stability

There are few recorded examples of stability failures of embankments on recent alluvial deposits in South Africa. Stability problems on recent deposits almost invariably occur during construction and as a consequence they can generally be controlled and rectified by the simple expedient of flattening the embankment slopes, constructing stabilising berms or by subsoil drainage systems. It is therefore probable that many incipient or partial stability failures are dealt with using pragmatic on site methods and, for various reasons, receive little exposure. This may be a disadvantage since in areas where stability failures are imminent the shear stresses are high relative to the shear strength, and consequently the shear strains will be high and higher than expected deformations will occur. However the cases that are recorded present no difficulties in analysis and tend to confirm that a simple investigation to determine the undrained shear strengths of the potentially affected clays and a routine circular slip, limit state, total stress analysis are sufficient for the majority of problems. The methods of determining the shear strength may either be in situ tests, such as vane shear or cone penetration testing, or undisturbed sampling followed by laboratory triaxial testing.

Effective stress triaxial testing and analyses will be required to model the situation where there is potential instability which will require construction control such as reduced rate of construction, or installation of subsoil drainage techniques to reduce excess pore pressures.

In order to provide a decision making process on whether or not embankments required sophisticated investigation and analysis, from both the stability and settlement aspects, the author wrote a publication while at the National Institute for Transport and Road Research, (1987) in the Technical Recommendations for Highways (TRH) series on "The Design of Road Embankments" - TRH 10. This was a companion volume to an earlier publication "The Construction of Road Embankments" - TRH 9 - which the author wrote in collaboration with a small committee, (1982). These two publications are referenced as National Institute for Transport and Road Research publications in the Author's and general references.

A2.2.2 Settlement

All embankments on alluvial deposits bear testimony to the settlement problem; every one has settled and where fixed structures traverse the embankments the all too familiar bump is evident. No figures are available for the cost of remedial works, and of the disruption caused, but it is probable that many millions of rands are spent in rectifying the problem. The cost, however, of very detailed investigation and subsequent prevention measures may also be high and the requirement follows for cost effective investigation so that reliable estimates of settlement and times for settlement can be made at all potentially major problem sites so that appropriate decisions for that site can be made.

The geotechnical investigation techniques for settlement estimations of embankments on recent alluvial deposits are similar to those for stability analyses, viz in situ tests, such as penetration testing, combined with undisturbed sampling and laboratory testing. The latter would generally be conventional oedometer tests although in some cases more sophisticated consolidation tests (Rowe Cell) in which pore pressures can be measured, may be employed.

Investigation of the compressibility of the sands is almost invariably confined to in-situ penetration testing since the difficulties of undisturbed sampling preclude reliable laboratory testing. The subsequent analysis relies almost entirely on a simplistic pseudo elastic approach in which moduli are derived for the sandy materials using empirical correlations between in situ test results and the moduli. Many years of worldwide experience have provided a range of correlation factors between penetration tests and moduli for sands and in broad terms it may be said that there is considerable confidence in this approach, although the reliability of the settlement prediction is probably much poorer than may be generally thought.

More fundamentally appropriate in situ tests, such as self boring pressuremeters and dilatometers appear to offer a superior approach. Both measure stress-strain characteristics directly and should not therefore require empirical correlations. The former, however, is more complex and therefore relatively expensive and the latter, although similar in many respects to cone penetration testing, has only recently become available in South Africa.

Settlements due to consolidation of clay strata are more problematic than settlements in sands, not only because they are relatively larger, but also because of the lengthy time component. Generally the investigation, testing and settlement analysis of alluvial clays should be straightforward. The techniques exist to sample even very soft clays; laboratory consolidometer testing is well established and these are backed by satisfactory analytical methods. In practice, however, the difficulties are greater than is commonly supposed in all three aspects of sampling, testing and analysis. There can be no doubt that at present this classic approach must remain as the benchmark, but it is important to appreciate that despite this, the reliability of the settlement and settlement rate estimates so obtained is often poor even when the best techniques are used. Sophisticated sampling and testing is expensive and although such expense is more than justified where major problems may exist, the reality is that the costs have a major bearing on the nature of any investigation. In variable alluvial deposits the problem is compounded by the necessity for ensuring that the samples tested are fully representative of the different materials, and for this a large amount of investigation may be required and the cost may be seen as prohibitive.

As already indicated analyses tend to concentrate on the primary consolidation aspect, since this is often considered of greater importance than local yield, immediate settlement or secondary compression. However this emphasis may often be misleading since secondary compressions may be very significant for a road embankment on soft clays, and also adequate understanding and back analysis of settlements must include local yield and immediate and secondary compressions. The methods of analysis are well established and the consensus is that they are adequate in themselves but the major difficulty, particularly in soft materials which frequently traverse both the normally and lightly overconsolidated states, is establishing reliable soil parameters.

The title of this work is, "The development of sounding equipment for the assessment of the time-settlement characteristics of recent alluvial deposits when subjected to embankment loads".

Two points in this should be clarified.

The first is that sounding is now more correctly called cone penetration testing.

The second is that time-settlement refers both to settlement and to time taken for that settlement ie compressibility and consolidation characteristics.

A3 GEOLOGY AND CLIMATE

The geology of Natal and the Natal coast in particular has been extensively described by Brink (1985), Francis (1983), King (1962, 1972), King and Maud (1964), Maud (1968), Moon and Dardis (1988), and Orme (1974, 1976).

A3.1 Geological History

The following relatively detailed account of the geology serves to point out the complexity of the estuarine deposits and hence the major difficulty in carrying out effective geotechnical investigation.

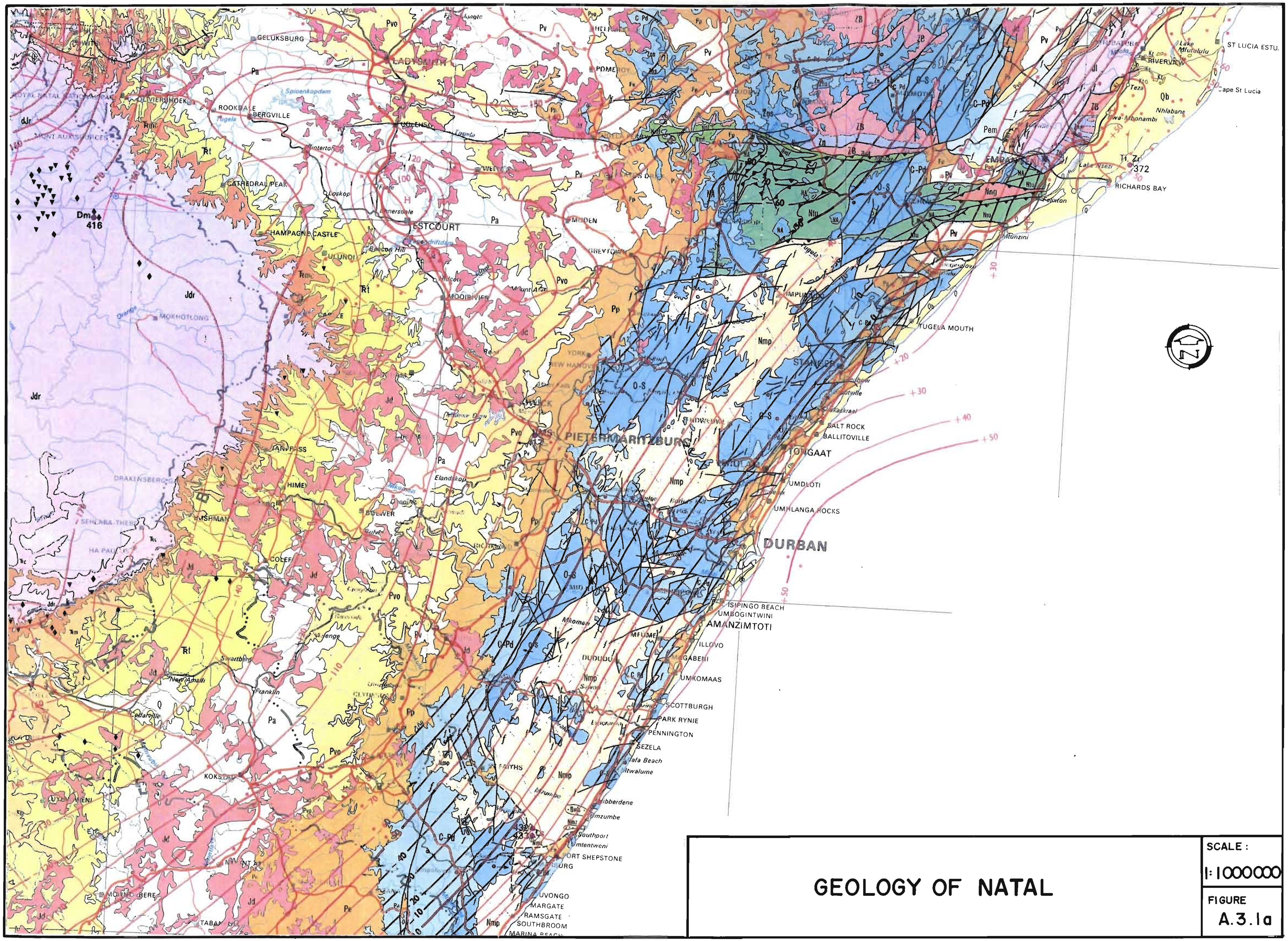
The geological history of the area has been well defined and is briefly described below, but there has been considerable discussion regarding the geomorphological processes relating to Southern Africa much of which has centred on the Natal hinterland and coastal zones - Figure A3.1.a shows the geology of Natal and Figures A3.1.b and A3.1.c show the geological legends and the lithology. Figure A3.2 shows the geology of the Durban area.

The oldest rocks are the granites of the late Proterozoic. They occur in a band approximately parallel to the coast running from Margate in the south to Empangeni in the North, a distance of about 300 km. The band is intermittent but is generally from 10 km to 30 km wide and is either at or close to the coast in the part south of Durban and say 25 km inland to the north of Durban.

The granites vary from the highly weathered deeply incised materials which form the scenically impressive Valley of a Thousand Hills between Durban and Pietermaritzburg, to the hard rock outcrops of the mouths at the Ibilanhlolo Rivers at Ramsgate on the extreme south coast.

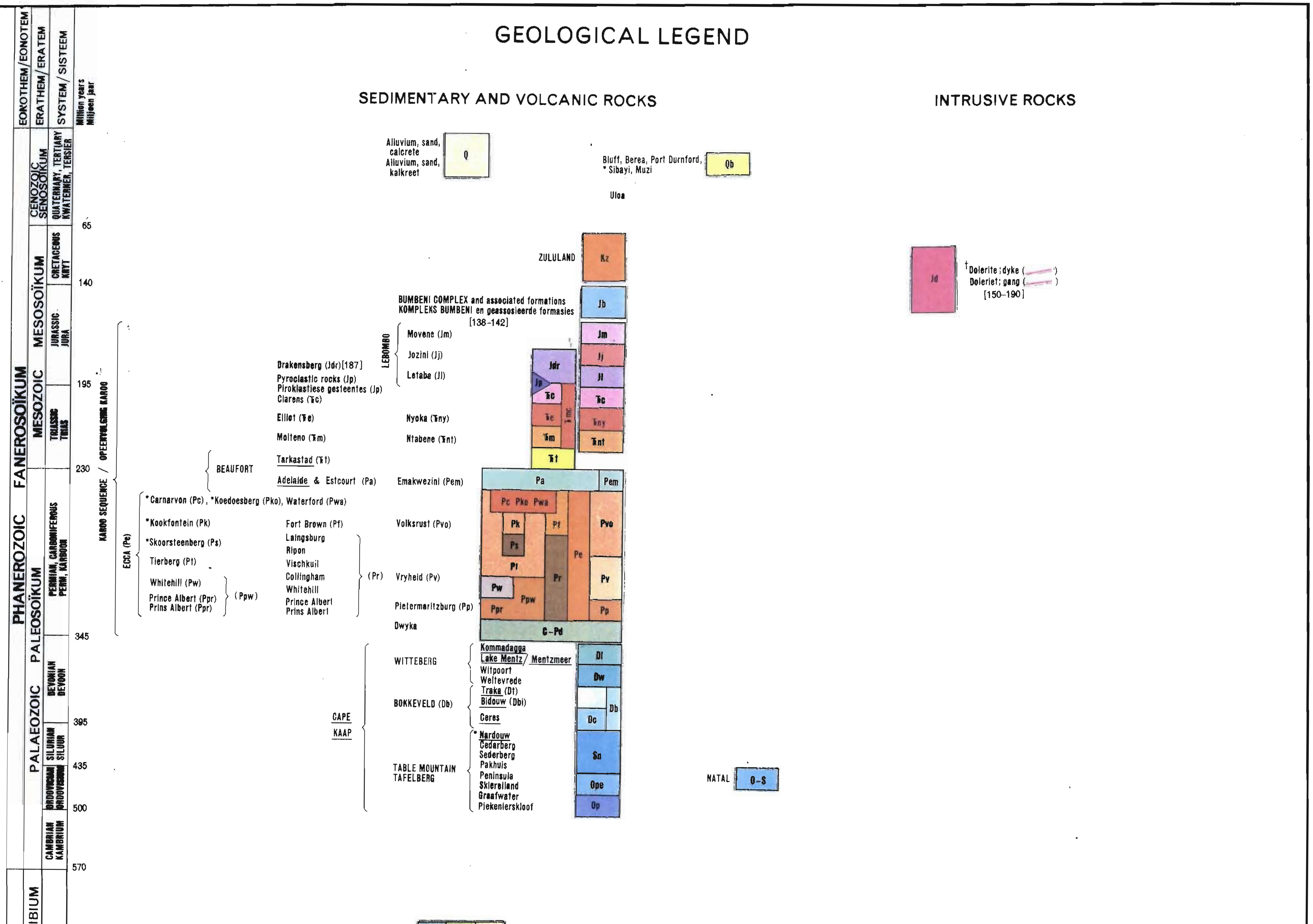
The granite intrusion was followed by the sedimentation of the Gondwana Era. The quartzitic sandstones of the Natal Group of the Cape Supergroup were deposited in this period. They form a broad band approximately 50 kms wide following the coastline and surround the granites, except at the south where the granite outcrops at the coast.

Although some of the more quartzitic sandstones have resisted weathering and have resulted in cliffs and sharply defined valleys, eg in the Kloof area, generally the sandstones are deeply weathered.



GEOLOGY OF NATAL

GEOLOGICAL LEGEND



GEOLOGICAL LEGEND

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FIGURE A.3.1b

Approximately one half of the Natal rivers rise in the granite and sandstone belt a distance of about 50 km from the sea, and these rivers traverse the granite and sandstone for most of their lengths before crossing a narrow coastal strip of more recent rock types, viz the Dwyka Formation and Karoo Sequence rocks.

The former results from the glaciation period. The tillites occur in a narrow fringe surrounding the Natal Group sandstones. The fringe is generally no more than say 5 km wide other than in the south where the main road between Port Shepstone and Harding traverses about 25 km of tillite. Generally the tillites are deeply weathered soft rocks with a reputation for being poor quality construction materials; however there are some tillite quarries from which hard blue grey aggregate is obtained suitable for road bases and surfacing and for concrete aggregates. The tillites because of their limited extent do not strongly influence either the river directions or their deposits.

Following the glaciation the Karoo basin was subjected to the major depositional period, the Karoo Sequence, which covered most of South Africa. In Natal this sequence is dominated by the Ecca Group shales. These occur both along the coast in a narrow fringe adjacent to the Dwyka tillites and primarily in a band about 35 kms wide from the southern Natal boundary up to Pietermaritzburg from where it widens to cover most of northern Natal.

Beaufort Group shales and fine grained sandstones occur in southern and mid Natal inland of the Ecca Group shales. Many of the Natal rivers rise in this area, in the narrow overlying fringe of the Molteno, Elliot and Clarens Formations or in the lavas of the Drakensberg Formation.

The Karoo sediments are very extensively intruded by dolerite dykes and sills which in much of the area, because of their greater resistance to weathering than the shales, dominate the landscape and control the river courses.

The Drakensberg mark the easterly edge of the Great Escarpment which divides the interior plateau from the coastal region. The escarpment marks the edge of the continental upwarping whereas the east is a downwarped zone. Erosion of the latter towards the coast defined the escarpment.

Subsequent periods of tectonic uplift and the ongoing erosion processes resulted in the removal of much of the Karoo rocks and in Natal exposed the underlying Dwyka, Cape Group sandstone and the granites.

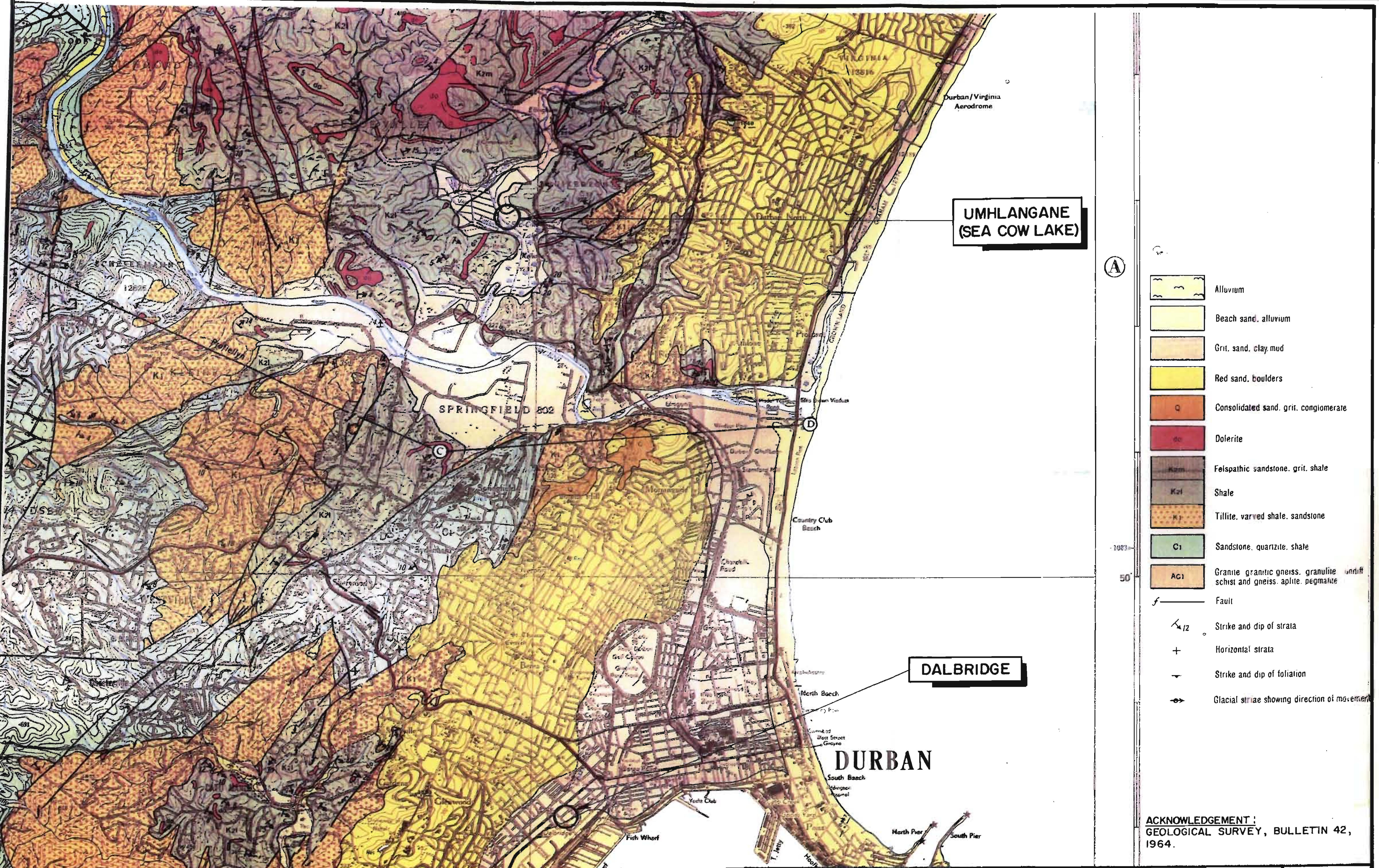
Of more recent and direct influence on the rivers of Natal have been the global climate changes and eustatic sea level changes.

The Natal rivers form part of the eastward drainage system from the escarpment to the coast. Gradients are steep in the upper reaches resulting in an ongoing degradational process. In the lower reaches, however, because of changes in eustatic sea levels or to tectonism the gradients are no longer steep and deposition occurs, hence the formation of the estuarine materials.

A further factor in the fluvial deposit process is the occurrence of unusual weather conditions. In Natal in recent times the most marked of these has been Cyclone Demoina resulting in the violent floods of January 1984 during which both extensive shallow deposition took place in some flat coastal lands and at the same time the Mfolozi River scoured the St Lucia estuary to a depth of 6 m. A number of the Natal rivers have shown recent scouring and redepositing and cone penetration testing has proved to be a valuable tool for measuring the thicknesses of very recent river deposits and hence for estimating future potential scour depths.

A number of the Natal rivers end in lagoons, often separated from the beaches by a ridge of sand dunes. Others have estuaries, whilst a few discharge directly into the sea with no intervening zones of more marked deposition. The majority of rivers traverse flatter coastal zones which because of their topography have provided the routes for the transportation systems, viz road and rail. It is in these areas that the depositional history has been most complex.

The coastline was essentially determined by the end of the Cretaceous (70 Ma). Since then the coastline has been subject to smaller changes induced by tectonic movements together with eustatic sea level changes. Several cycles of transgression and regression occurred during the Tertiary and Pleistocene periods and beaches have been defined at various levels along the Natal coast eg 30 m and 60 m. Along much of the coastline Pleistocene dune ridges are prominent and previous beach deposits are found in these.



GEOLOGY OF DURBAN

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FIGURE A·3·2

There is also a system of offshore submerged dunes formed when the sea level was 68 m below the present level. During the late Quaternary the sea level dropped to about -130 m but after 17 Ka the Flandrian transgression took place, due to deglaciation, resulting in a build up to approximately present day level and to present day configuration of the shore line, although this was not a uniform progression.

The detail of the shore line is determined not only by the major geological historical events but also by the marine processes of erosion and deposition. Generally these processes have not directly influenced the inland deposits except where barriers to the lagoons are continuously being formed and destroyed leading to changes in level in the lagoons.

A3.2 Estuarine Deposits

The sediments in the estuaries may be derived both from inland and also from the sea during a transgression period or through longshore drift and tidal action. Many of the estuarine deposits contain fragments of shells and both along the south coast and in the Durban Bay and Richards Bay silty clays there are marked zones of shells. These clays however are not marine clays in the usual geotechnical sense of being deposited in a marine environment.

The depths to bed rock of the Natal rivers vary considerably and are proportional to the size of the rivers. For example the Tugela, the largest river, has a channel depth of over 55 m, as have the Mfolozi, Mkomazi, Mhlatuze and the Mgeni, which are the largest of the rivers.

Generally the medium sized rivers, for example the Tongaat, Mhloti and Mtwalume have bed rock channels of about 30 m below mean sea level, and the smaller rivers have channels 20 m deep and less.

The present depths at the river mouths, or a few kilometres upstream where most of the drilling has been carried out for road or rail bridges, indicates that the sea levels were originally much lower than now, since the river channels previously continued to the old mouths across the continental shelf. The sea level must then have been lowered to -130 m before being restored by the Flandrian transgression.

In this period fluvial, lagoonal and marine deposition took place progressively from inland. The depositional pattern however is not simple due to the relatively minor regressions within the main transgression and also due to periodic flooding. The marine deposits are sands generally containing shells which were carried into the estuaries by wave and tidal action during the transgression. These marine deposits are found towards the sea end of the estuaries.

A further category of marine deposits is the sand bars and spits which are a feature of many of the Natal rivers. These marine sands are, however, transported upstream only a very short distance and are also subject to removal during floods.

The estuarine deposits which give rise to the majority of the engineering problems are those deposited under lagoonal conditions. In broad terms the finer silts and clays were deposited in the deeper water and the fine sands and silts in the transition zone between these and the coarser fluvial sands. During the period of rising sea levels the river borne deposits built up creating barriers between the main river course and the tributaries. This resulted in the development of small lakes or backwaters eg Sea Cow Lake on the Umhlangane tributary of the Mgeni, in which, because of the still water conditions and the sediments of the shorter tributaries being predominantly derived from shales, the deposits are often amongst the softest most organic clays found in Natal.

Subsequent flooding and scouring has removed many of the earlier Flandrian sediments and these have been replaced with more recent sediments so that the stratigraphic succession may present a very mixed sequence both vertically and transversely across an estuary. The materials can vary from considerable thicknesses of almost uniform organic black silty clays, as at the Umhlangane, through highly variable deposits of silty sands and clays to predominantly deep fine to medium sands as at the Tugela. Care however must be taken in characterising any of the rivers in this way since a short distance upstream or downstream the deposits may be quite different.

A3.3 Climate

The climate of Natal is subtropical and the predominantly summer rainfall largely thunderstorm variety gives a mean annual precipitation of over 900 mm. This includes the upper Tugela basin and the central to northern Zululand area where the rainfall is lower. In the area of Natal considered in this thesis, ie from the southern boundary at the Mtumvuna river to Richards Bay 300 kms to the north, the mean rainfall is considerably higher. It comprises high rainfall areas, over 1250 mm along the Drakensberg, the source of the major rivers, an intervening zone of lower rainfall, 750 - 1000 mm, and a coastal strip also of high rainfall of over 1250 mm. This area of Natal is characterised by having perennial streamflow up to about the Tugela, whereas further north the lower rainfall inland and the flat wide sandy coastal plain lead to high infiltration.

A feature of the Natal rainfall is that frequent severe flooding occurs and considerable damage has been caused both to the transportation routes and to property. Catastrophic flooding occurred in 1959, and with Cyclone Demoina in 1984. In the 1959 floods the railway bridges at the Mzimkulu (Port Shepstone), Illovo, Mkomazi, Mpambinyoni, Mtwalume, Mzumbe and Mbizane were all partially or completely destroyed. Severe flooding also occurred in 1987 during which many of these major bridges and their approach embankments were destroyed causing not only direct damage but also severe disruption to the local economy. In some cases bridge foundations had been piled into dense sand layers, (not to bedrock which was too deep for the available piling equipment) and the flood scour depths of up to 10 m had removed support for the piers causing collapse. In other cases the estuary crossings comprised a long approach embankment and a relatively short bridge. Typically the normal river courses, and hence bridges, are at the southern side of the estuary and meander within the estuary. During extreme flooding the embankment may then bear the brunt of the force of the straightened river causing a breach in the northern approach fill. This was perhaps fortuitous in many cases, since it is easier to replace part of an embankment than to rebuild a bridge. These flood events graphically illustrate how readily the sediments in the estuaries can be modified even within recent memory. In the smaller rivers severe floods may effectively totally remove the previous sediments and subsequently replace them with new materials. In the larger rivers the pattern of the sediments may be considerably altered.

The road development along the Natal coastal zone over the past two decades, together with other industrial and infrastructure development, has necessarily led to civil engineering works over recent geological deposits which were previously avoided because of the difficult engineering conditions. Due to the highly erratic and soft nature of these materials geotechnical investigations require to be both comprehensive to cover the variability, and of high quality to sample and test the soft clays. It is contended that sophisticated cone penetration testing is a most valuable if not essential technique, in conjunction with boreholes and sampling, to provide high quality cost effective investigations.

B INTERNATIONAL REVIEW OF CONE PENETRATION TESTING

B1 INTRODUCTION

In order to place in its proper perspective the work carried out and reported herein by the author on cone penetration testing it is first necessary to describe the international development and application of this form of testing. The author's contributions recognized in the international literature are also referred to in this part. The review comprises three sections. The first, which includes B2 and B3, is a historical review of the development of mechanical and piezometer cone penetration testing.

The second, B4, is a review of current methods of the interpretation of cone penetration test results.

And the third, B5, is a review of current practice regarding the geotechnical design of embankments on alluvial deposits.

The reporting on the developments of all forms of cone penetration testing has been dominated in recent years by three specialist international symposia, viz :

1974	1st European Symposium on Penetration Testing, ESOPT I, Stockholm.
1982	2nd European Symposium on Penetration Testing, ESOPT II, Amsterdam.
1988	1st International Symposium on Penetration Testing, ISOPT I, Orlando.

These have been closely followed in significance by the following :

1969	Conference on In situ Investigations in Soils and Rocks, London.
1975	ASCE Speciality Conference on In situ Measurement of Soil Properties, Raleigh.
1981	ASCE Symposium on Cone Penetration Testing and Experience, St Louis.
1983	ASCE Speciality Conference, Geotechnical Practice in Offshore Engineering, Austin.
1983	International Symposium In situ Testing, Paris.
1986	ASCE Speciality Conference, In situ 86, Blacksburg.

The regular International Conferences on Soil Mechanics and Foundation Engineering have also been the forum for a number of publications on cone penetration testing, from Fifth at Paris, 1961; through Sixth, Montreal 1965; Seventh, Mexico, 1969; Eighth, Moscow, 1973; Ninth Tokyo, 1977; Tenth, Stockholm, 1981; Eleventh, San Francisco, 1985 and Twelfth, Rio de Janiero, 1989.

The regional conferences on Soil Mechanics and Foundation Engineering, particularly those for the European Region, have also been the source of a number of valuable papers on cone penetration testing.

Many of these conferences and in particular the two ESOPTs and ISOPT have given states of the art of cone penetration testing and these, together with Sanglerat's book, The Penetrometer and Soil Exploration first published in 1972, more than adequately describe the position up to the early 1970's. No attempt is made herein to repeat these historical reviews up to the early 1970's, except in a brief summary in the following section, B2. A more detailed review of the subsequent development of piezometer cone penetration testing is given in B3.

B2 REVIEW OF MECHANICAL CONE PENETRATION TEST, 1930 - 1970

Cone penetration testing in its present form began in the Netherlands in the 1930's. The original purpose was to determine the depth and density of sand strata in the multilayered sands silts and clays so that piles could be designed.

Buisman (1935) and Barentsen (1936) are credited with the earliest publications on what was then called sounding. By the late 1930's the Soil Mechanics Laboratory in Delft had developed the equipment and interpretation so that it was in regular use for site investigations - Figure B2.1. At that stage the cone penetrometer was seen essentially as a miniature pile test to failure and the consequent pile design was simply scaling up the load by the ratio of the cone diameter to the proposed pile diameter and applying a factor of safety.

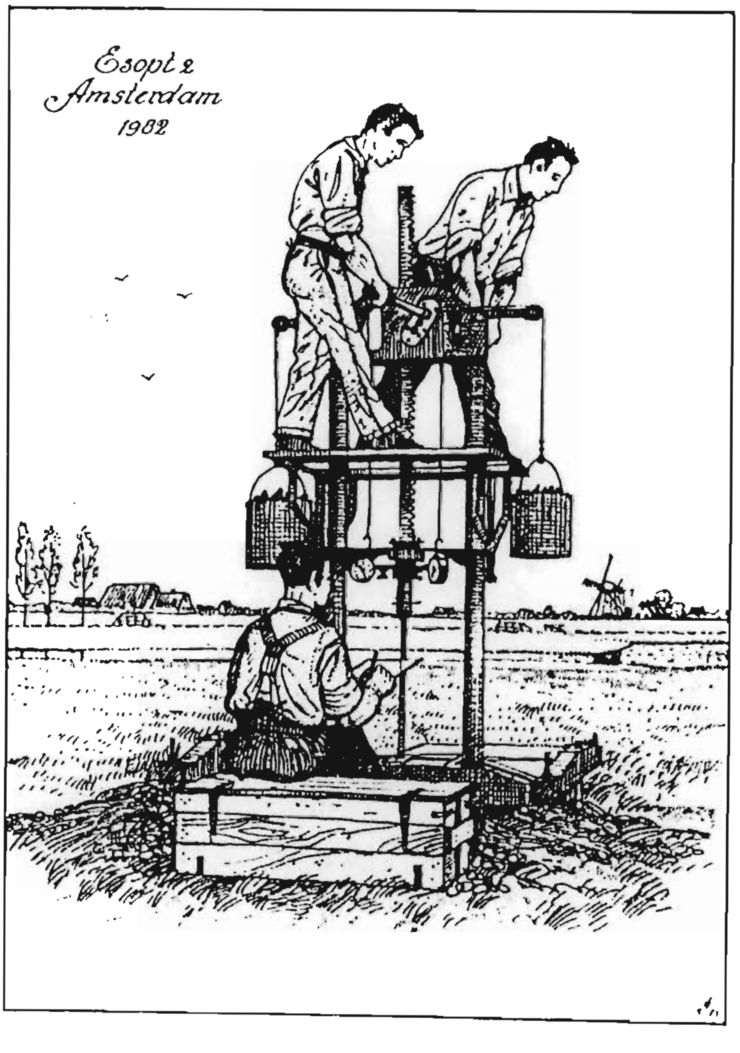
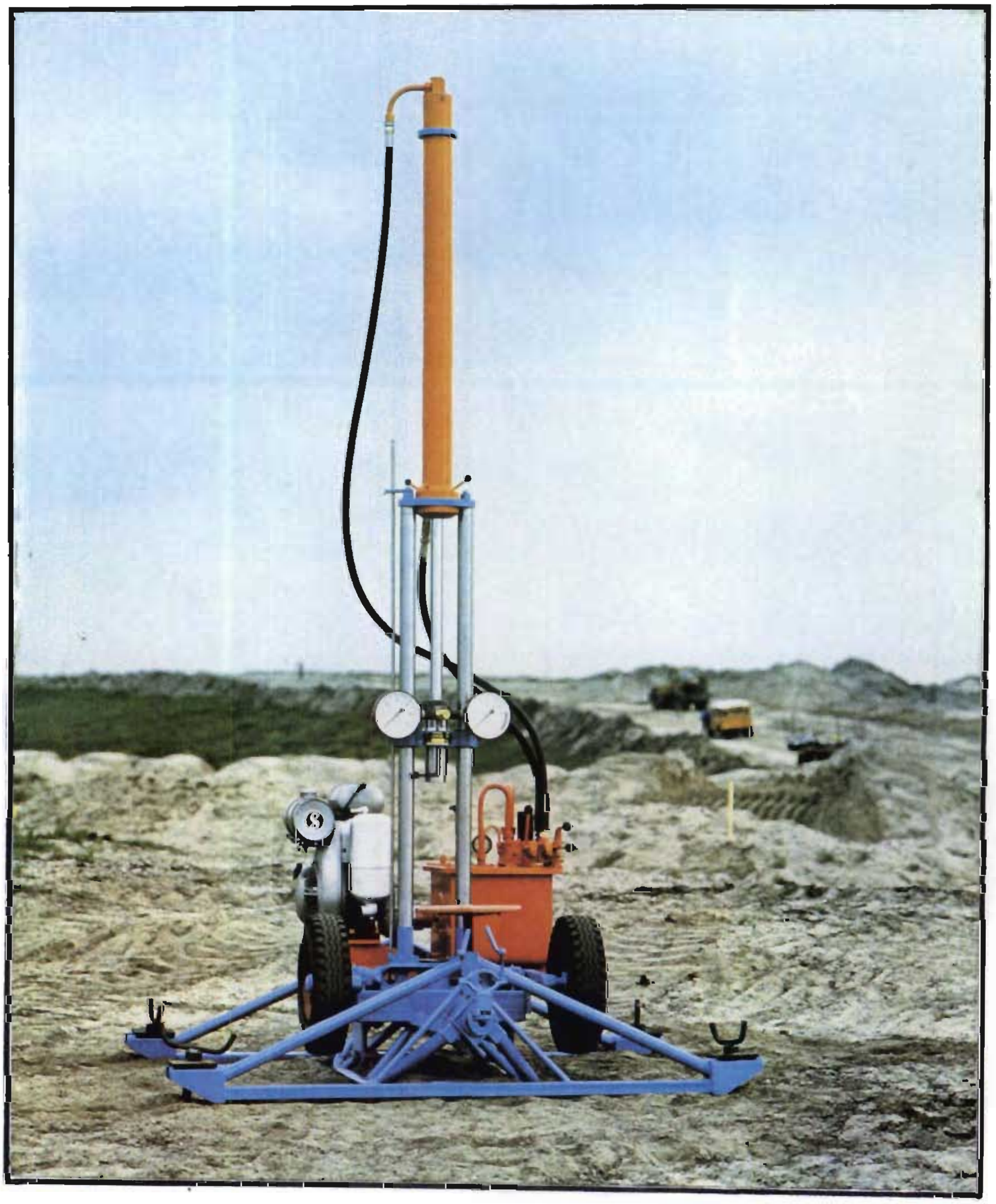


Figure B2.1 : Cone penetration test rig - 1930



100 kN CONE PENETRATION TEST RIG - 1960

Fig. B·2·2

After a period of no development during the 1939 -1945 war, Delft, in conjunction with Goudsche Machinefabriek of Gouda in 1946 produced hand operated sounding machines of 25 kN capacity using 1000 mm^2, 60^o mantle cone (35,7 mm diameter) operated through 1 m long, 35,7 mm diameter outer rods, or casings, and solid inner rods. The load measuring system was a hydraulic load cell with the pressures registered on high and low pressure gauges. By 1948 a similar hand operated 100 kN machine was produced which was superseded in 1959 by the petrol engine driven hydraulically operated 100 kN rated machine which became practically a world standard rig and which continues in operation today with few changes - Figure B2.2. A diagram of the rig with rods and cone is shown in Figure B2.3. Subsequently higher rated machines were produced, 200 kN and many have been mounted in special vehicles often designed so that the reaction necessary to resist uplift caused by penetration is provided by the vehicle mass rather than by anchors screwed into the ground. The rods and cones changed little from the Barentsen mantle cone until the advent of the Begemann friction sleeve in 1953. This did not however become widely used until his 1965 publication describing the use of the friction sleeve as an aid to determine the soil profile. The mantle and friction sleeve cones are shown in Figure B2.4. These continue in use today and the standards for the equipment and its operation are described in documentation on Cone Penetration Test (CPT), International Reference Test Procedure produced by the Working Party, Technical Committee on Penetration Testing of the International Society of Soil Mechanics and Foundation Engineering (1988) of which the author is a member.

In the 1970's electric cones were being produced (de Ruiter, 1971) of the same external dimensions - Figure B2.5 - as the previous mechanical cones but with some differences, viz no requirement for a mantle, which meant that the pressures measured by the two systems are at variance by about 10 - 20%. The electric cones were developed primarily because of the impetus given to cone penetration testing by the off shore oil industry since these cones could be operated underwater either from remote controlled submersible rigs or from diving bells.

It was estimated from data collected by the Technical Committee on Penetration Testing that by 1983, of the 150 000 m of CPT carried out annually, about 120 000 m used the mechanical cone, about 10 000 m used the electric cone and 20 000 m used the piezometer cone. Despite the advantages of the more sophisticated equipment the more rugged and economical mechanical cone continues to dominate the overall usage.

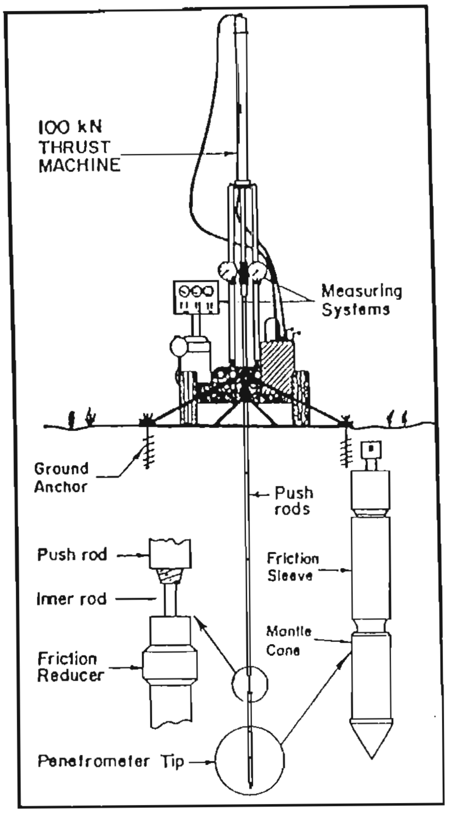


Figure B2.3 : CPT rig with rods and penetrometer (Jones, 1992)

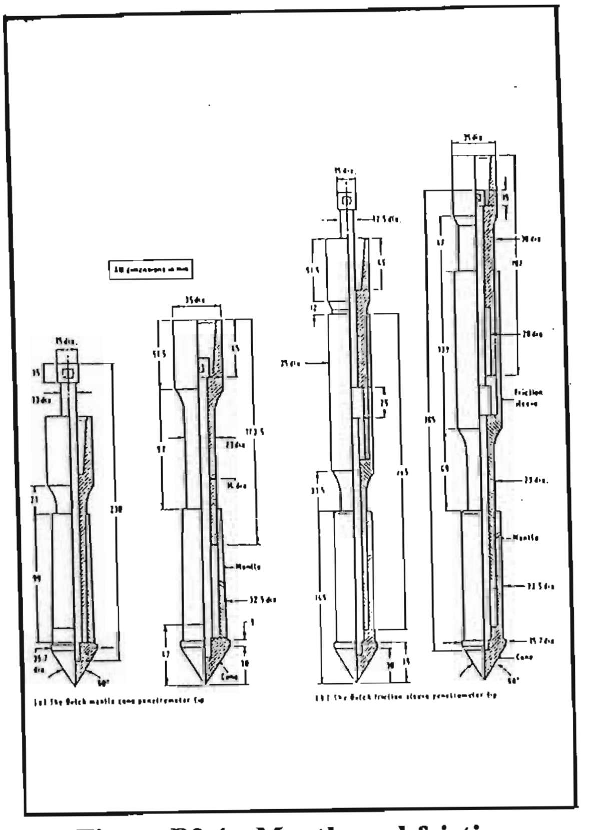


Figure B2.4 : Mantle and friction sleeve cones (Meigh, 1987)

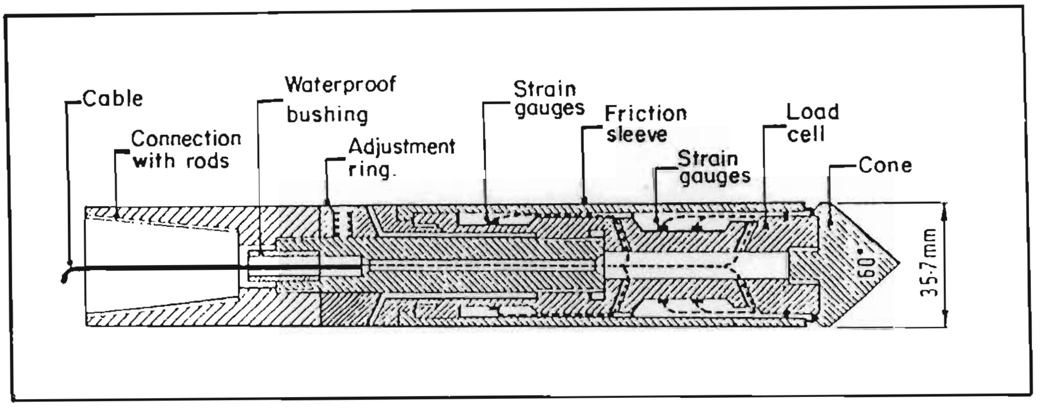


Figure B2.5 : Electric friction sleeve cone (Meigh, 1987)

This remains the overall position in South Africa except for investigations on soft alluvial deposits and in tailings dams where the piezometer cone is used.

Because of the importance the piezometer cone plays in such investigations and because of the author's part in piezometer cone development, its international history is reviewed in some detail in the following section, B3.

B3 REVIEW OF PIEZOMETER CONE PENETRATION TESTING

The earliest significant reference to the influence of the pore pressures generated during cone penetration on the penetration resistance was by Schmertmann (1974) at ESOPT I.

In a discussion on the effects of pore pressures generated during penetration of the quasi static cone bearing pressure, q_c, Schmertmann described the effects from a theoretical basis, noting in particular that they could be either positive or negative, and if ignored, could lead to either over conservative or under conservative interpretations of shear strength from cone pressures. No cone penetration test equipment was at that stage available to measure simultaneously cone and pore pressures, but reports had been published on the influence of pore pressures on the penetration resistance of instrumented piles and on the installation of conventional piezometers. A notable example of the latter was given by Penman (1961) in which the response times of various piezometers were discussed, Figures B3.1 and B3.2. Four observations from the paper are pertinent.

i) The general layout of both the strain gauge and vibrating wire piezometers are very similar to current piezocones.
ii) The response times of both the electrical piezometers are much shorter than the Bourdon gauge type and that of the vibrating wire type is very short and close to the calculated value using Hvorslev (1951) method.
iii) Observations were made of the pore pressures generated during installation of the piezometers by pushing them into position, and these were noted to be about twice the shear strength of the clay for the strain gauge piezometer with the relatively small filter size, and about the same value as the clay for the vibrating wire piezometer with the relatively large filter size.
iv) For all piezometer testing considerable emphasis was given to de-airing the systems.

This paper by Penman appears to have been referenced only once in the cone penetration testing literature, viz by Peignaud, (1979) yet it touched on many of the aspects of piezocone development which became more widely appreciated only twenty years later.

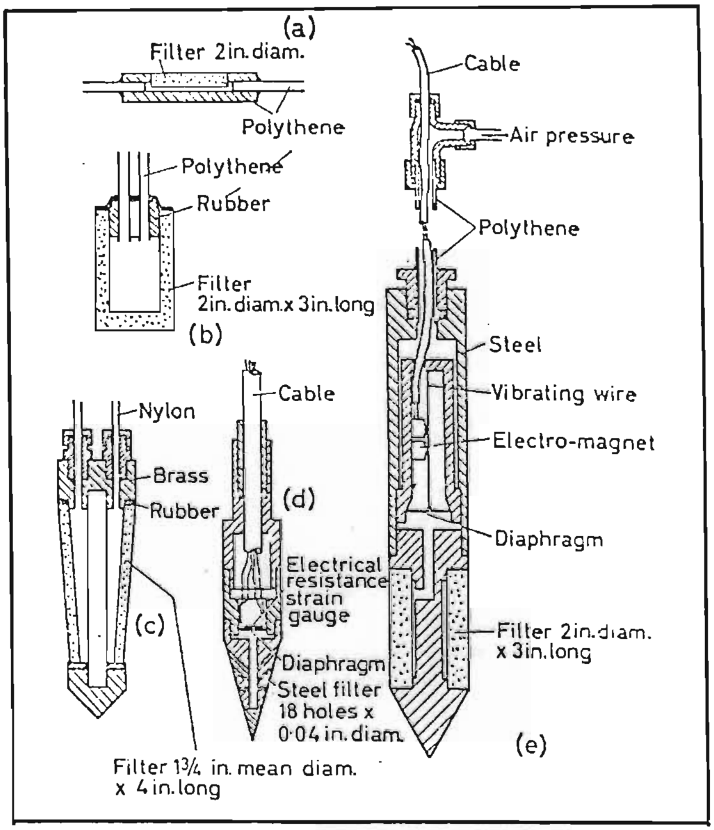


Figure B3.1 : Piezometers tested (Penman, 1961)

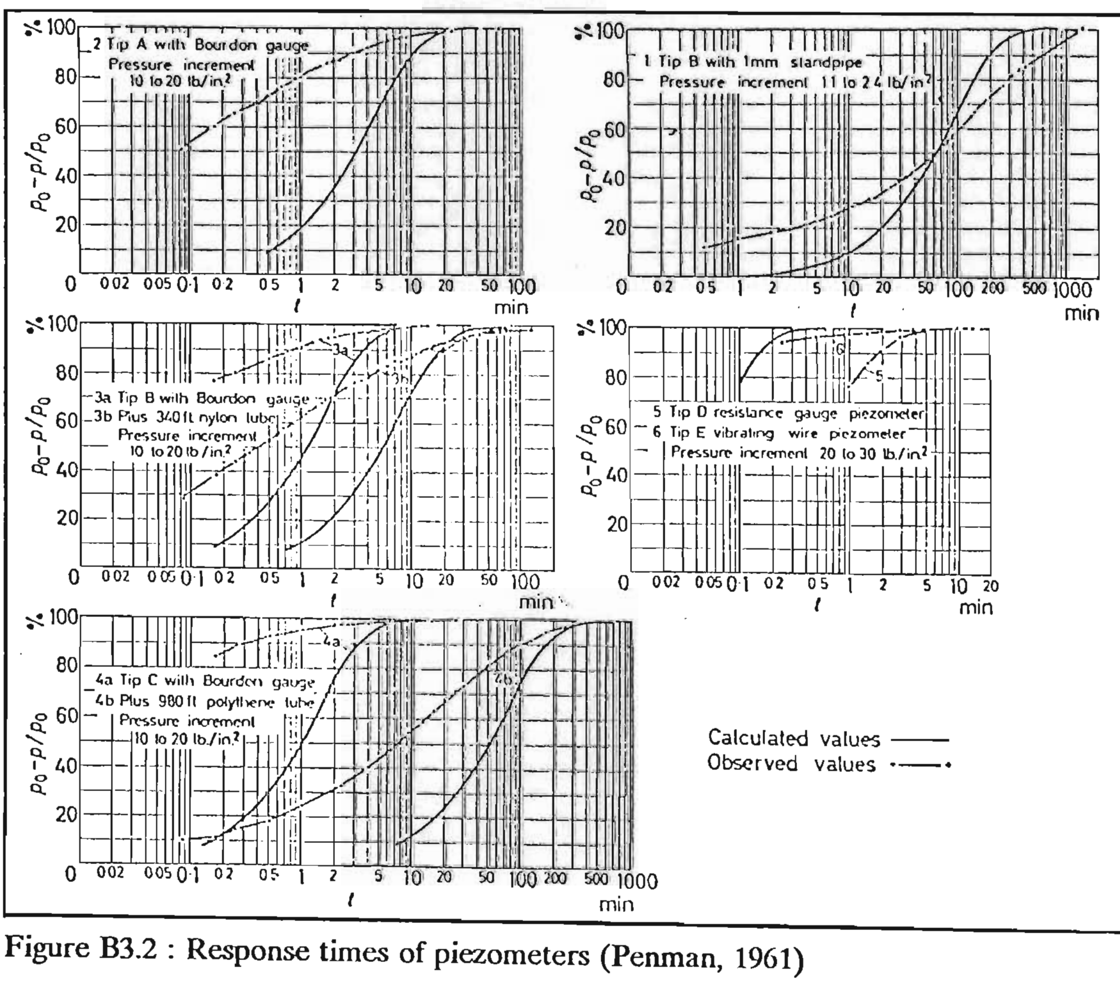


Figure B3.2 : Response times of piezometers (Penman, 1961)

The only other paper in ESOPT I 1974, which made any other significant reference to the influence of pore pressures was that by Janbu and Senneset which discussed effective stress interpretation of in situ static penetration tests. They and Schmertmann were primarily concerned with the proper interpretation of cone penetration testing to obtain shear strength parameters. The other papers at ESOPT I dealing with what was then called Dutch sounding, static or quasi-static penetration testing were almost exclusively concerned with two aspects : the growing use of electrical cones, which had been given a great impetus by the offshore oil industry, and the development of correlations of cone and friction sleeve values with shear strength, compressibility and soil type.

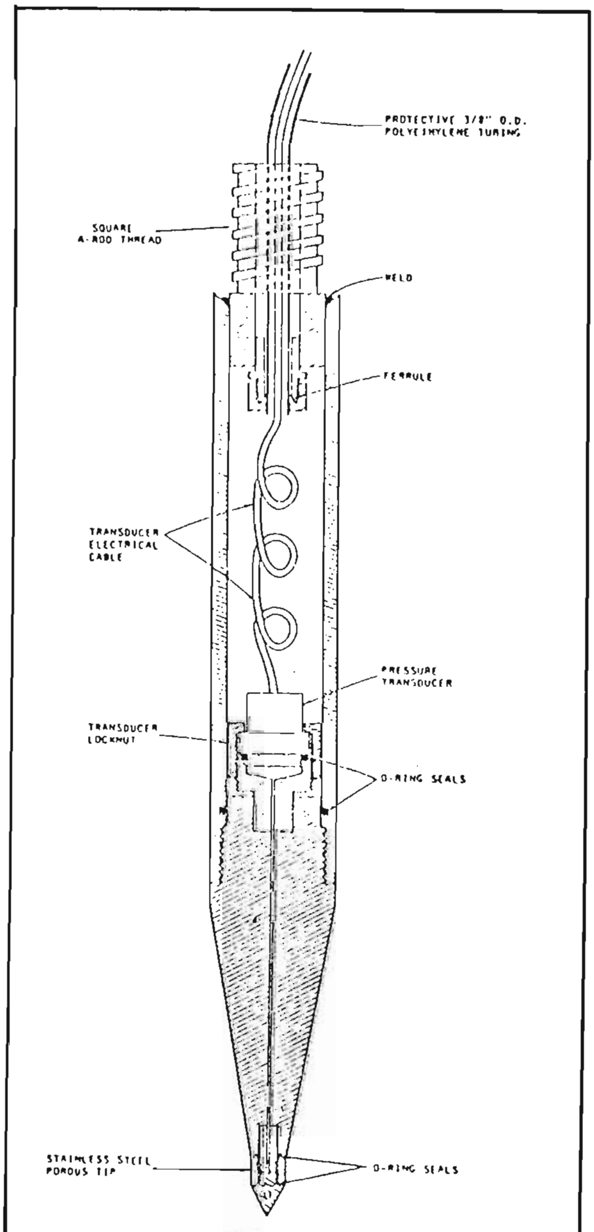


Figure B3.3 : Schematic of the piezometer probe (Wissa et al, 1975)

In 1975, a number of publications appeared which described the measurement of pore pressures during cone penetration. The Raleigh ASCE Speciality Conference on the In-situ Measurement of Soil Properties, 1975, contained three papers referring to pore pressure sounding instruments or piezometer probes, viz by Torstensson by Wissa Martin and Garlanger, and by Massarsch, Broms and Sundquist. All three papers described piezometer probes in which only pore pressures were measured during penetration, ie no facility existed for measuring cone pressures. The Torstensson and Wissa probes were very similar to one another and also to that described by Penman. Figure B3.3 shows the Wissa probe which has a small cone angle, 20°, compared to the 60° conventional cone penetrometers, and has a small filter element at the tip. Massarch et al described a probe which had filter elements at a number of positions and demonstrated the significance of the position on the pore pressures recorded. All systems gave electrical outputs to chart recorder systems at the surface via cables threaded through the rods. Both Torstensson and Wissa et al noted that the excess pore pressures generated during probing could be measured, as could the rate of dissipation of pore pressures on stopping penetration. The observations made by Wissa et al were particularly interesting and some are shown in Figure B3.4. They commented "During penetration of the probe the soil surrounding the probe fails in undrained shear. As a result, dense or stiff soils generate negative excess pore pressures when penetrated by the cone, whereas loose or soft soils develop positive excess pressures. The rate of excess pore water pressure dissipation with time after pushing of the probe is stopped is a function of the permeability and compressibility of the soil surrounding the tip of the probe. It should be possible to develop theoretical relations between these soil properties and the time rate of dissipation of excess pore pressure; nevertheless, until such relations are available, the data gives a qualitative indication of the type of soil being penetrated."

In 1975 Richards et al described a piezometer probe which measured differential pressures in the sea floor primarily for the purpose of obtaining a better understanding of submarine slope stability. The opening statement of the paper is quoted verbatim :

"More than a decade ago it was proposed that the likely relationship between excess pore water pressure in cohesive sediments and potential submarine slides, slumps and possibly turbidity current could be investigated by the in situ measurement of pore pressure in sea floor sediments (Richards, 1962). Negotiations with the Norwegian Geotechnical Institute (NGI) in 1964-5 led to the acquisition by the University of Illinois, Urbana, of two differential piezometer probes and a counter computer in 1966. This paper describes the probe system that was built around these units and the results of its limited testing at sea in 1967.

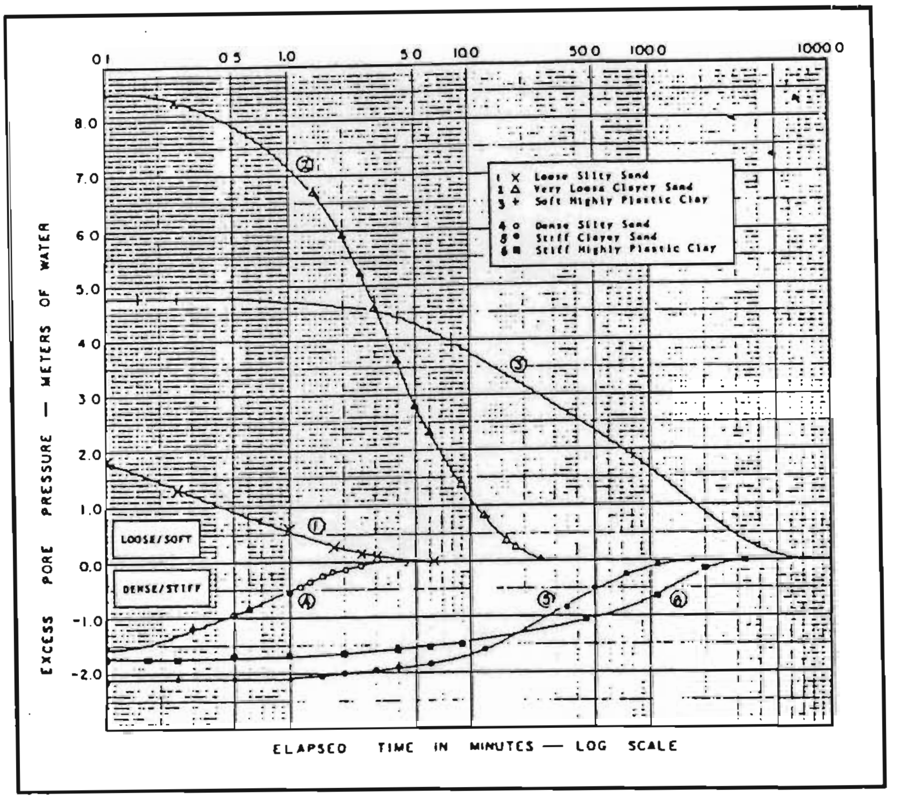


Figure B3.4 : Time-rate of dissipation of excess pore pressures (Wissa et al, 1975)

Previously the system was mentioned by Richards (1968) and Richards and Keller (1968). One test was very briefly summarised by Lai et al (1968)".

It is surprising that this earlier work should not have evoked more response from those involved in cone penetration testing. The earlier references were to discussion at a Geotechnical Conference, Oslo, to a paper in the American Association Petroleum Geology Bulletin and in American Geophysics Union Transactions and these were either not seen or the implications not appreciated by other cone practitioners. To some extent this demonstrates the difficulty in assessing claims for originality in the development of new ideas and equipment.

In 1975, Levillain described a piezometer cone, la sonde piézométrique, which measured only pore pressures. The primary purpose was to emphasize the pneumatic pressure measuring system which could be installed in a number of different piezometers ie not necessarily only in the cone push in type. No results of testing in soils were given.

In 1976 Parez, Bachelier and Sechet described the equipment and results obtained for a piezometer cone again only for measuring pore pressures not cone pressures. They used both a cylindrical filter in the shaft of the cone and individual small filters in the face of the cone and noted that very similar results in pore pressure magnitude and sign were obtained for both cones. They placed strong emphasis on the necessity for a rapid response instrument. The primary purpose of the work reported was to assess effective stresses around cones and hence adjust both cone and friction sleeve resistances to be used in the design of piles.

In 1978 Schmertmann published a comprehensive report on the use of the Wissa 1975 type piezometer probe to identify liquefaction potential of saturated fine sands. He compared the performance of a Wissa 20°, 1,75 in dia cone and a Wissa 60°, 1,5 in dia cone, the latter being similar in shape and size to the standard Dutch cone, except for the filter element at the tip, and concluded that the probes gave similar results but with the 20° cone giving better layering detail and the 60° cone giving a greater magnitude of generated pore pressure. He mentioned that the rate of dissipation of pore pressure could perhaps form the basis of estimating permeability and described the de-airing procedures and precautions required to keep the filters saturated in the field.

In 1978, Baligh, Vivatrat and Ladd published a comprehensive research document on the exploration and evaluation of engineering properties for foundation design of offshore structures which concentrated on cone penetration testing. This, inter alia, described further testing with the Wissa-type piezometer cone with the filter at different positions. It also summarized current theories on cone penetration. It noted that pore pressures dissipated on stopping penetration and that the study of dissipation rates and hence consolidation characteristics was being undertaken separately.

In 1979, Peignaud, described the driving of a cone shaped piezometer into clayey soils. He discussed the influence of the filter position on the pore pressures recorded and noted how this makes it difficult to compare test results using different cones. The tests were carried out at different penetration rates which was the primary emphasis of the work. In the clays tested the pore pressures generated were negative during penetration and became positive after dissipation to values dependent on the overburden pressure. At the range of penetration rates tested ie 5, 10 and 25 mm/s the negative excess pore pressures generated were penetration rate dependent but the final pressures were very similar for all rates. Peignaud showed an interesting result of negative excess pore pressure generated against degree of overconsolidation which is reproduced in Figure B3.5.

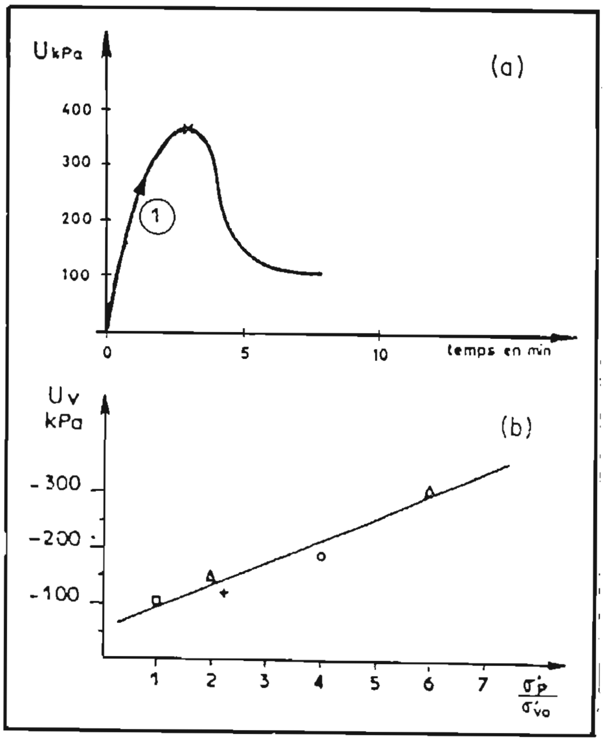


Figure B3.5 : a) Dissipation of pore pressure u with time (Peignaud, 1979)
b) Value of u_v as a function of OCR

In 1980, Baligh and Levadoux; Levadoux and Baligh; and Baligh, Azzouz and Martin published research reports on pore pressure dissipation after cone penetration, on pore pressures during cone penetration in clays and cone penetration tests offshore the Venezuelan coast. These detailed reports followed up the 1978 reported work. The cones used were the Wissa type, both 18° and 60°, and with the filter elements in different positions, and had not changed since the 1975 versions other than in some relatively minor details.

Also in 1980, Baligh, Vivatrat and Ladd presented a paper on soil profiling using pore pressure ratios which was a summary of the work described in their 1978 research report.

From 1981 onwards, in the space of two or three years, piezometer cone penetration testing measuring both cone and pore pressures moved forward in almost a quantum leap from an experimental tool to an off the shelf commercial production unit.

The author was responsible for four papers in this period viz, for 10th Int. Conf. SMFE, Stockholm 1981; ASCE Symposium Cone Penetration Testing and Experience, St Louis, 1981; ESOPT II Amsterdam, 1982; International Symp. In situ Testing, Paris 1983.

These described the South African development of and experience with the new piezometer cone which for the first time measured cone and pore pressures simultaneously.

As far as the author is aware these papers, and others at the same conferences, were the first to describe such piezometer cones. The author's papers, as well as giving results for a number of sites where piezometer cone testing had been carried out, also gave a method of deriving coefficients of consolidation from pore pressure dissipations and a method for soils identification from a comparison of generated pore pressures with cone pressures.

At the first of these conferences two other papers on similar piezometer cones appeared by Franklin and Cooper (1981) and by Parez and Bachelier (1981). Franklin and Cooper referred to their cone as a prototype developed from the Wissa probe in so far as the pore pressure measurements were concerned and having the filter element at the tip. The cone included strain gauge load cells for both cone and friction sleeve measurements. They did not draw firm conclusions from the limited testing in recent sands, but confirmed general trends previously observed by others using the Wissa probe. The paper indicated that the probe itself performed without problems provided that de-airing was carefully done and subsequent saturation ensured.

Parez and Bachelier described a piezometer cone with a conventional cone pressure measuring system and a cylindrical pore pressure measuring system in the shaft above the cone, ie strictly not a piezometer cone but having some of the same purposes. On the basis of the simplified approach of a cylindrical dissipation model used for sand drain design they suggested a time factor could be derived, for the estimation of coefficients of radial consolidation.

At the ASCE Symposium on Cone Penetration Testing and Experience, 1981, there were a number of papers describing piezometer cones and testing viz Baligh et al; Battaglio et al; Campanella and Robertson; de Ruiter; Marr; Muromachi; Tumay et al; and that by the author and colleague, Jones and Rust.

At the Second European Symposium on Penetration Testing, 1982, ESOPT II, some of the same authors had further publications together with papers from Abelev; Lacasse and Lunne; Marsland and Quarterman; Rocha Filho; Senneset et al; Smits; Sugawara et al; Tavenas et al; Torstensson; Tumay et al; Zuidberg et al including the author and colleague, Jones and Rust, (1982). Of the 86 papers on cone penetration testing 16 described piezometer cones and gave test results.

In 1982 two papers were published in the Canadian Geotechnical Journal by Roy, Tremblay, Tavenas and La Rochelle which described a piezometer cone and its use in sensitive clays, and in 1983 by Campanella, Robertson and Gillespie on testing in deltaic soils. Later in the same year Robertson and Campanella published a paper in two parts on interpretation of cone penetration tests in sands and in clays. These were in essence state of the art papers based on world wide experience recorded in the referenced literature.

In the following few years relatively little was published on piezometer cone testing that made any significant advances in the equipment or test methods, but the theoretical understanding had improved as better insight into the soil behaviour around a penetrating cone could be obtained from many more detailed field observations coupled with laboratory testing.

At the 11th ICSMFE, San Francisco, 1985, Campanella et al and Jamiolkowski et al presented recent developments of in situ testing and new developments in field and laboratory testing which indicated little new development since ESOPT II in 1982.

At the ASCE Speciality Conference, In situ 86, Blacksburg, a number of papers were published on piezocone testing by Keaveny and Mitchell; Lunne et al; Mayne; Olsen and Farr; Robertson et al.

At ISOPT I in 1988 at Orlando, Campanella and Robertson presented a special lecture on the current status of the piezocone test. Essentially this had little new to report since the burst of information in 1981/82, but it reflected a much broader acceptance of the method and the search for greater understanding of the results obtained. At the instigation of those most involved in piezocone testing, including the author, a section was included in the International Reference Test Procedures on the piezocone with the intention of reinforcing acceptance of the technique.

Special lectures were also given on cone penetration testing by Parkin, Mitchell, Jamiolkowski et al, Hughes, and Wroth. Wroth's contribution should be singled out for it urged that the many correlations suggested for the CPT results and in particular for CPTU results should "be based on physical insight, set against a theoretical background, and be expressed in suitably dimensionless form".

At ISOPT I there were nineteen technical papers on piezometer cone testing out of a total of sixty CPT papers. Other than those describing special purpose cones, the remaining papers barely described the cones themselves; they concentrated on the results obtained from actual testing and derived relationships for defining overconsolidation, shear strength parameters, consolidation parameters and soil classification. Four of these discussed overconsolidation or stress history viz Mayne and Bacchus; Rad and Lunne; Sugawara; Sully, Campanella and Robertson and the importance of the OCR in predicting the behaviour of soils was stressed.

The paper by Houlsby and Teh who used a strain path method and a large strain finite element analysis, has added significantly to the insights on cone penetration testing . A finite difference method is used for the dissipation analysis and a new method of interpreting piezocone pore pressure dissipation data is given.

Four of the papers at ISOPT I described the use of pore pressure dissipation tests in order to derive coefficients of consolidation viz Lutenegger et al; Senesset et al; Sills et al; Tang Shidong and Zhu Ziao-Lin. The methods used are all similar and could almost be called conventional by this stage. The paper by Sills et al is particularly valuable in that it compared coefficients of consolidation derived from the piezocone (using Baligh and Levadoux, 1980), from laboratory tests and from back analysis of the settlement of the embankment.

The Lutenegger et al paper also compared piezocone derived parameters (using Gupta and Davidson, 1986) with laboratory and field data. They claim "excellent agreement" but in fact their results appear to show almost an order of magnitude difference between the field measured coefficients of consolidation (based on in situ piezometer readings) and the piezocone data, with the laboratory data spread covering this range.

Senneset et al, using Torstensson's spherical expanding cavity solution and their definition of the effective sphere radius and also the 1982, Senneset et al, method, derived coefficients of consolidation, from cone dissipation tests and compared these to laboratory test results and claim the "data seem to correlate fairly well".

The Tang Shi-dong paper, which is primarily concerned with piling, made only general reference to the dissipation of pore pressures insofar as they influence pile bearing capacity.

The remaining dozen papers on piezocone in the 1988 ISOPT I discussed a variety of aspects which, although of interest to piezocone practitioners, are not directly relevant to this present work on embankment settlement.

ISOPT I can be seen as representing the coming of age of piezocone equipment development and to a lesser extent of the interpretation methods. It was perceived that the initial almost dramatic advances had been made and that a period requiring solid definitive testing for data acquisition was necessary so that the most appropriate theoretical models could be calibrated. This is not an easy task since it will require high quality piezocone testing, high quality sampling and laboratory testing, followed by performance monitoring of constructed works.

B4 CURRENT METHODS FOR THE INTERPRETATION OF CONE PENETRATION TESTING

B4.1. Introduction

At the beginning of the period covered by the author's close involvement with cone penetration testing ie 1965, the cone systems on a world wide basis were the simple mechanical type. The Dutch 60°, 10 cm^2 mantle cone, with or without a friction sleeve was the most popular of these. The body of knowledge was firmly in western Europe whence the methods of interpretation emanated.
These were comprehensively described by Sanglerat (1972). At the earlier stages the emphasis of cone penetration testing had been on the assessment of the characteristics of cohesionless soils for the purpose of foundation design, primarily for piles. Certainly Sanglerat covered a much wider scope than solely pile design, but many of the contributions in the chapters describing experiences in different countries concentrated on this aspect and often on deriving correlations between what was then called Dutch Cone Penetration Testing, or quasi state penetration testing and Standard Penetration Testing - SPT - so that any design method applicable for the latter could be simply adapted for use with the CPT.

Nevertheless interpretation methods for all aspects of the CPT were described by Sanglerat and have not in most cases significantly changed except insofar as the advent of the piezocone has allowed development of these methods.

The following sections describe the current generally accepted interpretation methods as they apply to the establishment of the characteristics of alluvial deposits which are required for the design of embankments. These characteristics are shear strength, compressibility, consolidation, soils identification and over consolidation ratio. The application of these to embankment design is discussed in B5.

B4.2 Shear Strength from Cone Penetration

Shear strength derivations view cone penetration as a classic bearing capacity problem which can be expressed as :

$$q_f = c_u N_c + \sigma_{vo} N_q + 0,5\ B_\gamma N_\gamma \qquad \text{B4.1}$$

where q_f = ultimate bearing pressure

c_u = undrained shear strength

σ_{vo} = overburden pressure

B = foundation width

γ = subsoil density

and N_c, N_q and N_γ are bearing capacity factors depending on the shape of the foundation and the friction angle ϕ of the soil.

B4.2.1 Cohesive soils

For the case of undrained shear in a clay where $\phi = 0$, since $N_\gamma = 0$ when $\phi = 0$ and $N_q = 1$ then the equation B4.1 reduces to :

$$q_c = c_u N_c + \sigma_{vo} \qquad \text{B4.2}$$

where q_c = cone pressure

However, contrary to conventional bearing capacity theory which dealt with surface or shallow foundation and for which values of N_c could be computed and depended only on the geometry, it was shown by Gibson (1950) that for a deep circular foundation N_c was also dependent on the rigidity index I_r of the soil, where $I_r = G/s_u$:

$$N_c = \frac{4}{3}\left(\ln \frac{G}{s_u} + 1\right) + 1 \qquad \text{B4.3}$$

At a typical rigidity index of 200 the equation B4.3 gives an N_c value of 9.4 and would require a rigidity index of about 10^4 to give an N_c value of about 15. Experience shows that this analysis for a deep circular foundation does not give appropriate values for N_c for cone penetration testing and the problem becomes one of determining such values. The well established soil mechanics route has been followed of empirically determining values of N_c by carrying out independent measurements of undrained shear strength, say from undisturbed sampling and undrained triaxial testing. Much research has been carried out in this way to determine N_c for a range of soils and indeed it continues to be recommended practice that local correlations should be made for any clays where values are not already available. Tables of values are available based on descriptions of the clay type, the sensitivity and overconsolidation ratio. An ongoing difficulty with this approach is the implication that for a particular clay the undrained shear strength has a unique value. It has been generally recognized and discussed by Wroth (1984) that the undrained shear strength is a function of the type of test. Figure B4.1 taken from Wroth demonstrates this and it follows that to be consistent a hierarchy of N_c values should be used in equation B4.2 depending on which equivalent undrained shear strength is required from cone penetration testing. An alternative approach is to achieve consensus on how N_c should be derived for cone penetration testing whether it be strictly theoretical or empirical, and use this defined set of values to determine c_u (cone) which will then become an additional undrained shear strength in the existing hierarchy. Wroth (1988) recommends the former approach and suggests that, "undrained triaxial compression test (after appropriate reconsolidation of the specimen) - is used exclusively in all such correlations."

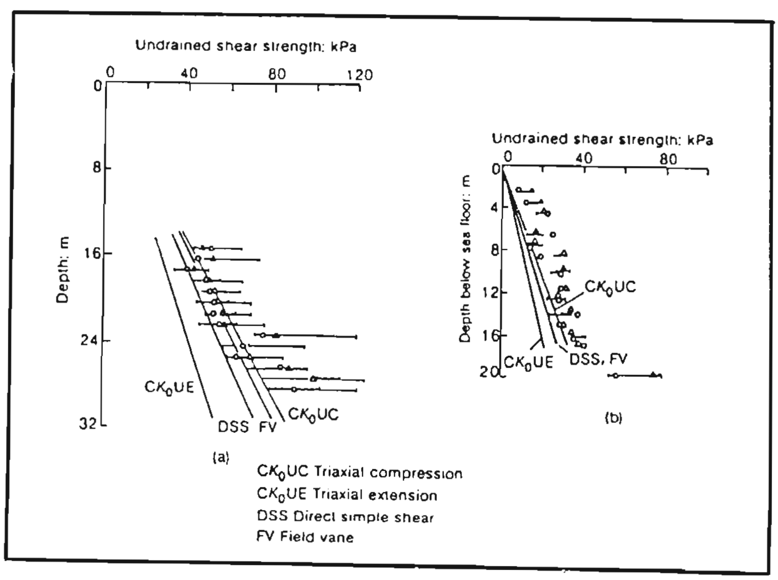


Figure B4.1 : Undrained shear strengths in different tests (Wroth, 1984)

Houlsby and Teh suggest that the cone factor N_{kt} defined as :

$$N_{kt} = \left(\frac{q_t - \sigma_{vo}}{s_u} \right)$$

can be represented as :

$$N_{kt} = \frac{4}{3}(1 + lnI_r)\left(1{,}25 + \frac{I_r}{2000}\right) + 2{,}4\ \alpha_f - 0{,}2\ \alpha_s - 1{,}8\Delta \qquad B4.4$$

where the last term $\Delta = (\sigma'_{vo} - \sigma'_{ho})/2s_u$ is the initial shear stress ratio and is a correction to allow for the "incorrect" use of $(q_t - \sigma_{vo})/s_u$ instead of $(q_c - \sigma_{ho})/s_u$ to define N_{kt} and the α terms relate to the cone face and shaft roughness. The second term with I_r is a correction since finite element modelling showed some deviation from the theoretical spherical expanding cavity solution. Figure B4.2 and B4.3 show N_{kt} varying with Δ and I_r when α is constant and demonstrates the high dependence on Δ and I_r. In practice the variation is conditioned by the fact that for any given soil, variations in Δ and I_r with OCR tend to cancel each other out.

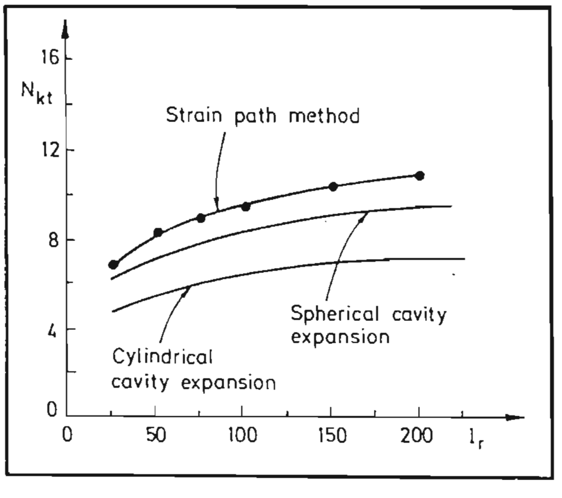


Figure B4.2 : Cone factors from strain path method (Houlsby and Teh, 1988)

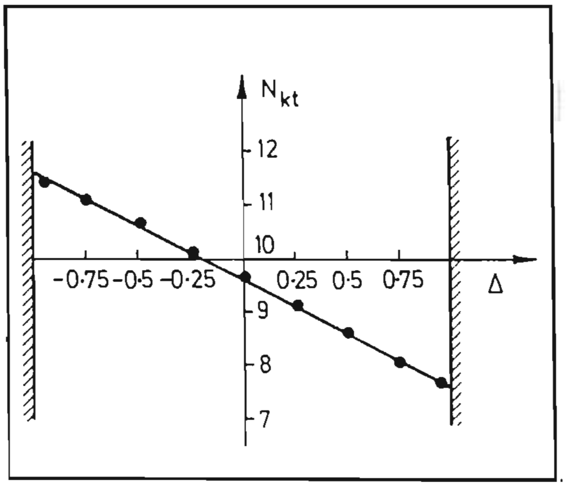


Figure B4.3 : Variation of cone factor with Δ (Houlsby and Teh, 1988)

The N_{kt} values obtained agree more closely with those found in practice than any other theoretical method and this approach appears most promising.

A consequence is that equation B4.2 continues be valid but that the value of N_c can be derived theoretically and is dependent inter alia on the rigidity index. Unfortunately, however, this index has a very wide range and can only be measured by extensive laboratory testing. It must be expected, therefore, that values of N_{kt} (which have a fairly small range) derived from experience will continue to be used in practice albeit with more effort being expended to justify the Houlsby and Teh approach.

B4.2.2 Cohesionless soils

The author's work is concerned primarily with the application of the CPT to embankment design on predominantly cohesive soils so the shear strength of sands is relatively unimportant since it is not critical in embankment performance. Little discussion is therefore given to this aspect.

For cohesionless soils equation B4.1 reduces to :

$$q_c = \sigma_{vo} N_q \qquad \text{B4.5}$$

and N_q varies non linearly in the range 0 - 500 as ϕ increases from zero to 50°. This suggests that q_c should increase indefinitely as the depth increases but practice shows this does not happen and different materials have well defined upper bound values of q_c. The simplistic model is therefore inadequate to cover all situations but nevertheless considerable experience has shown that cone resistance can be related to stress level, relative density and angle of shearing resistance. Many analyses have derived angles of friction, ϕ, directly from relationships between ϕ and q_c eg de Beer (1948), Harr (1977), Durgunoglu and Mitchell (1975), and it is probably the last of these which is favoured.

The stress level has generally been represented by the ambient vertical effective stress σ'_{vo} rather than by the horizontal effective stress, σ'_{ho}, since σ'_{vo} can be calculated or estimated from the soil unit weight and hydrostatic conditions whereas σ'_{ho} cannot readily be obtained. Recent work, however, has confirmed that it is the horizontal stress that controls penetration resistance. Wroth (1988) categorically states that the conventional practice of normalizing cone pressure with respect to σ'_{vo} should be discontinued and that σ'_{ho}, the horizontal effective stress should be used. Been and

Jefferies (1985) and Been et al (1987) have defined a state parameter which describes the condition of a sand. This parameter Ψ is the difference between the actual void ratio of the sand, e_o, and the void ratio, e_{ss}, at the steady state at the same mean effective pressure, p', (p' = 1/3 (σ'_{vo} + 2 σ'_{ho}). The state parameter combines the influence of relative density and stress level and leads to the proposed relationship :

$$\frac{q_t - p_o}{p'_o} = k \exp(-m\psi) \qquad \text{B4.6}$$

where k and m are soil constants related to the gradient of the steady state line, λ_{ss}, in a plot of e against ln p'. In order to evaluate the above both σ'_{ho} and λ_{ss} are required, and k and m are given by Been et al for a wide range of sands.

A similar position is therefore arrived at as for cohesive soils which is that for proper interpretation of cone data some independently acquired information is necessary. In routine testing it is unlikely that this more sophisticated approach requiring significantly additional investigation would be undertaken and reliance is therefore placed on simpler empirical interpretation calibrated by local experience.

B4.3 Compressibility from Cone Penetration Testing

The first publication on settlement prediction from CPT was that by de Beer and Martens (1957) and in practice little has changed in principle since then. The basic settlement equation is :

$$\Delta h/h = \frac{1}{C} \ln\left(1 + \frac{\Delta\sigma_v}{\sigma_{vo}}\right)$$

or

$$\Delta h/h = \frac{2,3}{C} \log\left(1 + \frac{\Delta\sigma_v}{\sigma_{vo}}\right) \qquad \text{B4.7}$$

where

Δh	=	settlement of elemental layer
h	=	thickness of elemental layer
C	=	constant of compressibility of layer
$\Delta\sigma_\nu$	=	increase in pressure in layer due to load
σ_{vo}	=	initial stress in layer

The total settlement is obtained by adding the settlements of all layers.

In order to obtain stresses in the individual soil layers a Boussinesq stress distribution is used and the problem becomes one of ascribing a value to C on the basis of q_c the cone pressure. Buisman (1935), using a Boussinesq stress distribution for the penetration of a hemisphere, arrived at the expression, for a cohesionless soil :

$$C = 3/2\ q_c/\sigma_{vo} \qquad \text{B4.8}$$

which by substitution in equation B4.7. leads to :

$$\Delta h/h = \frac{2,3\ \sigma_{vo}}{1,5\ q_c} \log\left(1 + \frac{\Delta\sigma_v}{\sigma_{vo}}\right) \qquad \text{B4.9}$$

Experience showed that settlements in sands computed on this basis were too high and a number of authors suggested modifications to improve the predictions, notably Meyerhof (1965) and Schmertmann, (1970) and gave values of 1.9 and 2 respectively instead of 3/2 in B4.8, thus the elastic modulus, E, is twice the cone pressure, q_c.

This approach has subsequently been used for cohesive materials by replacing the coefficient 3/2 (later 2) by a factor α_m which is dependent on the nature of the soil hence :

$$q_c = \frac{1}{\alpha_m m_v} \qquad \text{B4.10}$$

$$M = \alpha_m q_c \qquad \text{B4.11}$$

α_m	=	constrained modulus coefficient
m_ν	=	coefficient of volume compressibility
M	=	constrained modulus

A large body of research has been concerned with the evaluation of α_m for a range of materials by comparing cone penetration test q_c values in the field with laboratory consolidometer measured m_v values from undisturbed samples taken at corresponding depths. Bachelier and Parez (1965) and Gielly et al (1969) published comprehensive correlations of q_c with m_v. The latter gave a different definition to the constrained modulus coefficient using $\alpha_m = 2{,}3/\alpha_o$ but generally both data sets gave similar values for comparable soils. These were summarized by Meigh (1987) and are given in Table B4.1.

Table B4.1 : Coefficient of constrained modulus for normally-consolidated and lightly overconsolidated clays and silts (after Sanglerat, 1972)

Soil	Class	α_m (= M/q_c)	
		Mantle Cone	Reference Tip
Highly plastic clays and silts	CH, MH	2 to 6	2.5 to 7.5
Clays of intermediate or low plasticity	CI, CL		
$q_c < 0,7$ MN/m^2		3 to 8	3.7 to 10
$q_c > 0,7$ MN/m^2		2 to 5	2.5 to 6.3
Silts of intermediate or low plasticity	MI, ML	3 to 6	3.5 to 7.5
Organic silts	OL	2 to 8	2.5 to 10
Peat and organic clay :	Pt, OH		
50 < w < 100%		1.5 to 4.0	1.9 to 5.0
100 < w < 200%		1.0 to 1.5	1.25 to 1.9
w > 200%		0.4 to 1.0	0.5 to 1.25

The results given by Gielly et al (1969) were also given in the form of C_c against q_c and are shown in Figure B4.4.

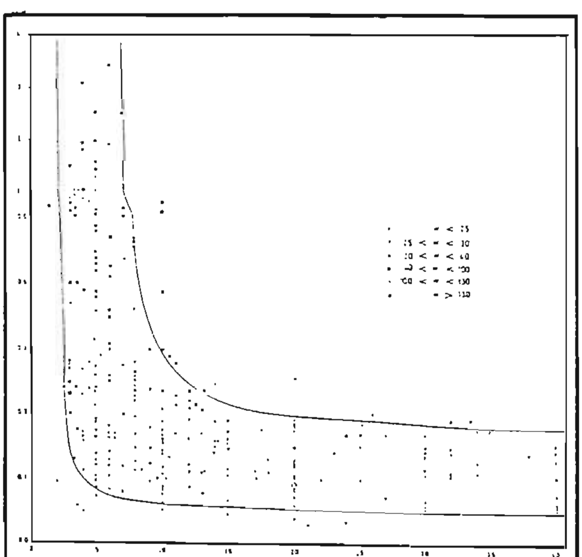


Figure B4.4 : Compression index C_c, versus cone pressure, R_p (Gielly et al, 1969)

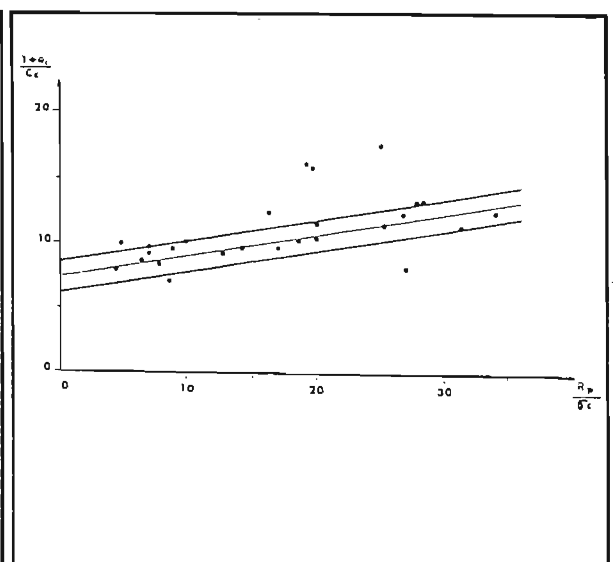


Figure B4.5 : $(1 + e_c)/C_c$ against R_p/σ_c (Gielly et al, 1969)

The data extracted from this comprehensive laboratory testing included Atterberg Limits and natural moisture contents and Gielly et al state that general agreement is shown with Skempton's (1944) equation for undisturbed normally consolidated clays although there is a large scatter :

$$C_c = 0{,}007\ (w_L - 10) \qquad \text{B4.12}$$

The generally held view is that correlations of C_c with liquid limit are relatively poor so the scatter does not necessarily cast doubt on the C_c against cone pressure relationships and therefore on the α_0 coefficients.
Gielly et al also show a diagram of $(1 + e_c)/C_c$ against R_p/σ_c where e_c and σ_c are the void ratio and consolidation pressure. (R_p is the probe resistance or cone pressure, q_c).

This is the straight line relationship, equation B4.13, shown in Figure B4.5, taken from Gielly et al paper. This serves to confirm that compressibility is proportional to the cone pressure but that the relationship is conditioned by the void ratio :

$$\frac{1 + e_c}{C_c} = 7{,}4 + 0{,}16\ \frac{R_p}{\sigma_c} \qquad \text{B4.13}$$

Schultze and Menzenbach (1961) and later Schultze and Mezler (1965) carried out laboratory and field tests to obtain correlations of SPT N values (1961) and subsequently cone pressures, q_c, (1965), against coefficient of compressibility, m_v. They proposed equation B4.14 given below in which the constants C_1 and C_2 have a range of values dependent on the position of the field test relative to the water table and the nature of the material :

$$\frac{1}{m_v} = C_1 + C_2\ N \qquad \text{B4.14}$$

Table B4.2 gives values for C_1 and C_2 and it can be seen from the correlation coefficient that for sands and clayey sand the correlation is excellent and good respectively, but for sandy clay is much poorer.

Table B4.2 : Coefficients C_1 and C_2 for different material types

Soil Type	Above Water Table	Below Water Table	Sand	Clayey Sand	Sandy Clay	Loose Sand
No of tests	15	17	14	19	27	18
C_1 (bar)	52	71	39	43	38	24
C_2 (bar/blow)	3.3	4.9	4.5	11.8	10.5	5.3
Correlation coefficient	0.758	0.900	0.954	0.886	0.783	0.764

Based on the Schultze and Menzenbach (1961) publication the author developed similar equations for alluvial deposits in Durban from the results of a large (6,1 m x 6,1 m) plate loading test and from screw plate tests. The equation B4.14 and factors C_1 and C_2 were adapted from SPT to CPT by using the then generally accepted q_c to N value correlations and adjusted on the basis of the plate test results. These were initially reported by Kantey (1965) in discussion at Montreal and later in a paper by Webb (1969). These equations, devised largely by the author, are given below :

Fine to medium sand :

$$M = \frac{1}{m_v} = 5/2\ (q_c + 3000) \qquad \text{B4.15}$$

Clayey sands ($I_p < 15$) :

$$M = \frac{1}{m_v} = 5/3\ (q_c + 1500) \qquad \text{B4.16}$$

Estuarine deposits, mixed layered sands, silts, clayey sands, clays :

$$M = \frac{1}{m_v} = 2\ (q_c\ +\ 2500) \qquad \text{B4.17}$$

In the above equations q_c and m_v are in kPa and m^2/KN respectively.
These equations were derived from large scale field tests with the backing of extensive laboratory testing on the more readily sampled clayey material and subsequently calibrated against the measured settlements of large fills (10 m high 33 m wide). They continue to be used with some confidence in estuarine deposits. The equations give higher values of α_m, hence somewhat lower values of m_v, than either the original Buisman (α_m = 3/2) or the modified Meyerhof and Schmertmann (α_m = 2,0) correlations as shown in Figure B4.6.

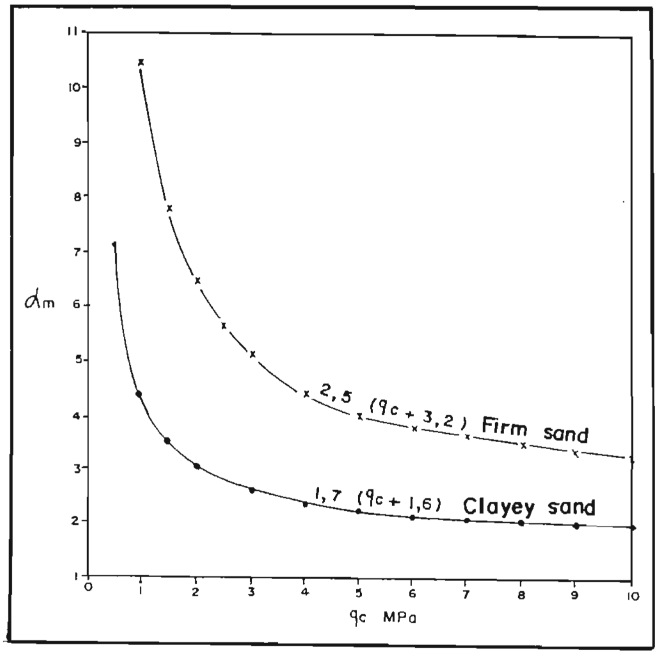


Figure B4.6 : Constrained modulus coefficient versus cone pressure

Although α_m is in reasonable agreement with Schmertmann's value of 2,0 above q_c of about 2 - 3 MPa, the divergence at low pressures becomes large. This has the effect of reducing the rate of change of modulus with decreasing cone pressure since the constants in the equations viz 3000 and 1500 kPa become dominant at low cone pressures. For example in equation B4.16 if q_c halves from 1000 kPa to 500 kPa then the constrained modulus only reduces from about 4 MPa to 3 MPa; also there is a minimum modulus, when q_c is zero of 2,5 MPa. Therefore these equations cannot be considered reliable at low cone pressures (say < 2 MPa).

Nevertheless, the author contends that because of the original derivation of these equations from large scale field tests, and calibration against embankment settlements, they may be used as one of the methods of settlement prediction for sands, silty sands and clayey sands where cone resistances are not less than about 2 MPa. Below this cone pressure the under conservative M values derived from the equations may result in under prediction of settlements.

It is clear then that an experimental approach has been adopted for deriving coefficients of compressibility by comparing laboratory consolidometer values with q_c measurements in the field hence obtaining values of α_m, the constrained modulus coefficient. This is, a valid approach and because no satisfactory comprehensive theoretical understanding of cone penetration has yet been achieved it must for the time being remain the pragmatic approach. Nevertheless it is important to examine the basic assumption that cone pressures are a measure of compressibility of clays.

It is generally agreed that undrained shear strengths, s_u, can be reliably modelled in cone penetration using :

$$s_u = \frac{q_c - \sigma_{vo}}{N_{kt}} \qquad \text{B4.2}$$

N_{kt} was originally obtained from simple bearing capacity theory and confirmed in practice by many correlations of cone pressure either with triaxial testing or with in situ shear vane test. More recently, as discussed earlier in this section, Houlsby and Teh derived a theoretical model for N_{kt}, given as equation B4.4 which is dependent, inter alia on the rigidity index I_r of the soil.

The implication of the assumption that :

$$M = \frac{1}{m_v} = \alpha_m q_c \qquad \text{B4.10}$$

is that :

$$\frac{1}{m_v} = \alpha_m (N_{kt} s_u + \sigma_{vo}) \qquad \text{B4.18}$$

Using the simplifying assumptions that for soft saturated medium plasticity clays $\sigma'_{VO} \approx 2/3\ \sigma_{VO}$ that $s_u/\sigma'_{VO} \approx 0{,}3$, and $N_{kt} \approx 15$;

$$\frac{1}{m_v} = \alpha_m s_u (N_{kt} + 5)$$

$$\frac{1}{m_v} = 20\alpha_m s_u \qquad \text{B4.19}$$

and from Poulos (1975) :

$$E' = \frac{(1-2\nu')(1+\nu')}{(1-\nu')\ m_v} \qquad \text{B4.20}$$

and :

$$E_u = \frac{3\ E'}{2(1+\nu')} \qquad \text{B4.21}$$

then :

$$\frac{1}{m_v} = \frac{2(1-\nu')}{3(1-2\nu')} E_u = 20\alpha_m s_u \qquad \text{B4.22}$$

For soft clays the drained Poisson's ratio varies from 0,45 to 0,35 and from 0,35 to 0,30 for firm clays (Poulos, 1975)

Therefore for soft clays :

$$E_u = 5{,}4\ \alpha_m\ s_u \text{ to } 13{,}8\ \alpha_m\ s_u \qquad \text{B4.23}$$

and for firm clays :

$$E_u = 13{,}8\ \alpha_m\ s_u \text{ to } 17{,}2\ \alpha_m\ s_u \qquad \text{B4.24}$$

It has been found from laboratory tests correlated with CPT's that for clays the range of α_m values is fairly limited viz 2 to 10 (see Table B4.1) and from equation B4.24 it can be seen that E_u/s_u should therefore be in the range of about 25 to 175.

Observations of settlements of embankments and structures indicate that E_u/s_uhas a very much wider range which questions the applicability of equations B4.23 and B4.24 since they are based on the assumption of elastic homogeneous isotropic soils.

Thus, equations B4.23 and B4.24 are interesting rather than practically useful and heighten the dilemma illustrated by the Gielly et al data, which is that although there appears to be relationships between E_u and s_u and therefore between E_u and q_c the relationships are ill defined. It is possible that by definition of the initial state of the soil, more useful E_u/s_u values could be derived, and since the initial state of clays can be largely defined by the initial void ratio and the over consolidation ratio (OCR), there is the possibility that valid E_u/s_u relationships can eventually be obtained. In practical terms the void ratio can be estimated from the natural moisture content and the OCR from local knowledge, and to some extent from cone penetration testing; therefore it is not unreasonable to use these parameters to attempt to define α_m for a particular soil within the general range of α_m values.

Simons (1974, 1975) emphasizes that the range of E_u/s_u values is very large, 40 - 3 000, and implies that there is so little discernible pattern to the relationship between E_u and s_u that this approach to settlement prediction is unreliable.

He points out that "the shear stress level is a factor which has a great influence on E_u, low values of E_u/s_u would be expected for highly plastic clays with a high shear stress level, and higher values for lightly loaded clays of small plasticity". It is therefore necessary to define the factors which influence E_u/s_u. The time honoured soil mechanics methods are either an adequate theoretical model or alternatively sufficient empirical data to substantiate a relationship.

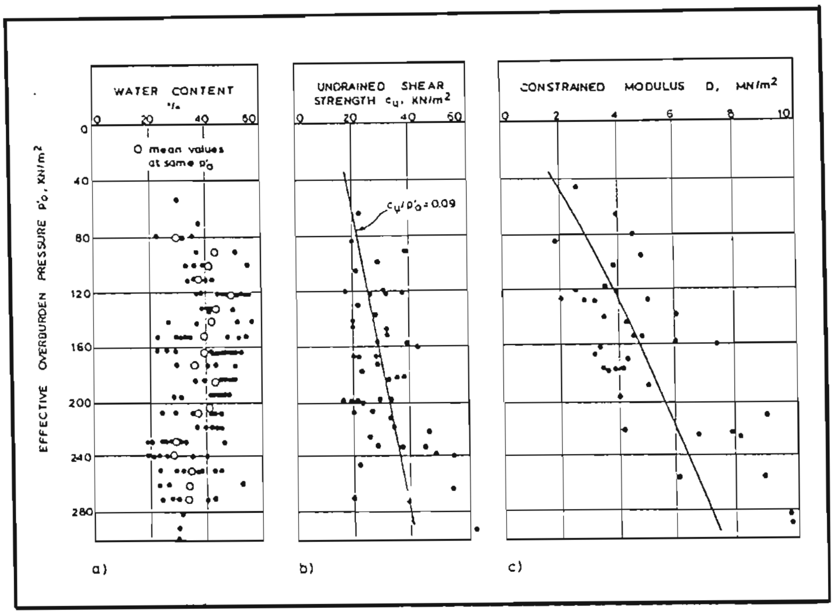


Figure B4.7 : Water content, undrained shear strength and constrained modulus against effective overburden pressure (Coumoulos and Koryalos, 1977)

Until such time as E_u/s_u can be comprehensively modelled it must be obtained empirically for specific soils .

For example Coumoulos and Koryalos (1977) give data for soft normally consolidated clays near Athens expressed as shear strength and deformation modulus against effective overburden pressure, Figure B4.7. They state that the individually calculated results give a E_u/s_u in the range 100 to 170 and that the relationship is linear within small increments of stress. However if the linear regression lines of s_u and E_u against effective stress are compared then clearly a linear relationship between the two is forced, with, in this case, a value of 280. Also the correlation coefficients for straight line modelling of both s_u and E_u are low hence not overmuch significance should be given to the tentative E_u/s_u relationships shown, other than that they demonstrate a valid practical approach. No specific data is given in the paper on possible overconsolidation hence the statement that the clay is only about 3 000 years old and "can be considered as normally consolidated". However the c_u/p_o' relationship, which is a straight line as would be expected for a normally consolidated clay, shows an intercept with the shear strength axis at zero effective overburden pressure of about

15 kPa, which is higher than would usually be expected for a normally consolidated clay (up to 5 kPa). The author suggests therefore, that the upper clay may be lightly overconsolidated, and hence the low value of s_u/σ_{vo} of 0,09 is not appropriate, which in turn invalidates the actual derived E_u/s_u value.

The s_u/σ'_{vo} can be compared with that reported by Kenney (1976) shown in Figure B4.8, or with the relationship between s_u/σ'_{vo} and plasticity index values given by Skempton (1954), see equation B4.52.

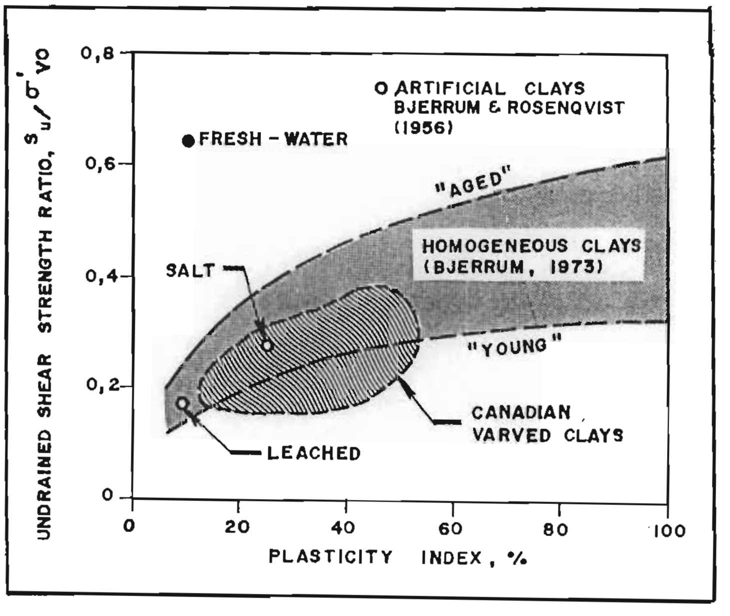


Figure B4.8 : Variation of strength ratio with plasticity for normally consolidated clays (Kenney, 1976)

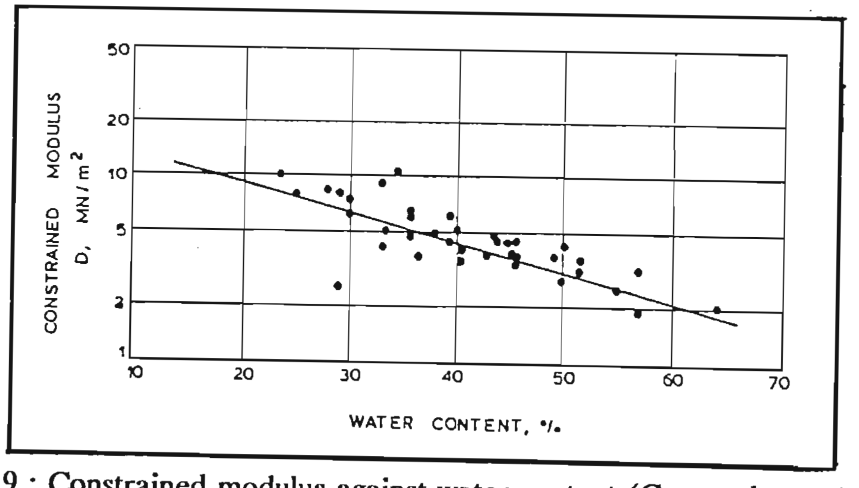


Figure B4.9 : Constrained modulus against water content (Coumoulos and Koryalos, 1977)

From the Coumoulos and Koryalos results it is clear that the selected s_u/p_o' of 0,09 is not consistent with the above empirical relationship and should not be considered as representing a normally consolidated clay. Similarly it can be seen from their natural moisture contents and plasticity data that the liquidity index is generally low (approximately 0,5) and does not decrease with depth, as would be expected for a normally consolidated clay. The plasticity indices for the layer under consideration encompass a very large range of about 10 to 50 and it must be concluded that describing this layer as uniform is misleading. Not surprisingly the relationships of E/s_u and s_u with effective overburden pressure and with one another are ill defined.

This is not intended as criticism but merely to highlight the very considerable difficulties in obtaining reliable data which could be used not only on a local basis but transferred to a data base from which to build general relationships for the determination of E_u/s_u.

Coumolos and Koryalos also give a relationship between constrained modulus and water content shown in Figure B4.9 which has a coefficient of correlation of 0,81. Whilst correlation with moisture content may be valid for a uniform clay, since the uniformity will by definition have eliminated most important characteristics - viz plasticity, stress state and overconsolidation, - correlation with a wider range of descriptors than solely moisture content must be necessary to generalize the situation. Tsotsos (1977) makes exactly this point and Figures B4.10 and B4.11 illustrate it. The lines shown on Figure B4.10 correspond to soils which fit the equation :

$$I_p = A\ (w_L - B) \qquad \text{B4.25}$$

where A varies from 0,78 to 0,36 and B correspondingly varies from 4 to 35, the centre of the five lines being the standard "A" line with values of 0,73 and 20. Figure B4.11 shows compression indices against moisture content for the same soils (up to a liquid limit of 80) and demonstrates the influence of the plasticity characteristics on the compression index since the moisture content is dependent on the plasticity in saturated soils. Using this approach and obtaining similar data for local soils it is then possible to assess the change in compressibility corresponding to changes in natural moisture content and in this way the constrained modulus coefficient, α_m, could be modified on the basis of the natural moisture content.

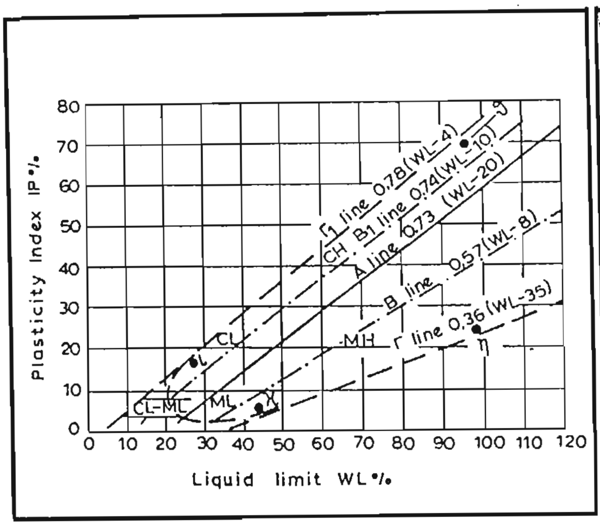


Figure B4.10 : Plasticity index versus liquid limit (Tsotsos, 1977)

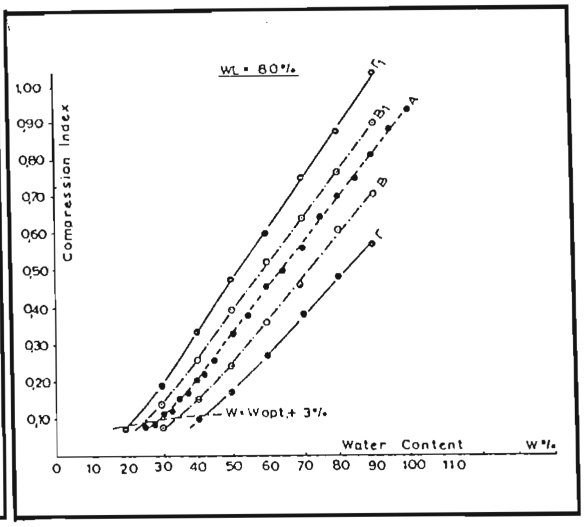


Figure B4.11 : Compression index versus water content (Tsotsos, 1977)

In summary the justification for the use of cone penetration testing in clays to measure compressibility remains that of practicality.

The wealth of field data reported in the literature gives moderately consistent values of the constrained modulus coefficient, α_m, directly related to cone pressures, q_c. The range of α_m values is, however, large and the selection of the appropriate value within the range is ill defined which results in crude estimates of settlement. It appears that better definition of the appropriate α_m value for a particular soil requires further parameters such as moisture content or void ratio as well as OCR and the applied stress level. An alternative is to derive locally applicable values of α_m from back analysis of settlements.

The lack of a theoretical basis does not invalidate the semi empirical approach and the situation may be considered as analogous to the measurement of undrained shear strengths by cone penetration testing. Initially values of the cone factor, N_{kt}, were obtained by comparative testing with laboratory or in situ vane tests, but now N_{kt} can be derived analytically, (Houlsby and Teh). As yet the analytical values do not agree closely with long established empirical values and the process of reconciling the differences will continue until agreement is reached.

B4.4 Consolidation Characteristics from Cone Penetration Testing

The problem of the prediction of embankment settlement is twofold in that the amount of settlement and the time it takes to settle are interrelated and of vital interest. If the settlement occurs rapidly, ie during construction, or at the other extreme say 50 years, then the amount of settlement is relatively unimportant. It is in the nature of things, however, that road, or rail, embankments on recent alluvial deposits fall between these two extremes so that reliable predictions of both amount and time of settlement are required.

The methods of prediction of settlement rate are relatively well established and depend on consolidation theory and appropriate laboratory testing on undisturbed samples to measure consolidation parameters. The consolidation theory usually applied is that of Terzaghi (1943). There are a number of limitations to this one dimensional theory, viz it assumes: only one dimensional drainage; D'Arcy's law applies for any hydraulic gradient; homogeneous fully saturated soil; soil grains and pore fluid are incompressible; constant compressibility and permeability; linear and time independent relationship between effective stress and void ratio (strain); infinitesimal strain rate and flow velocities, and it ignores secondary compression. To overcome at least some of these limitations many adaptations have been developed, Terzaghi - Rendulic or Davis and Poulos (1972), or alternative theories eg Biot (1941). Essentially the former two are three dimensional, the second allowing different rates of pore pressure dissipation and settlement - and the Biot theory allows the uncoupling of the direct proportionality between dissipation of pore water pressure and effective stress.

The use of the more refined theories, whilst philosophically more satisfying, do not seem to have produced any significant improvements in practice in the prediction of settlement rates. Significant over and under predictions are common and the potential errors are such that rational engineering decisions are extremely difficult on, for example, whether an embankment will require accelerated drainage, or even a structural solution, instead of construction in the available time.

The reasons given for the discrepancies are many and varied. The more common are that sampling disturbs the sample sufficiently to cause major changes in the consolidation characteristics; samples are not representative of the real conditions; the consolidation characteristics are stress dependent and the real stresses in the field are not modelled correctly; the theoretical consolidation model is not appropriate, and horizontal and vertical permeabilities are very different. In addition, it is recognized that the applied stress level relative to the preconsolidation pressure, σ_{vc}, is of vital importance and also that the accurate measurement of the latter is difficult in soft clays because of problems of sample disturbance. The rates of settlement in these smaller and larger strain zones each side of the preconsolidation pressure are different, hence it is essential to determine the coefficients of consolidation applicable for both zones.

Despite these difficulties the consensus remains that consolidation time predictions can satisfactorily be performed through the use of coefficients of consolidation which would usually be measured by laboratory tests. A major purpose of the author's work has been to evaluate coefficients of consolidation from cone penetration testing.

From the historical review given in B3 it can be seen that relatively few authors have addressed the problem of rates of dissipation of excess pore pressure around a piezocone in order to estimate coefficients of consolidation. Essentially they are Torstensson and Wissa et al in the 1975 ASCE Speciality Conference on In situ Measurement of Soil Properties, Raleigh; the 1980 research report publications by Baligh and Levadoux, and Baligh, Assouz and Martin, and the 1981 10th Conference ISSMFE, by Franklin and Cooper, by Parez and Bachelier and by Jones and van Zyl. Franklin and Cooper referred to Torstensson, 1975, for interpretation; Parez and Bachelier put forward a cylindrical solution acting as a reverse vertical sand drain and having a radius R equal 4r where r is the cone radius. The drain theory gave a time factor T_r of 0,03 for 50% dissipation and utilizing the expression between the coefficient of consolidation (radial) c_{vr} and the time for 50% dissipation, t_r, as follows :

$$c_{vr} = \frac{4R^2\, T_r}{t_r} \qquad \text{B4.26}$$

Parez and Bachelier (1981) indicate that for one of the two sites tested the agreement in c_v for the piezocone, laboratory and backfigured from a site record were very close viz 1.0, 1.5 and 1.0 x 10^{-2} cm^2/sec; whereas for the second site the piezocone and field results were 1,7 and 4 x 10^{-2} cm^2/sec respectively. The pore pressure was measured along a cylindrical section of the penetrometer some distance above the cone so the system was not a piezometer cone in the sense that cone and pore pressures were measured simultaneously at the cone.

The author, with van Zyl, (1981) eschewed a theoretical modelling approach and adopted a semi empirical method of direct correlation of piezocone measured, t_{50}, (time for half dissipation) with laboratory measured, c_v tempered with experience based on embankment settlement observations in selecting the appropriate laboratory measured c_v values. This resulted in the equation :

$$c_v = \frac{50}{t_{50}} \qquad \text{B4.27}$$

where t_{50} is in minutes
and c_v is in m^2/year

The temptation of having an easy to remember equation overcame the scientific compulsion to have consistent units and since in practice one generally measures t_{50} in minutes and requires the coefficient of consolidation in m^2/year for calculation of embankment settlement times, the form of equation is convenient.

The author, (Jones, van Zyl and Rust, 1981) justified this approach by using an idea based on Blight (1968) who estimated allowable vane shear testing rates based on consolidation of a sphere. If the penetration of a cone is stopped and no memory of how it arrived at the stop position exists, then the pore pressure dissipation pattern will be that for a sphere with the apex of the cone at the centre, where the surface of the sphere is at hydrostatic pressure. But how the sphere arrived is vital, because the residual effects are controlled by finite dissipation times. After penetration therefore, the pore pressure is not only that of the final position, but also includes increments from the cone's previous positions which may be considered as a series of stops at different time increments: these successive discrete spheres at different positions form a cylinder behind the final sphere of equal radius to the sphere. Thus, during steady penetration the cylinder dominates the pore pressure response, but that after penetration the final position and pore pressure response is predominantly that due to the equivalent sphere. The measured response, however, can only be measured at one position (without having more complex cones) and since this is at the shoulder of the cone a compromise results. Theoretical studies by Levadoux and Baligh (1980) suggest that this concept of a spherical/cylindrical model and the position of measurement of the pore pressure are reasonable. The difficulty remains however of ascribing an appropriate value for the radius of the sphere and cylinder.

There are three approaches to this: determine it theoretically; determine it experimentally or determine it empirically.

When the author's work was being carried out to assess c_v from cone dissipation tests, viz 1978, no satisfactory comprehensive theory existed.
The only experiments envisaged that could have led to the definition of the effective radius of the cylinder/sphere were penetration tests into samples containing numerous piezometers. Not impossible, but certainly daunting, particularly if a range of material types was to be explored.

The third option of empirical correlation with other measures of consolidation times was therefore selected.
Equation B4.27 implies that the function R^2T_r, ie drainage radius and time factor can be represented by a constant with in this case of value 50. It was accepted that the effective radius was dependent on the soil, but since the method was intended for use in the alluvial deposits of South Africa, and particularly of the Natal coast along which the geological history is consistent, this material dependence was not considered to be a significant problem. A large amount of laboratory test data and field experience was already available on these materials and the range of parameters for the more problematic materials was not extensive. Typically coefficients of consolidation for the soft clays are in the range of about 1 - 10m^2/year and values higher than this are in any case generally indicative that significant long term problems will not arise since a large proportion of the settlement will take place during construction. This range of c_v results in measured piezocone 50% dissipation times of 5 - 50 minutes which is acceptable for an in situ test.

The author's 1981 equation B4.27 has been used for the past 10 years in South Africa and the efficacy of the method is demonstrated in Part D, the application of piezometer cone testing in South Africa.

The research reports by Baligh et al published in 1980 generally became available during the following year, and represented a major advance in the application of theoretical methods to the dissipation of pore pressure around a cone. The work is very comprehensive and it is not possible to give any but the most general overview. They defined cone penetration as an axisymmetric two dimensional steady state problem which is essentially strain controlled. Baligh (1975) working originally from experiment and theory on the penetration of wedges developed a strain path method of analysis which modelled the measured strains with considerable veracity. This method was then used to model laboratory measured excess pore pressures and their dissipation around cones in Boston Blue Clay. They indicated that the soil immediately around a cone after penetration has stopped is being loaded as the pore pressures decrease, but further away is unloaded as pore pressures increase before subsequently decreasing; also that accurate measurements of the ambient and generated pore pressures are essential if the dissipation rate is to be reliably estimated. Errors between measured and estimated, horizontal coefficient of consolidation, c_h, occur at both high and low levels of consolidation and $\bar{U}$ = 0,5 gives the most satisfactory fit.

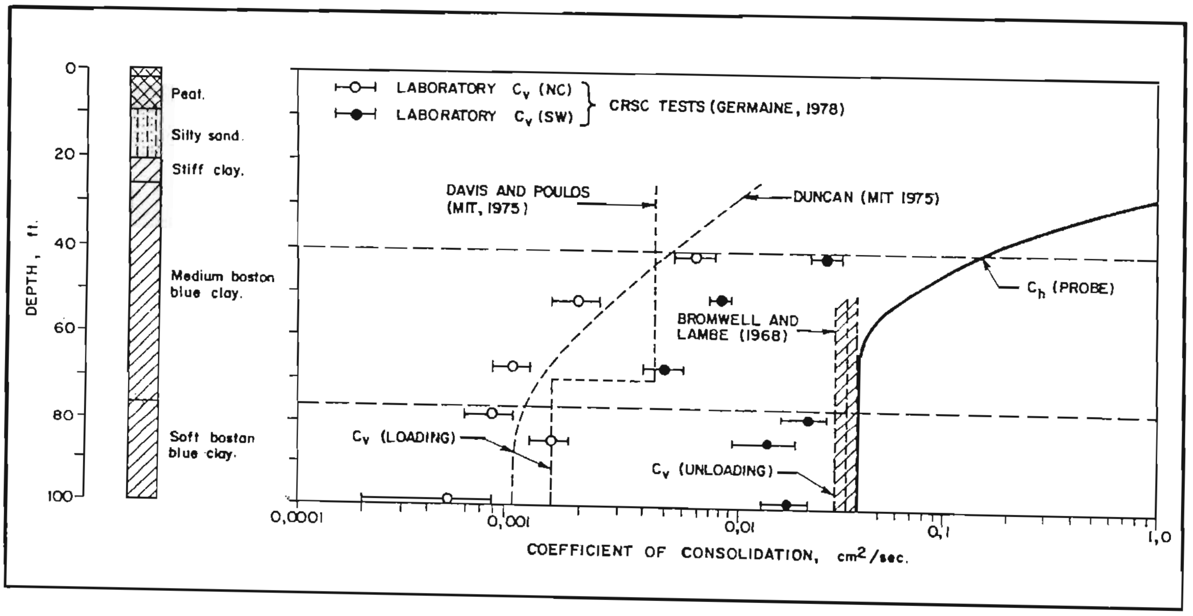


Figure B4.12 : Comparison of predicted and measured coefficients of consolidation in Boston Blue Clay (Baligh and Levadoux, 1980)

Figure B4.12 taken from their report illustrates that their method of estimating c_h (probe) gives coefficients of consolidation which agreed with c_v obtained by backfiguring from a real unloading case (an excavation). The estimated c_n were about twice to five times as large as those obtained during laboratory unloading and about twenty to forty times larger than the loading laboratory c_v and than the two sets of site c_v (loading) from back analysis of embankment records.

Figure B4.12 shows good agreement between the carefully conducted laboratory tests to obtain c_v and the backfigured site c_v, a correspondence that all too often does not appear to apply.

Baligh and Levadoux (1980) show that from the general expression

$$c_h = \frac{k_h}{m_v \gamma_w} \qquad \text{B4.28}$$

and from virgin compression (normally consolidated)

$$m_v\ (NC) = \frac{CR}{\Delta\sigma_v} \log\left(1 + \frac{\Delta\sigma_v}{\sigma_{vc}}\right) \qquad \text{B4.29}$$

and from recompression (over consolidated)

$$m_v\ (OC) = \frac{RR}{\Delta\sigma_v} \log\left(1 + \frac{\Delta\sigma_v}{\sigma_{vc}}\right) \qquad \text{B4.30}$$

Using the conventional notation and where CR and RR are the compression ratios for the normally and overconsolidated ranges and hence for small increments of effective stress the above become :

$$m_v\ (NC) = \frac{CR}{2.3\ \bar{\sigma}_{vc}} \qquad \text{B4.31}$$

$$m_v\ (OC) = \frac{RR}{2.3\ \bar{\sigma}_{vc}} \qquad \text{B4.32}$$

then $$c_h\ (NC) = \frac{RR}{CR}\ c_h\ (OC) \qquad \text{B4.33}$$

Assuming early consolidation around a cone is in a recompression mode then equation B4.33 applies :

$$c_h \text{ (probe)} = c_h \text{ (NC)}$$

$$\text{and } \sigma_{vo} = \sigma_{vc}$$

$$\text{therefore } c_h \text{ (NC)} = \frac{\text{RR (probe)}}{\text{CR}} \; c_h \text{ (probe)} \qquad \text{B4.34}$$

$$\text{or } c_v \text{ (NC)} = \frac{\text{RR (probe)}}{\text{CR}} \frac{k_v}{k_h} \; c_h \text{ (probe)} \qquad \text{B4.35}$$

The above equations result in a method of determining the required coefficient of consolidation c_v (NC) for embankment loading directly from cone dissipation tests, but it is first necessary to determine :

i) RR (probe)

ii) k_v/k_h

iii) c_h (probe) from the field dissipation tests.

i) RR (probe) : no theory existed (1980) for obtaining RR (probe) hence it can only be obtained for specific sites on the basis of measurements of the other parameters. From the tests available Baligh and Levadoux estimated that RR (probe) was in the range 0,5 x 10^2 to 2 x 10^2. RR (probe) is analogous to RR measured in a consolidometer, not having the same values but varying in the same range, which is fairly limited. Equation B4.35 is not therefore highly sensitive to the variation in RR (probe), and if the method were to be utilized in practice it would be anticipated that appropriate RR (probe) values could be readily estimated.

ii) k_v/k_h has to be measured by appropriate tests. For any significant investigation of embankment settlement this will in any case be necessary to evaluate potential two dimensional consolidation and possibly to judge the possibility of the use of sand drains. The range in the ratio for normally consolidated clays is fairly limited, say 1 : 1 to 1 : 2, and an estimate will not result in a significant error in assessment of settlement times for embankments.

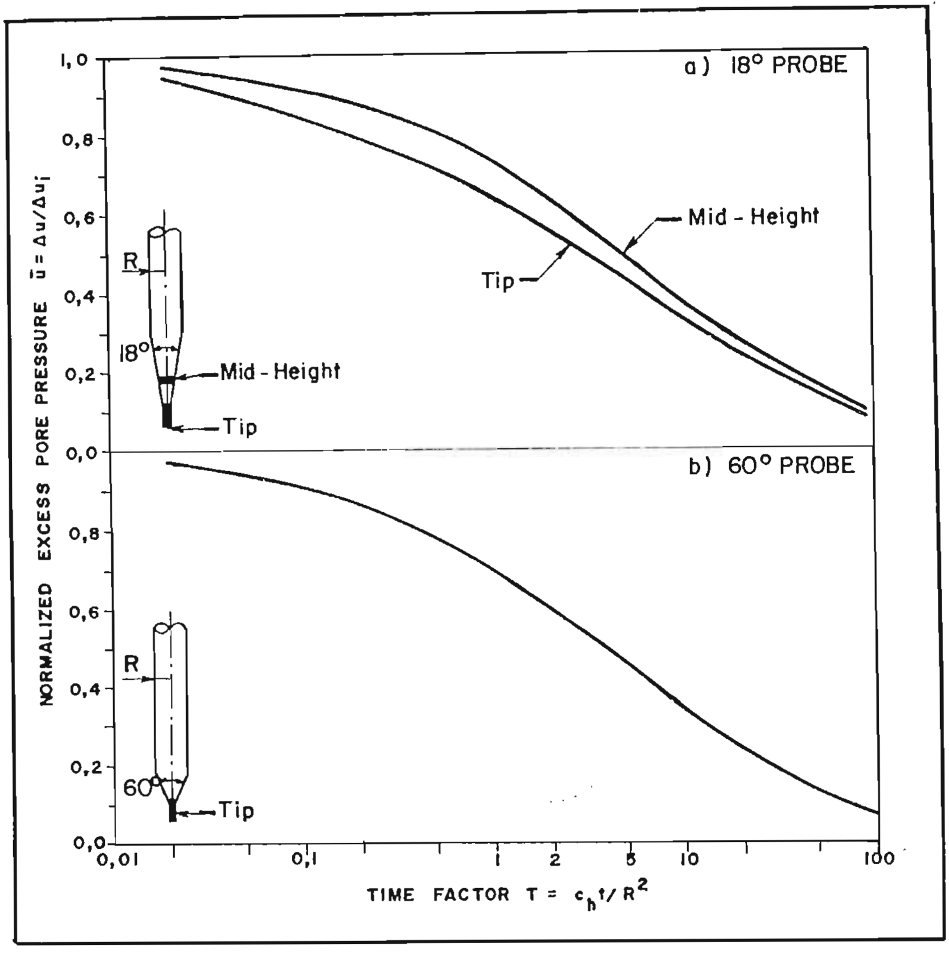


Figure B4.13 : Dissipation curves for predicting c_h (probe) (Baligh and Levadoux, 1980)

iii) c_h (probe) : the measured dissipation data can be expressed as normalized excess pore pressure against time and would generally be plotted on a log time basis. Experience shows such plots are generally similar in shape to the usual one dimensional laboratory consolidation test curves. It would however be invalid to assume that they are the same ie that they follow a simple one dimensional consolidation model, and it is necessary to develop a theoretical model for cone dissipation. The procedure is then to calculate the dissipation curves for a range of values of c_h and fit the measured data to these curves so that the best fit results. Where this occurs is the required c_h (probe) value. For various reasons (geometry etc) and because experience shows it to be so, the best fit is generally found in the mid range of pore pressure dissipation so is taken as the 50% level. Baligh and Levadoux give model curves for 18° cones with pore pressure measurements at the tip and at mid height, and for a 60° cone with pore pressure measurement at the tip ie not for the now common 60° cone with pore pressure measurement at the base of cone; Figure B4.13 illustrates these. Subsequent to 1980 solutions have been derived by various authors for the pore pressure measurement at the base of a cone.

Prior to the Baligh work analytical solutions were based on the assumption that pore pressures caused by steady state cone penetration could be modelled by cylindrical and spherical cavity expansion methods (Ladanyi, 1963). Certain assumptions have to be made for this analysis:

- the soil is isotropic
- initially subjected to isotropic state of stress
- the soil behaves as an elastic perfectly plastic material during cavity expansion.

Torstensson (1977) developed this analysis on the basis of linear uncoupled one dimensional consolidation using a finite difference approach to estimate normalized excess pore pressure dissipation curves. These are shown in Figure B4.14. A curve matching technique is used to fit actual recorded dissipation data with the theoretical curves and hence a time factor T can be obtained for any degree of consolidation and c_h calculated, since t and R (radius of cone) are known. It will be seen from Figure B4.14 that the soil type plays a role through there being different curves for a range of rigidity index (I_r) values. Baligh and Levadoux in a critical appraisal of this method commented that determining the appropriate rigidity index for undrained shear is subject to large errors, viz an order of magnitude and that the values of I_r given by Torstensson do not include a large enough range. An increased range up to say 5 000 causes an increase in the effective radius of the expanding cavity hence a slower dissipation and an increase in the derived coefficient of consolidation. No guidance is given on whether the spherical or cylindrical cavity solution should be used and these give very different results. The definition of radius is not sufficiently clear, ie if the filter element is on the cone the radius is very different from that of the shaft.

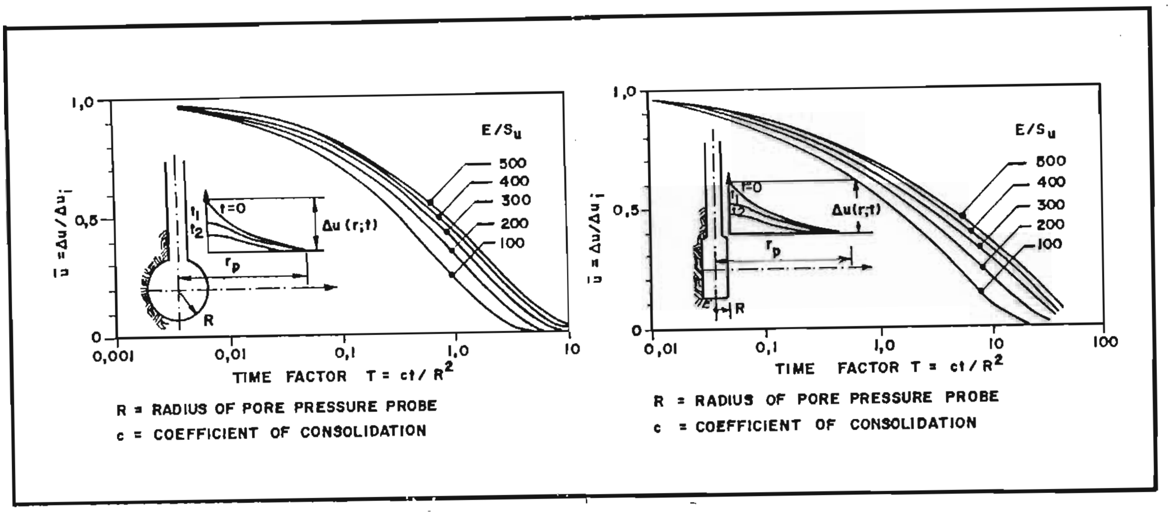


Figure B4.14 : Pore pressure dissipation around spherical and cylindrical probes (Torstensson, 1977)

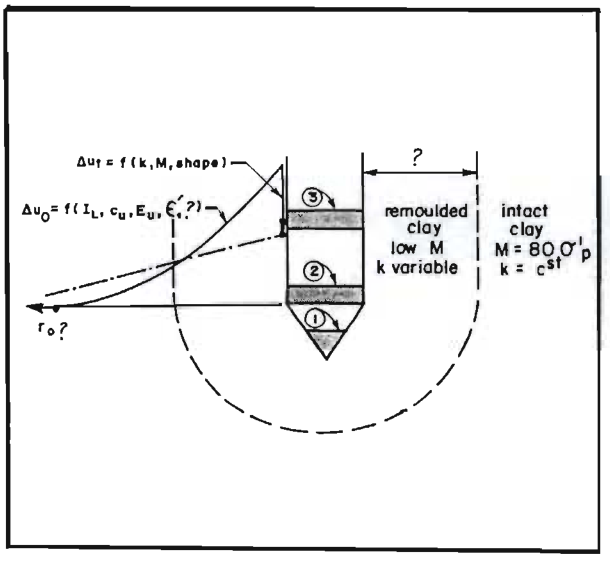
In their 1980 analysis described in the preceding pages Baligh and Levadoux attempted to overcome the shortcomings they perceived in the Torstensson approach in order to derive theoretically appropriate values for c_h (probe).

In 1982 papers by Roy, Tremblay, Tavenas and La Rochelle and by Tavenas, Leroueil and Roy, described piezocones and the parameters which can be assessed from the results of piezocone testing, including dissipation tests. They refer to the work of Torstensson (1977) and Baligh and Levadoux (1980) and whilst agreeing with many of the latters' observations, conclude that in "view of the shortcomings of the theories, an empirical approach to the interpretation of pore pressure dissipation data seems preferable". Roy et al state that their data in Canadian clays suggested that the rate of dissipation is mainly governed by the consolidation characteristics of the intact clay away from the probe. Their empirical approach matches field dissipation t_{50} times to laboratory measured preconsolidation pressures and directly measured permeabilities, using the following relationship :

$$t_{50} = \frac{T_{50}\, r_o^2 \gamma_w}{m\, k\, \sigma'_p} \qquad \text{B4.36}$$

where M the modulus of deformability of the intact clay equals $m\sigma'_p$.

It is not clear from the Roy et al ESOPT II paper what the appropriate values of t_{50} r_o and m are, but their graphical representation gives a constant value of : $T_{50}r_o^2/m$ of 1,2 x 10^{-5} m^2, Figure B4.15. This approach required independent measurements of permeability to develop the relationship, but once obtained k can be estimated from the cone measured t_{50}. They also point out that from this relationship and the stress/strain characteristics, the change of c_h resulting from changes in modulus during consolidation could be estimated.



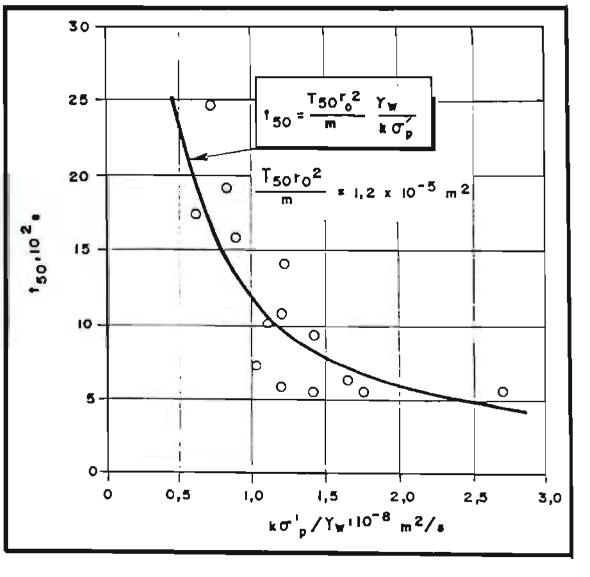


Figure B4.15 : a) Schematic features of consolidation around a cone

b) Correlation between t_{50} and $k\sigma'_p/\gamma_w$ for Champlain Sea clay (Roy et al, 1982)

Roy et al (1982) compared actual dissipation tests to the Torstensson (1977) theoretical dissipation curves and concluded that tips with different piezometer positions closely match the cylindrical or spherical models, and hence the time factors T given by Torstensson are appropriate and r_0 is the actual radius of the probe. In essence, therefore, despite expressing doubts concerning a theoretical approach, the authors support the Tortensson method.

Campanella, Robertson and Gillespie (1983) and Robertson and Campanella (1983) discuss the various solutions and tend to the view that in their experience the Torstensson cylindrical solution is appropriate. However they also suggest that it is very similar to the Baligh and Levadoux solution, a point which the latter say (in 1980) is not so, since they make a major distinction between c_h (probe) and c_h (soil) because the c_h (probe) is measuring an unloading situation, or loading above the preconsolidation pressure, whereas it would be common to require c_h (soil) for a loading case in the normally consolidated zone if the material is normally to lightly overconsolidated.

Levadoux and Baligh, (1986) and Baligh and Levadoux, (1986) in two papers which are essentially a summary of their 1980 report, describe their method again and clarify that their method gives c_h (cone) which applies for three basic situations viz :

i) foundation unloading ie excavation
ii) foundation loading in the overconsolidated range
iii) foundation loading in the normally consolidated range.

For i) and ii) above ie unloading and overconsolidated loading

$$c_h \text{ (cone)} = c_h\text{(field)} \qquad \text{B4.37}$$

$$c_h \text{ (field)} = \frac{k_v}{k_h} c_h \text{ (cone)} \qquad \text{B4.38}$$

iii) normally consolidated loading

$$c_v \text{ (field)} = \frac{RR\text{(cone)}}{CR} \frac{k_v}{k_h} C_h \text{ (cone)} \qquad \text{B4.39}$$

Thus Baligh and Levadoux (1986) confirm their 1980 view that a major distinction should be made between loading in the normally consolidated and overconsolidated ranges since RR/CR is typically small, for example about 0,01/0,30. Many so called normally consolidated soft clays are in fact lightly overconsolidated and construction of a typical embankment may comprise both loading in the overconsolidated range ie from initial effective stress up to the preconsolidation pressure, and in the normally consolidated range beyond this pressure.

In their 1986 papers Levadoux and Baligh in discussing the methods of estimation of coefficients of consolidation refer to two empirical methods (Jones and van Zyl, 1981 and Tavenas et al 1982) and to two rational methods (Torstensson, 1977) and their own and imply that at that stage these were the only known methods.

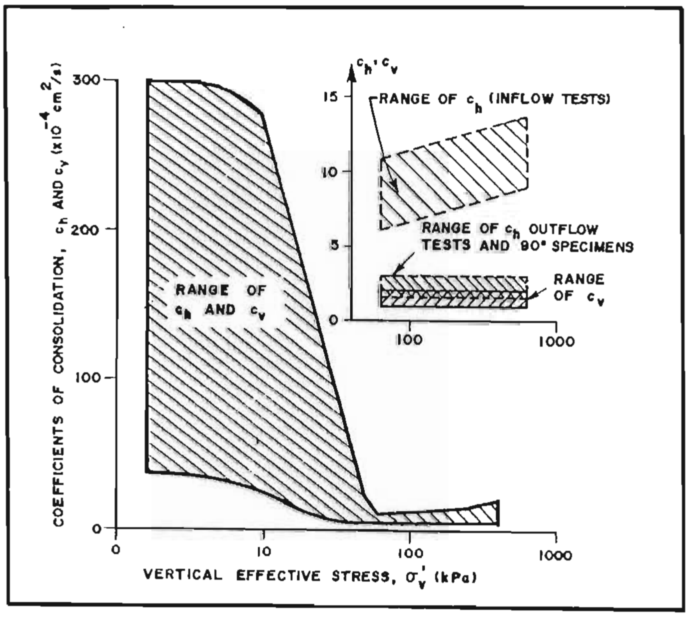


Figure B4.16 : Values of c_h from laboratory tests (Sills et al, 1988)

In 1988, at ISOPT I, Sills et al described the consolidation of an embankment in Rio de Janiero and state that they used the Baligh and Levadoux (1980) method and compared the results of their piezocone data with laboratory and field consolidation data, the latter backfigured from field settlement measurements. Their data illustrates all too well the designer's problem, Figure B4.16 showing the spread of laboratory coefficients of consolidation from 3 to 300 x 10^{-4} cm^2/s.

There is very poor agreement between their piezocone derived c_h, the laboratory values and the backfigured coefficients, which may in part be because they do not take account of Baligh and Levadoux differentiation of stress levels for the estimation of c_v ie overconsolidated and normally consolidated ranges, and the data indicates that the embankment stressed the subsoil through both ranges. Using only the recompression approach they obtain c_h (cone) values of 90 -250 x 10^{-4} cm^2/sec with an average value of 133 x 10^{-4} cm^2/sec.

The measured time for 50% dissipation is about 10 minutes so the Jones and van Zyl (1981) method gives a c_v of about 5 x 10^{-4} cm^2/sec. This compares with the low end of the laboratory range given in Figure B4.16 (for an effective stress of about 25 kPa) and the Sills et al 133 x 10^{-4} cm^2/sec with the higher end of the range. The backfigured field measurements gave c_v values of about 12 x 10^{-4} cm^2/sec. If the normally consolidated Baligh and Levadoux corrections had been used, then values of c_v would have been very much lower, probably about 3 - 5 x 10^{-4} cm^2/sec. This suggests that the emphasis given by Baligh and Levadoux on the distinction between consolidation rates in the normally and overconsolidated ranges would, if applied in this case, have given much closer agreement between measured and predicted coefficients of consolidation.

In ISOPT I, 1988, Houlsby and Teh introduced another approach to the analysis of dissipation tests with piezocones. They estimated generated pore pressures using Henkel, (1959) :

$$\Delta u = \Delta \sigma_{oct} + \alpha \, \Delta \tau_{oct} \qquad \text{B4.40}$$

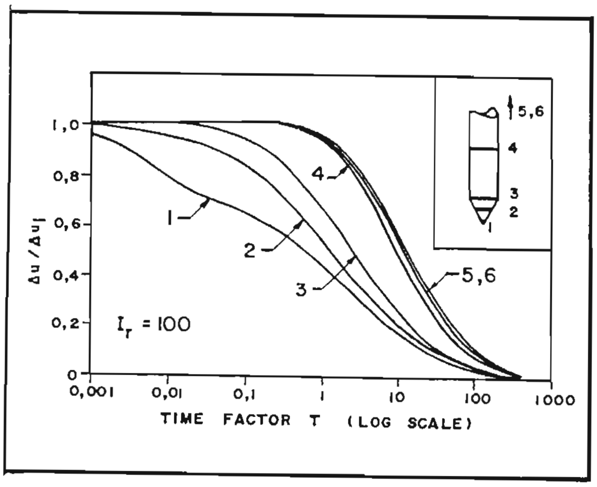
Using uncoupled Terzaghi-Rendulic consolidation theory solved by a finite difference method, they showed that the shape of the pore pressure dissipation curves is influenced by the rigidity index, I_r, since the initial pore pressure distribution is determined by I_r : this is because the excess pressures develop primarily in the plastically deforming zone the radius of which is a function of $\sqrt{I_r}$. They then showed that the dissipation curves can be unified in the range I_r of 50 to 500 if a modified factor T^* is defined in terms of I_r :

$$T^* = \frac{c_h\ t}{R^2\sqrt{I_r}} \qquad \text{B4.41}$$

Figures B4.17 a) and b) are taken from their paper as is Table B4.3 which gives T^* for the piezometer at a number of different positions on the penetrometer.

Table B4.3 : Modified time factors T^* from consolidation analysis

Degree of Consolidation	Location				
	Tip	Cone Face	Shoulder	5 rad. above shoulder	10 rad. above shoulder
20%	0.001	0.014	0.038	0.294	0.378
30%	0.006	0.032	0.078	0.503	0.662
40%	0.027	0.063	0.142	0.756	0.995
50%	0.069	0.118	0.245	1.11	1.46
60%	0.154	0.226	0.439	1.65	2.14
70%	0.345	0.463	0.804	2.43	3.24
80%	0.829	1.04	1.60	4.10	5.24



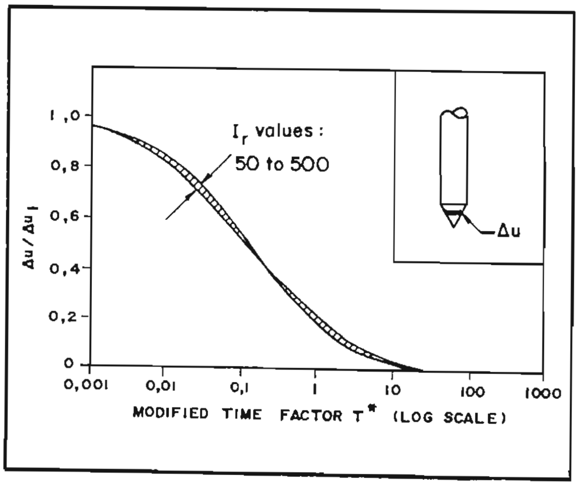


Figure B4.17 :a) Excess pore pressure dissipation curves for different filter positions

b) Excess pore pressure dissipation curves for modified time factor, T^*.

(Houlsby and Teh, 1988)

They checked their time factors against those given by previous analytical studies, Torstensson (1977) and Randolph and Wroth (1979), and concluded that similar values are obtained at the shoulder if account is taken of I_r. Whilst this approach is encouraging it requires calibration against field and laboratory data before the method can be adopted for practical use. For example if applied to the Sills et al data it results in even larger c_h piezocone values and therefore poorer agreement with the field data. It is not suggested that the Houlsby and Teh approach is incorrect, but demonstrates the potential gulf between measuring small scale c_v values by piezocone or laboratory tests and successfully applying these to field scale settlement predictions.

B4.5 Soils Identification from Cone Penetration Testing

B4.5.1 Mechanical friction sleeve soils identification

Soils identification from cone penetration testing began with Begemann's (1953) friction sleeve cone. The Begemann method continues to be widely used throughout the world with measurements from Begemann mechanical friction sleeves and from electric cones with friction sleeves. The original Begemann data base has been extended to cover a wider variety of soils and in many countries local correlations between friction ratio and soil type have been derived. This line of development continues, and at ISOPT I (1988) a number of papers and discussions dealt solely with this aspect.

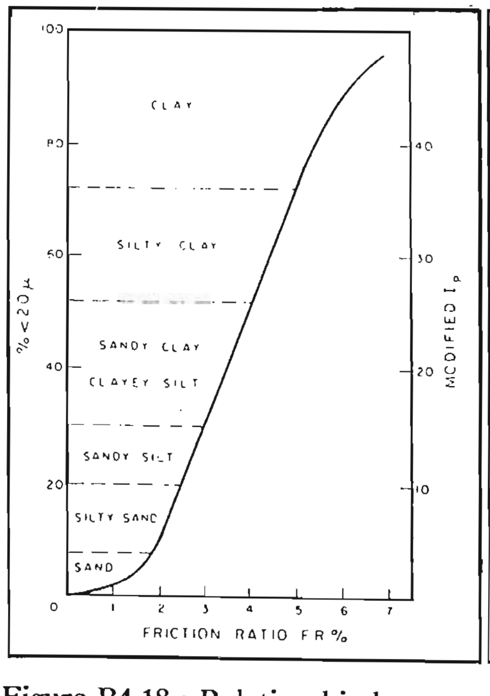
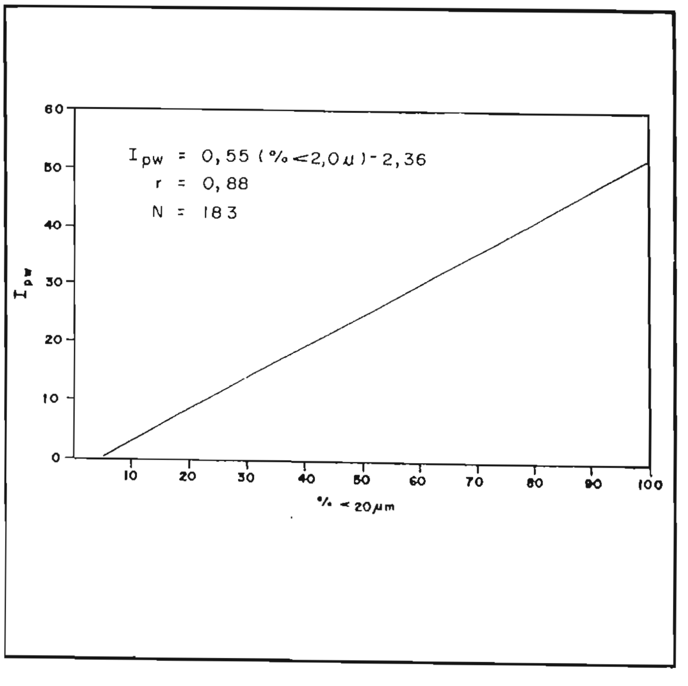


Figure B4.18 : Relationship between friction ratio and soil description



B4.19 : Plasticity index against percentage <20μm for Natal alluvial clays

The Begemann friction sleeve was introduced to South Africa by the author in 1971. It was found to be of limited value in soft alluvial deposits, because the conventional mechanical cone load measuring systems were too insensitive at the low pressures experienced in soft materials. To overcome this problem the author (Jones 1975) used the Begemann mechanical friction sleeve with an enhanced load measuring system (the equipment is described in Part C). Friction ratios measured using the improved system were compared with borehole soil samples to build a data base from which Figure B4.18 was drawn up. The figure includes both the percentage of material smaller than 20μm and the modified plasticity index I_{pw} as well as the material description. (The modified I_{pw} is simply the standard I_p multiplied by the percentage passing the 0,425 mm sieve ie it is the I_p of the whole sample). The laboratory testing enabled a correlation to be made of the modified plasticity index with the percentage 20μm - Figure B4.19. This is confirmed from time to time as further information becomes available. The 183 data points are not shown in the figure but these gave a correlation coefficient of 0,88 for the linear regression line given by the following equation:

$$I_{pw} = 0,55\ (\% < 20\mu m) - 2,36 \qquad \text{B4.42}$$

and simplified to :

$$I_{pw} = 0,5\ (\% < 20\mu m) \qquad \text{B4.43}$$

The paper concluded with the somewhat optimistic comment "a preliminary assessment of the time settlement characteristics of the subsoil becomes possible" and suggested that this could be achieved by establishing a relationship between coefficients of consolidation, c_v, and the plasticity index of the whole sample. Hardly a primary correlation, but despite all the caveats a useful secondary correlation was proposed and this is given in Table B4.4 taken from TRH10, NITRR (1987).

Table B4.4 : Subsoil description and coefficient of consolidation

Subsoil Description	Permeability	Coefficient of consolidation c_v m^2/year
Intact clays	very low	0,1 - 1,0
Fissured clay, alluvial silty clay	low	1,0 - 10
Clayey sand, sandy	medium	10 - 100
Silty sands	high	100 - 1000
Sands	very high	1000>

B4.5.2 Piezometer cone soils identification

The development of piezocones since the late 1970's created a new dimension for soils identification. Almost immediately the potential of the piezocone for identification was recognized. In 1980 Baligh, Vivatrat and Ladd discussed soils identification using the results of a standard cone and of a Wissa type probe (piezometer only) at adjacent positions. They demonstrated the potential of using a pore pressure ratio u_t/q_c (where u_t is the developed total pore pressure and q_c the cone pressure at the same depth). Since they were working primarily in one deposit, Boston Blue Clay, their interpretation was aimed primarily at assessing the overconsolidation ratio. They observed that the pore pressures measured during penetration followed a similar pattern to the cone pressures, except that in the layered soil system the cone pressures showed more distinct jumps between the layers than the pore pressures. They argued that in lightly overconsolidated clays undrained shearing results in decreased effective stresses which implies increased pore pressures not only to resist the penetration compressive stresses but also the large shear stresses. Conversely in heavily overconsolidated clays either smaller or even negative pore pressures will be developed due to shear. The compressive stress pore pressures may or may not compensate for the negative shear stress induced pore pressure and will be related to the degree of overconsolidation.

They summarized by stating that "The ratio u_t/q_c should provide a new promising method for soil identification. However, more data are needed to establish general correlations". In short, therefore, because of the limitations of their equipment and the fact that their work was confined to Boston Blue Clay and to Atchafalaya Clay, neither having much variability in soil type, their observations although valuable, particularly with regard to overconsolidation ratios, were restricted.

In 1981 (Stockholm and St Louis) the author published two papers which inter alia discussed soils identification. It was suggested that the Baligh, Vivatrat and Ladd (1980) usage of u_t/q_c where u_t is the total pore pressure is unsatisfactory and that the excess pore pressure, u_e, should be used where :

$$u_e = u_t - u_o \qquad \text{B4.44}$$

and u_o is the ambient pore pressure which would usually (no flow situation) be the hydrostatic pressure. The author suggested the use of a normalized subsoil index :

$$\frac{u_t - u_o}{u_o} \Big/ \frac{q_c - \sigma_{vo}}{\sigma_{vo}} \qquad \text{B4.45}$$

The author noted that u_e could be negative, and hence the index negative, indicating dilatant materials, and also that the index was "a measure of the pore pressure parameter at failure, A_f".

These papers coined the use of the description CUPT which subsequently succumbed to the arguably more rational acronym, CPTU.

In 1982 (ESOPT II) and 1983 (Paris) the author presented his Soils Identification Chart -Figure B4.20 based on u_e and (q_c - σ_{vo}) ie these were not normalized. The move away from normalized parameters was made with some reluctance, but there are advantages, viz, close to the water table and to the ground surface the normalized pore pressure and cone pressure parameters may become very large and cannot readily be contained within an arithmetic scale; it is useful to use the actual pore pressures rather than a normalized parameter since this results in developing a feel for the values; the chart is simpler to use. The normalized and unnormalized versions are in any case similar, since the normalising parameters u_o and σ_{vo} are directly related and of similar values.

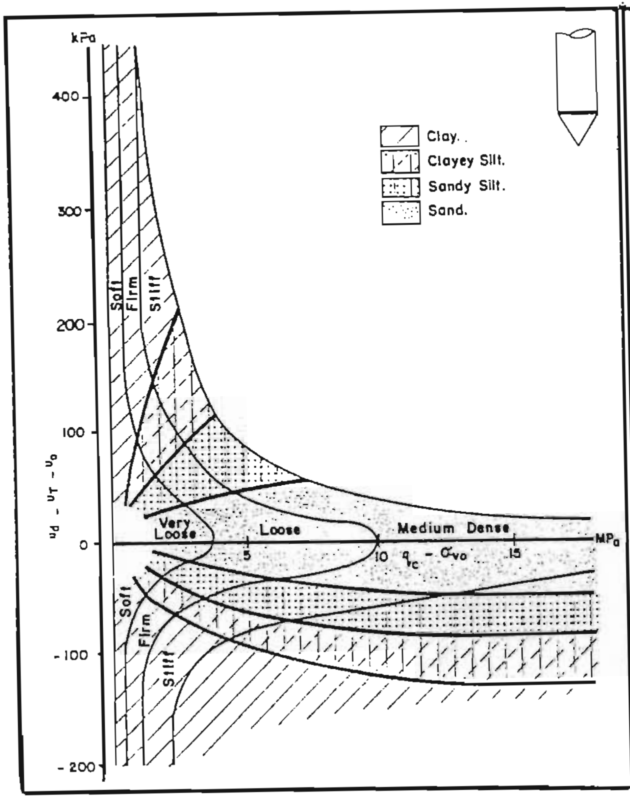


Figure B4.20 : Soils Identification Chart (Jones and Rust, 1982)

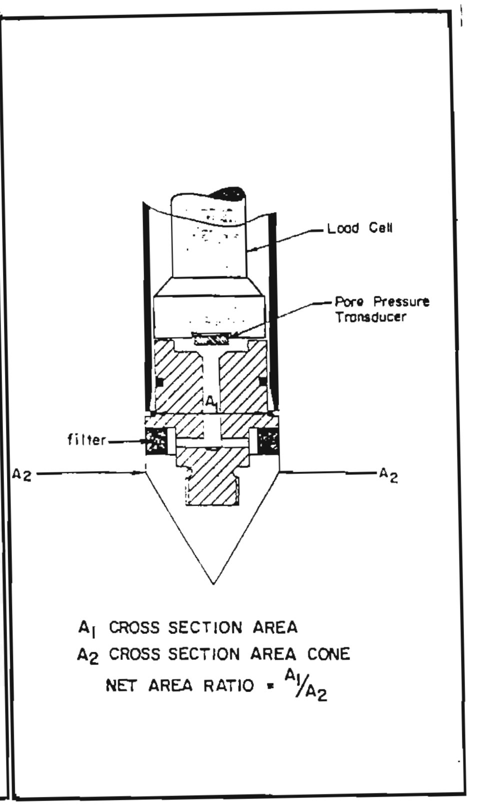


Figure B4.21 : Piezocone net area correction

The cone pressure, q_c, should be corrected using the unequal end area correction suggested by Campanella et al (1982) and illustrated in Figure B4.21 and given in the following :

$$q_T = q_c + u\,(1 - a) \qquad \text{B4.46}$$

where q_T = corrected cone pressure

q_c = measured cone pressure

u = measured pore pressure

a = net area ratio (see Figure B4.21)

The boundaries between the soil types on the soils identification chart, Figure B4. 20, are slightly curved since this is what the data appeared to show. However the boundaries should be seen as transition zones and these could be represented as straight lines of constant values of :

$$B_q = u_e/(q_c - \sigma_{vo}) \qquad \text{B4.47}$$

where B_q is defined as the CPTU pore pressure parameter.
The boundaries for the different soil types are given in Table B4.5.

Table B4.5 : B_q values for different soil types

Soil Type	B_q
Sand	0 - 0,01
Silty sand	0,01 - 0,03
Silt	0,03 - 0,09
Clay	0,09 - 0,5

In practice it is found that at the low cone pressures found in soft and very soft clays (q_c - σ_{vo}) is small and the chart becomes difficult to use. A further version (unpublished) is given in Figure B4.22 with the cone pressure axis at a logarithmic scale, hence finer discrimination is possible at low cone pressures, and the soil type boundaries are represented by lines of constant B_q.

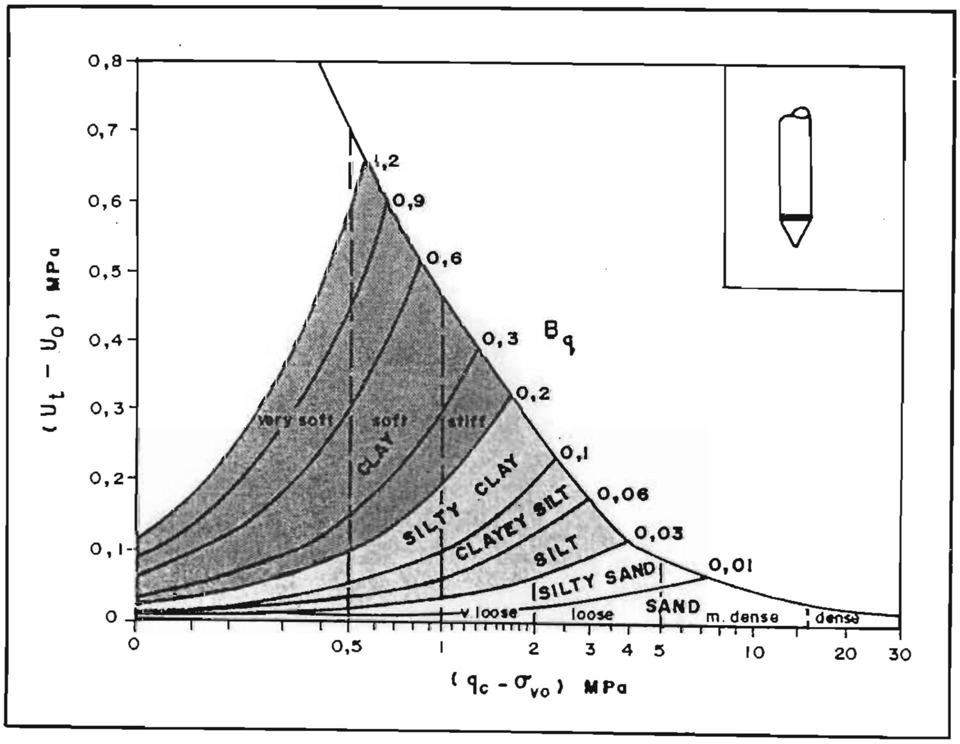


Figure B4.22 : Soils Identification Chart (Jones, 1992)

It is useful to examine the significance of the cone pore pressure parameter, B_q, not only as a convenient means of defining the boundaries between soil types. Peignaud (1979) using Vesic (1972) cavity expansion theory deduced that :

$$\Delta u = (1{,}73A_f - 0{,}57 + \ln I_r)\ c_u \qquad \text{B4.48}$$

where A_f = Skempton's (1954) pore pressure parameter at failure.
Hence at failure in a normally consolidated clay since A_f = 0,95 then :

$$\Delta u = (1{,}07 + \ln I_r)\ c_u \qquad \text{B4.49}$$

Then for typical values of I_r from say 50 to 500 :

$$\Delta u = 5\ c_u \text{ to } 7{,}3\ c_u$$

and from $\quad q_c = N_{kt}c_u + \sigma_{vo}$

$$\frac{\Delta u}{q_c - \sigma_{vo}} = (5 \text{ to } 7.3)\ /\ N_{kt} \qquad \text{B4.50}$$

In soft clays, therefore, if N_{kt} is in the range of 9 to 15 then B_q has a value of about 0,3 to 0,8. The measured values of u_t are dependent on the cone geometry and in particular on the filter position, and any soils identification chart should emphasize this by including a diagram of the cone showing the filter position.

Tavenas et al (1982) developed the idea of a pore pressure factor $N_{\Delta u}$ analogous to N_{kt} the cone factor so that :

$$\Delta u = N_{\Delta u}\ c_u \qquad \text{B4.51}$$

Using pore pressures measured at the base of the cone and c_v measured by field vane tests, they found in Canadian clays that for the range of liquidity indices, I_L, shown, then :

for $0,8 < I_L < 2,0$

$$N_{\Delta u} = 7,9 \pm 0,7 \qquad \text{B4.52}$$

for $I_L > 2,0$

$$N_{\Delta u} = 11,7 \pm 2,0 \qquad \text{B4.53}$$

Tavenas et al concluded that since in soft clays the generated pore pressures are relatively high in the measuring range of the equipment, and, conversely, the cone pressures are low, then it would be preferable to assess the undrained shear strength from pore pressure measurements, rather than from cone pressure measurements, provided of course that the appropriate values of the pore pressure factor $N_{\Delta u}$ are known.

The preceding discussion regarding pore pressures generated during cone penetration has considered only normally consolidated materials. It is generally agreed that the measured pore pressures are not only a function of the cone geometry but also of the soil stress history, sensitivity, rigidity index, fissuring and cementation. The use of identification charts or indices based on pore pressure measurements is therefore subject to the constraints that these factors are usually not known except from pre-existing knowledge of the soil. The author's chart is for normally consolidated, relatively insensitive soils which show no evidence of fissuring or cementation.

From available data it would appear that the excess pore pressure is strongly dependent on OCR - Figure B4.23 and u_e can be almost halved as the OCR changes from 1 to 4. However this change, although of considerable significance if equation B4.51 is to be used for estimating shear strength, does not greatly influence the sector on the author's soil identification chart into which the soil fits, because, for example, the range of B_q

values from the sand/silty sand boundary to the silty clay/clay boundary covers practically one order of magnitude.

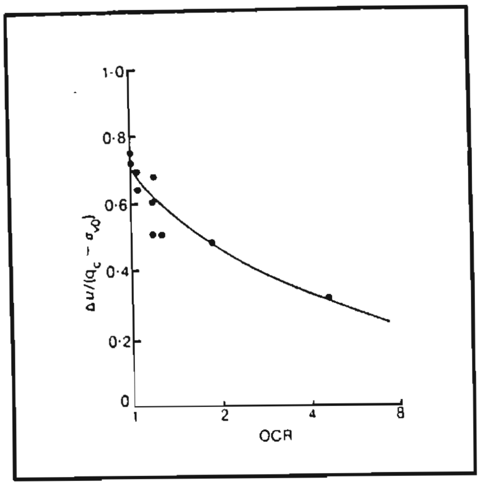


Figure B4.23 : Variation of piezocone pore pressure ratio with OCR at Onsoy (Wroth, 1988)

Nevertheless it must be concluded that soils identification using a chart based on cone pressures and excess pore pressures cannot give an unambiguous description of the soil. Further information is required some of which is available from piezocone testing . The first is that an estimate of the OCR can be obtained in cohesive materials from a comparison of the deduced s_u from q_c compared with the expected s_u at the same depth using Skempton's (1954) equation B4.52 if the plasticity is known :

$$s_u/\sigma'_{vo} = 0{,}11 + 0{,}0037\ I_p \qquad \text{B4.52}$$

The measured and expected s_u/σ'_{vo} can be compared and using Figure B4.24 an OCR can be estimated.

The second data set available is from the dissipation times, which are a direct reflection of the permeability and hence of the nature of the soil.

Indeed, the soils identification chart could be developed for cohesive soils to give an estimate of overconsolidation ratio using the above approach, provided the plasticity index is known. The chart may also be improved by including a measure of the dissipation times. However whilst it is interesting to consider such developments in soils identification from piezocone testing, the present method developed by the author is sufficient for all practical purposes for normally consolidated soils. Further refinements would simply be a matter of adding data to the chart which would not modify the verbal description of the soil but would quantify some aspect of it, eg permeability. To a large extent a similar argument arises for overconsolidation except that this aspect is so important in the case of the prediction of embankment performance that any possibility of assessing overconsolidation from piezocone results should be pursued.

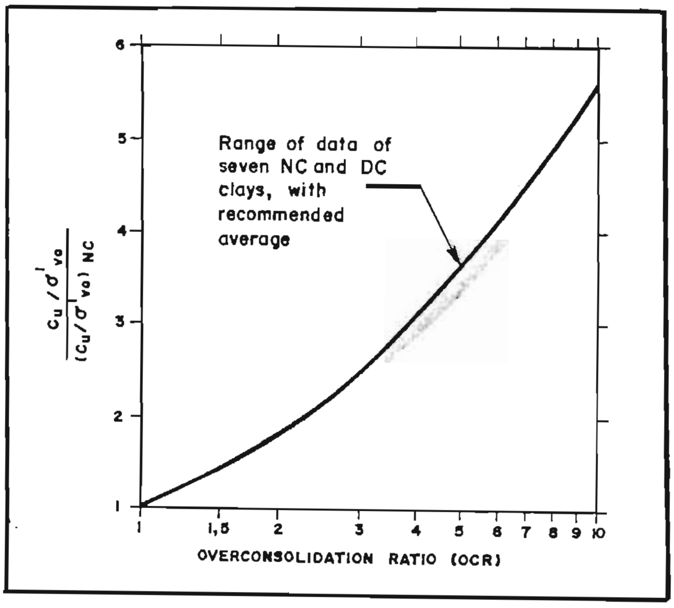


Figure B4.24 : Basis for estimation of OCR (Schmertmann, 1978)

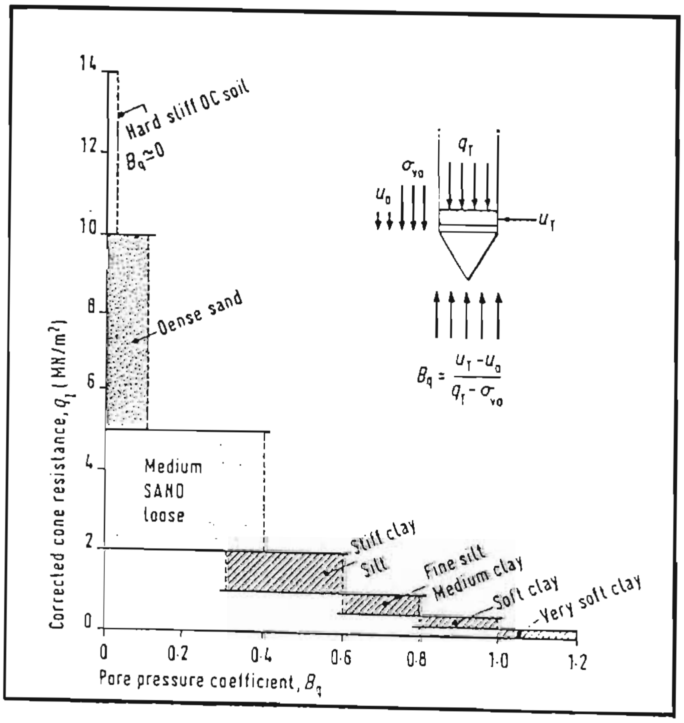


Figure B4.25 : Classification chart based on cone resistance and pore pressure ratio (Senneset and Janbu, 1985)

In 1984 Senneset and Janbu produced a soils identification chart, Figure B4.25. It is essentially in a similar form to the author's 1982 chart except that it plots B_q, against q_t. However comparative checks between the two seem to show considerable differences. The differences can be seen by comparing the B_q values given by the author's chart at boundaries between material types with those on the Senneset and Janbu chart. The comparison is given in Table B4.6.

Table B4.6 : Comparison of B_q values at soil type boundaries

Soil Type	B_q	
	Jones and Rust	Senneset and Janbu
Dense sand	0 - 0,01	0 - 0,1
Medium sand	0 - 0,01	0 - 0,4
Silty sand	0,01 - 0,03	
Stiff clay; silt	0,09 - 0,3	0,3 - 0,6
Medium clay; fine silt		0,06 - 0,8
Soft clay	0,3 - 0,6	0,8 - 1,0
Very soft clay	0,6 - 0,9	1,0 - 1,2

It is clear from the Senneset and Janbu chart that their definition of the consistency boundaries (loose, medium dense, dense and very dense) for the sands is different from those in general use, but this alone does not account for the differences where they would appear to measure moderately high pore pressures in medium dense sands.

The consistency boundaries for their cohesive soils are also different from those generally used, but again this would not appear to account for the differences between the B_q values given by the two charts.

The author's chart was drawn up on the basis of considerable comparative field and laboratory testing and in that sense cannot be incorrect for the particular materials and cone system. Whether it can be successfully applied elsewhere remains an open question and no doubt the same applies for the Senneset and Janbu chart.

In 1986 Robertson et al proposed a soils identification chart in which they utilized cone pressures, pore pressure ratios and friction ratios from piezocones which measured all three. This is shown in Figure B4.26.

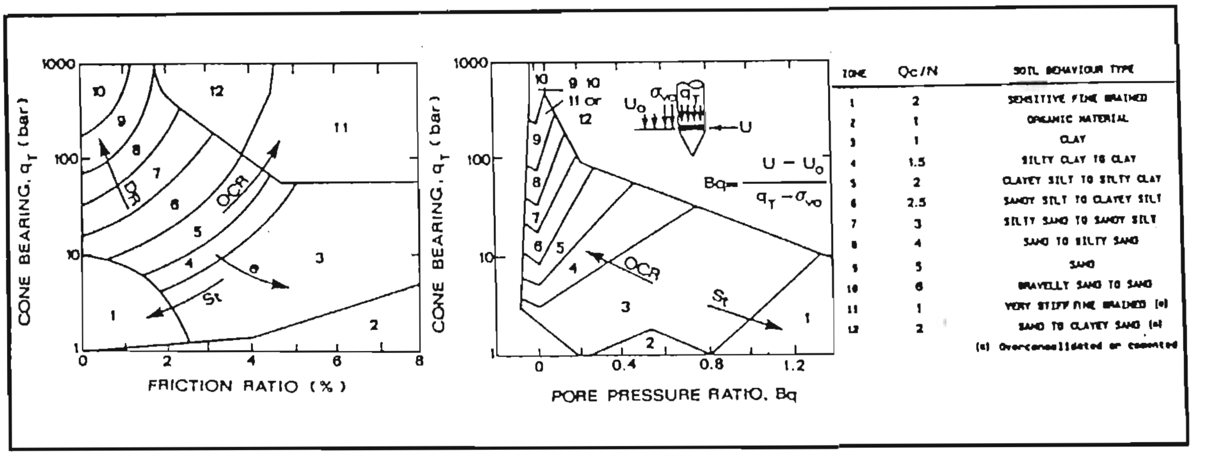


Figure B4.26 : Soil behaviour type chart from CPTU data (Robertson et al, 1986)

The pore pressure ratio chart is similar to that of Senneset and Janbu in that it uses the same parameters ie q_t and B_q although the former is plotted on a logarithmic scale. There seems to be no advantage in this, particularly since it covers the range of q_t up to 100MPa when for most soils where piezocone testing is appropriate, q_t values seldom exceed 20MPa. Nevertheless the Robertson et al pore pressure chart has a considerable advantage over the Senneset and Janbu chart in that the boundaries between material types are less rigidly defined in terms of either of the plotted parameters so there are more subtle overlaps due to combinations of these parameters. The friction ratio chart is similar to the pore pressure ratio chart and Robertson et al suggest both should be used to define the soil type and where possible dissipation rates (t_{50}) should also be used to further assist in the categorisation. The Robertson et al chart attempts to indicate both overconsolidation and sensitivity of the clays and although these are useful indications there does not appear to be substantiation for these other than they are expected trends.

A further aspect of soils identification is the minimum thickness of layers which can be detected. Experience shows that with the filter element immediately above the base of the cone, and with the element thickness about 4 - 5 mm, then layers of similar thickness can readily be detected. However this does not mean that their soil types can be identified because penetration through such a layer takes only about 0,2 seconds and in this time a representative pore pressure is not measured: also since the cone height is 36 mm it cannot measure a representative cone pressure in any layer thinner than this. Nevertheless the cone pressure and particularly the pore pressure fluctuations very clearly indicate thin layers of less than 10 mm, if the material is significantly different from that surrounding it.

In summary soils identification from piezocones is well established and is demonstrated in the author's 1982 chart and the Robertson et al, 1986, charts. Both authors emphasize the need for local calibrations and the use of standardised piezocones.

Neither chart can satisfactorily deal with differentiating between normally consolidated and overconsolidated soils although the Robertson et al chart does give some indication of this, as it also does for sensitivity. For the purpose of investigation of Southern African alluvial soils the author's chart must be favoured since it was evolved from data taken from these soils. It must be noted that charts based on pore pressure measurements register the behaviour of the soil rather than the actual grain size and some anomalous results may be expected in soils with unusual structures or fabrics.

B5 GEOTECHNICAL DESIGN OF EMBANKMENTS

In the context of this thesis, design is concerned primarily with the prediction of settlements and times for settlements of embankments on alluvial deposits using cone penetration testing.
This design, however, is no different using cone penetration testing to obtain the necessary soil parameters, to design using similar parameters obtained from other forms of testing.

The classic one dimensional settlement calculation method is due to Terzaghi (1943) and is usually represented as :

$$\delta = m_v \, \Delta\sigma_v \, \Delta h \qquad \text{B5.1}$$

where δ = vertical strain in layer

$\Delta\sigma_v$ = vertical stress increment in layer

Δh = layer thickness

m_v = coefficient of volume compressibility in the appropriate stress range

The total settlement ρ_t is then given by :

$$\rho_t = \rho_{od} = \Sigma \, m_v \, \Delta\sigma_v \, \Delta h \qquad \text{B5.2}$$

Skempton and Bjerrum (1957) suggested a method which would take account of the magnitude of pore pressures actually generated in the subsoil under a foundation in a realistic three dimensional situation and these depend on both the geometry of the situation and the degree of overconsolidation of the soil. This method then takes separate account of the immediate settlement which is due to undrained pseudo elastic compression, ρ_u, and is expressed as :

$$\rho_t = \rho_u + \mu\rho_{od} \qquad \text{B5.3}$$

Figure B5.1 gives values of μ.

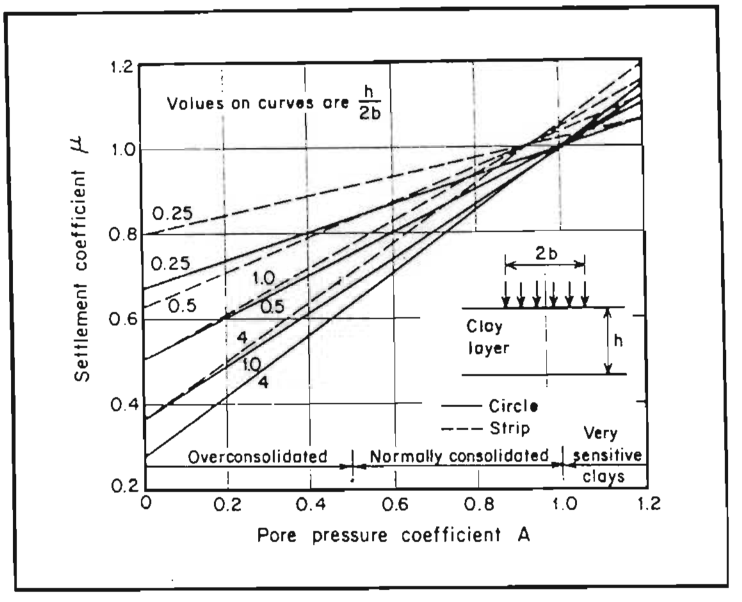


Figure B5.1 : Settlement coefficient versus pore pressure coefficient (Skempton and Bjerrum, 1957)

This approach appears to be inherently more satisfactory than the Terzaghi method since it separates the settlement into the observed separate components of immediate and consolidation settlement and in practice it is important to separate these since they broadly represent the during and after construction components. In the case of embankments the former may be relatively unimportant and the latter is the only settlement which gives rise for concern. The Skempton and Bjerrum method, however, requires a soil parameter to model the undrained compression in addition to the m_v usually obtained from laboratory consolidation tests. This may be achieved by undrained triaxial tests at the appropriate stress conditions, although in practice reliable results are difficult to obtain.

Over the years a number of refinements have been developed of the Terzaghi consolidation settlement equation B5.1. One such refinement is that since m_v in the overconsolidated range and normally consolidated range are very different, then if the loading stresses the soil in both ranges it must be taken into account in the calculation.

This results in the expression :

$$\rho_t = \sum \left(\frac{C_r}{1 + e_o} \Delta h \log \frac{\sigma'_{vc}}{\sigma'_{vo}} + \frac{C_c}{1 + e_c} \Delta h \log \frac{\sigma'_{vf}}{\sigma'_{vc}} \right) \qquad \text{B5.4}$$

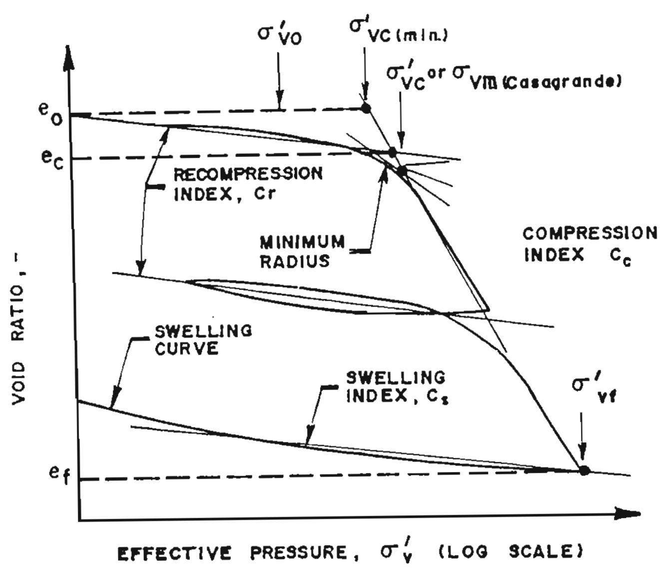


Figure B5.2 : Terminology used for oedometer tests

Figure B5.2 indicates the conventional definition of the terms in the above expression. A second refinement has been the introduction of a method of calculating secondary settlements, ρ_s. The most common method is given in the following equation :

$$\rho_s = \sum \left(\frac{C_\alpha}{1 + e_o} \Delta h \right) \log \frac{t_s}{t_p} \qquad \text{B5.5}$$

where C_α is the coefficient of secondary compression viz the change in void ratio per unit change in logarithm of time after the end of primary consolidation, t_p. In the above expression t_s is the time to which the secondary compression is calculated.

C_α would usually be obtained from consolidation testing although with the usual 24 hour load cycles this may not be possible and much longer laboratory testing times may be necessary. Alternatively C_α may be obtained from local experience or from published information. For example Mesri and Godlewski (1977) showed that there is a unique relationship between C_α and C_c for any soil and that C_α/C_c lies in the range 0,025 to 0,10; the higher values occur in organic soils.

Davis and Poulos (1968) developed methods of settlement prediction based on elastic analyses in which moduli values, E, and Poisson's ratios, υ, are required for the undrained and drained states to model the equivalent of the immediate and total settlements given by the Skempton - Bjerrum method (equation B5.3). The elastic settlement equations are :

$$\rho_u = \sum \frac{1}{E_u} \left(\Delta\sigma_z - \nu_u \left(\Delta\sigma_x + \Delta\sigma_y\right)\right) \Delta z \qquad \text{B5.6}$$

$$\rho_t = \sum \frac{1}{E'} \left(\Delta\sigma_z - \nu'\left(\Delta\sigma_x + \Delta\sigma_y\right)\right) \Delta z \qquad \text{B5.7}$$

An important condition of the above is that elastic soils are defined as those in which the settlement is independent of the stress path viz the total settlement is not dependent on rate of loading and will be the same whether the embankment is built in many stages or one stage (provided no overstressing occurs with the latter). In other words the final components of immediate and consolidation remain the same whatever stress paths occur. It is emphasized that equation B5.7 represents the total settlement and therefore includes the component due to the immediate settlement given by equation B5.6, ie equation B5.7 is equivalent to equation B5.2 the Terzaghi settlement derived from consolidometer test results.

For homogeneous elastic soils the stress-strain drained behaviour is defined by Young's Moduli, E', and Poisson's ratio, υ' and a consolidometer test gives :

$$m_v = \frac{1}{E'} \left(1 - \frac{2\nu'^2}{1 - \nu'}\right) \qquad \text{B5.8}$$

Note that m_v and $1/E'$ are not equal even for the relatively simple case of elastic homogeneous soils and this distinction is important and often overlooked when the derivation of compressibility parameters from in situ tests is discussed.

For homogeneous elastic soils the undrained and drained elastic moduli can be related through the shear modulus G viz

$$2G = \frac{E_u}{1 + \nu_u} = \frac{E'}{1 + \nu'} \qquad \text{B5.9}$$

and for undrained clays Poisson's Ratio is 0,5
therefore :

$$E_u = 3 \frac{E'}{2(1 + \nu')} \qquad \text{B5.10}$$

Davis and Poulos (1968) estimated the relative value of immediate undrained settlement to total settlement for a range of drained Poisson's ratios and geometry - Figure B5.4. They also estimated the error involved in using the conventional one dimensional approach instead of a three dimensional analysis - Figure B5.3. From Figure B5.4 for road embankments where h/a is about 0,5 - for a typical single carriageway main road over alluvial deposits on the Natal coast - then for soft clays if υ' is 0,4, the proportion of immediate undrained settlement may be about 0,35 of the total. It is also noteworthy that this proportion is highly dependent on the ratio of embankment width to subsoil depth. Similarly, from Figure B5.3 for the same embankment, the calculated one dimensional settlement should be increased by about 15% to derive the equivalent three dimensional settlement.

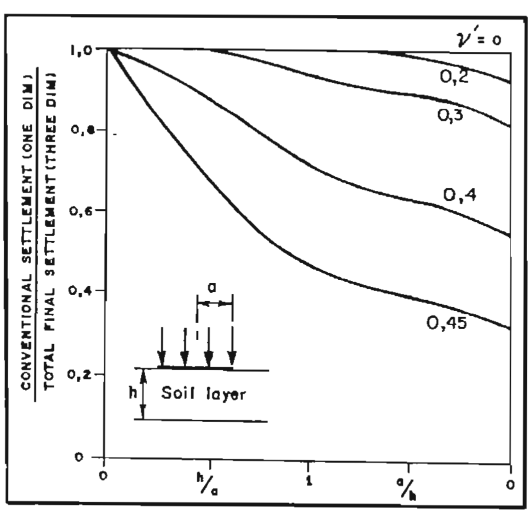


Figure B5.3 : Error in settlement for one dimensional approach (Davis and Poulos, 1968)

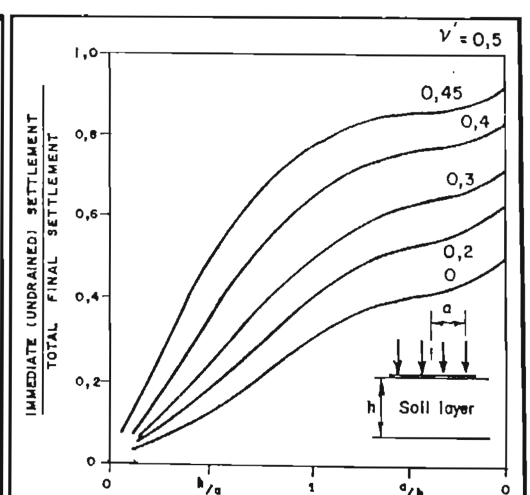


Figure B5.4 : Relative importance of immediate settlement (Davis and Poulos, 1968)

Measurement of E_u is difficult and therefore values must either be obtained from equation B5.10 which requires a knowledge of the value of Poisson's ratio, or values of E_u can be estimated from E_u/c_u relationships such as shown in Figure B5.5. It would be useful to have values of E_u/c_u from back analysis of settlements for local soils; unfortunately such data is not readily obtained other than from detailed research level investigations.

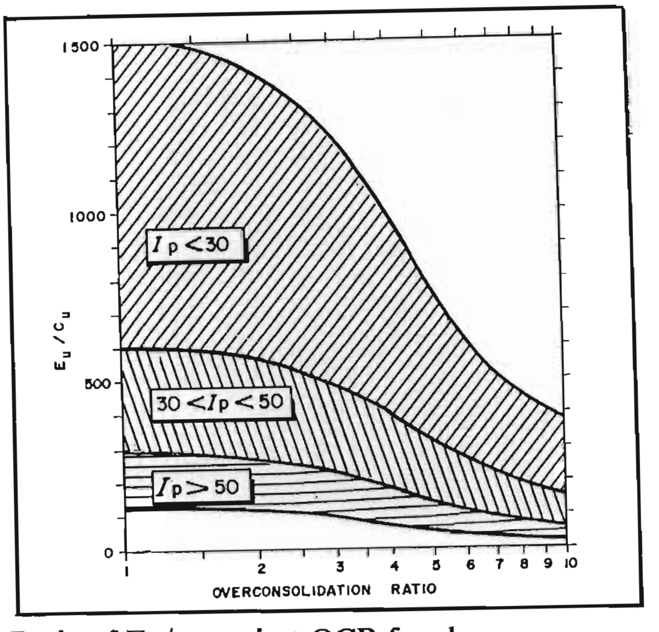


Figure B5.5 : Ratio of E_u/c_u against OCR for clays

For anisotropic soils the equations B5.9 and B5.10 should be modified to account for differences in the horizontal and vertical Poisson's ratios and the resulting estimated settlements may change by 20% for high ratios of E'_h/E'_v.

Burland, Broms and de Mello (1977) discuss these points extensively in their state of the art of settlement predictions for foundations, but not for embankments where the factors of safety or stress ratios will generally be much higher than for structural foundations. They conclude that because of all the complications, both theoretical and practical, the elastic methods have no advantage over the Terzaghi approach in giving accurate predictions of settlement.

For embankments on soft clays there is a further complication in settlement prediction which is the result of the relatively high stresses imposed compared with the in situ stress. Local yield occurs which is non elastic and cannot be predicted on the basis of elastic type parameters. In order to predict deformations it is necessary to use suitable (stress-strain) relationships and numerical methods.

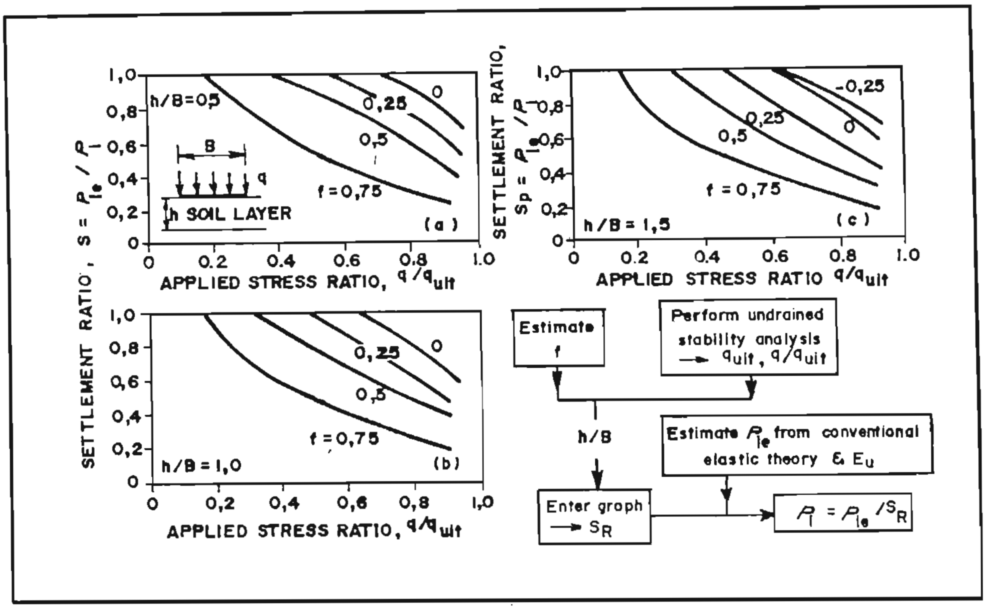


Figure B5.6 : Relationship between settlement ratio and applied stress ratio for strip foundation on homogeneous isotropic elastic layer (D'Appolonia et al, 1971)

D'Appolonia et al (1971) developed a method to obtain a settlement ratio S_R defined as ρ_{ie}/ρ_i : where ρ_{ie} is the immediate elastic settlement and ρ_i is the actual immediate settlement including local yield. S_R is obtained from sets of curves -Figure B5.6 relating S_R, the applied stress ratio q/q_{ult} and f, the initial shear stress ratio. The initial stress ratio is defined below and can be obtained from Figure B5.7 :

$$f = \frac{\sigma'_{vo} - \sigma'_{ho}}{2\ s_u} \qquad \text{B5.11}$$

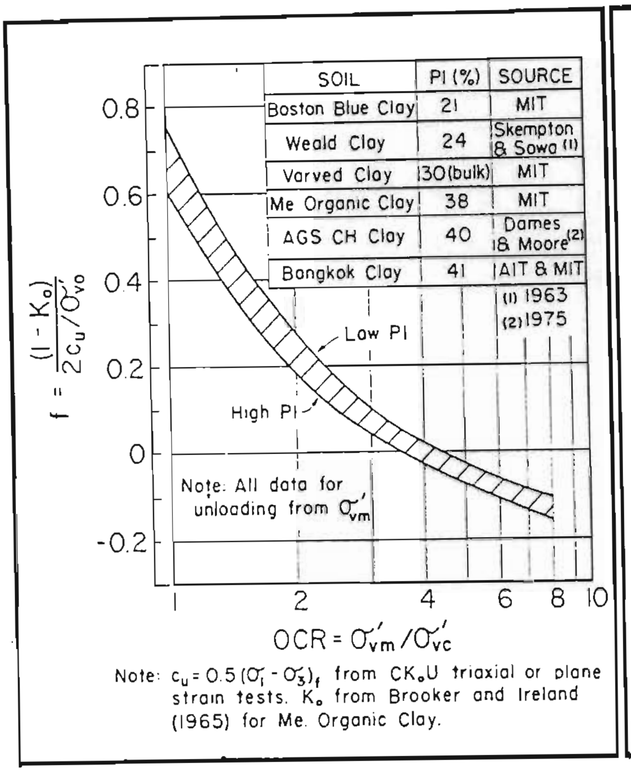
or

$$f = \frac{1 - K_o}{2\ s_u/\sigma'_{vo}} \qquad \text{B5.12}$$

For soft normally consolidated clays f is usually in the range 0,6 to 0,75, and for embankments along the Natal coast h/B is about 0,5. Such embankments usually have a stability problem so q/q_{ult} is at least 0,6 and may well be up to 0,8.

From Figure B5.6 it can be seen that for these conditions, and with f of say 0,65, then S_R is 0,6.

It is therefore clear that local yield may have a significant influence on the total settlement.



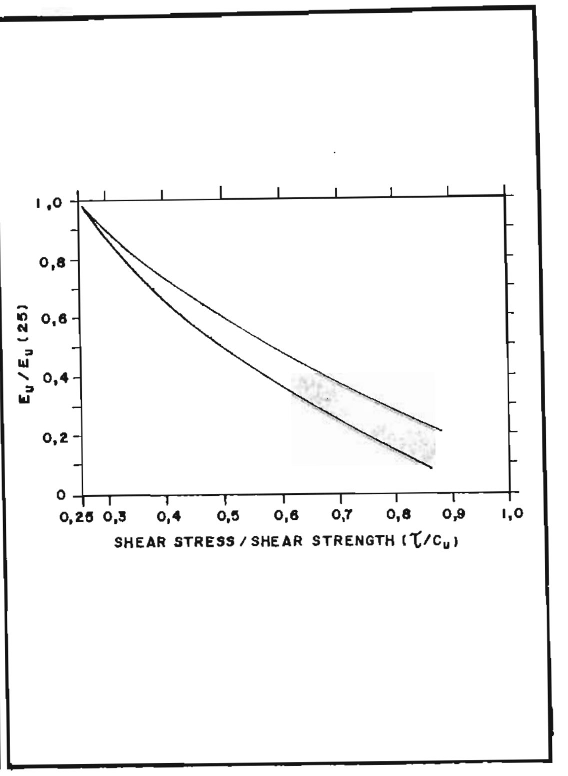


Figure B5.7 : Relationship between initial shear stress ratio and OCR (Ladd et al, 1977)

Figure B5.8 : Reduction of E_u with increasing stress level (Ladd et al, 1977)

In summary, therefore, for the estimation of settlements of embankments on alluvial deposits, and for the back analysis of such settlements, it is necessary to estimate the following separate components of settlement :

- Local yield
- Immediate pseudo elastic
- Primary consolidation
- Secondary compression.

These components are not independent of one another, and the relationships between them are not simple. In most practical cases it will not be possible to determine the specific relationships for particular soils and stress conditions and it is necessary to make use of the general relationships shown in Figures B5.3 to B5.8.

For routine cases where reliance is placed on the interrelationships between the separate settlement components then the primary consolidation is the first component to be determined. This is based on measurement of the coefficient of volume compressibility, m_v from laboratory consolidometer testing or from in situ testing, viz cone penetration testing and application of the equations given earlier in this section.

It follows that a reliable estimate of m_v is essential. Generally, such estimates can be obtained from conventional laboratory testing since the test is a direct measurement of compressibility, albeit under different conditions from those in the field. To overcome this difficulty the method has re-emerged of in situ screw plate tests and these, and pressuremeter tests, should also lead to direct measurements of m_v, or elastic parameters, and thus give reliable estimates of settlement. Cone penetration testing is an indirect measurement of m_v and relies on correlations of cone pressure, q_c, with compressibility as discussed in previous sections. In essence the correlations are multistaged :

- the first is that q_c and c_u are correlated (and there is little argument on this in principle, although the value of the correlation factor N_k is open to discussion and left to local correlations to establish);
- the second stage is that E_u and c_u are related and again there is much evidence to support this in principle, but the actual correlations for most soils have not been established;
- the third stage is that E' and E_u are related; this is theoretically so, but in practice the relationship is often observed to have a wider range than expected;
- the fourth stage is that m_v and E' are related; this again is theoretically so and the relationship is a function of Poisson's ratio, but is complicated in practice by non homogeneity and anisotropy.

It is thus a formidable task to assess each of the above stages with sufficient reliability so that a final reliable q_c to m_v relationship can be determined, since consideration of the stages indicates that Poisson's ratio, overconsolidation ratios, stress ratios, soil type, in situ state, inhomogeneity, and anistropy are all involved.

Recourse is therefore made to direct correlations of q_c with m_v from laboratory consolidation tests and from direct field observation of q_c with measured settlements. The author's aim has been concerned firstly with developing the cone penetration test equipment so that accurate measurements can be made in soft materials; secondly with correlating CPT q_c values with m_v derived from the back analysis of embankment settlements, and thirdly with deriving coefficients of consolidation, c_v, from piezocone dissipation tests by correlations with observed rates of embankment settlement.

Part C describes the first and Part D the latter two.

PART C : SOUTH AFRICAN DEVELOPMENTS IN CONE PENETRATION TESTING

C1 INTRODUCTION

An historical review of international developments in cone penetration testing was given in Part B so that a background could be provided against which South African developments could be assessed. Section B2 described the mechanical cone systems from those in the 1930's to the present day.

Section B3 covered the development of electrical piezometer cone systems from their beginning in the late 1970's to the present and included mention of the author's contribution to this.

Section B4 discussed the interpretation of cone penetration testing with the emphasis on parameters required for the design of embankments on soft alluvium where large settlements and long consolidation times give rise to significant engineering problems. The author's contribution to interpretation methods is again mentioned.

Part B5 described the application of cone penetration testing for the design of embankments and is a review of current international practice in settlement and consolidation time analyses.

Part C has a similar format to Part B of an historical review, C2; followed by mechanical cone penetration testing, C3; through a description of a consolidometer cone apparatus developed by the author, C4; to piezometer cone developed by the author, C5.

The application of cone penetration testing by the author is described in Part D.

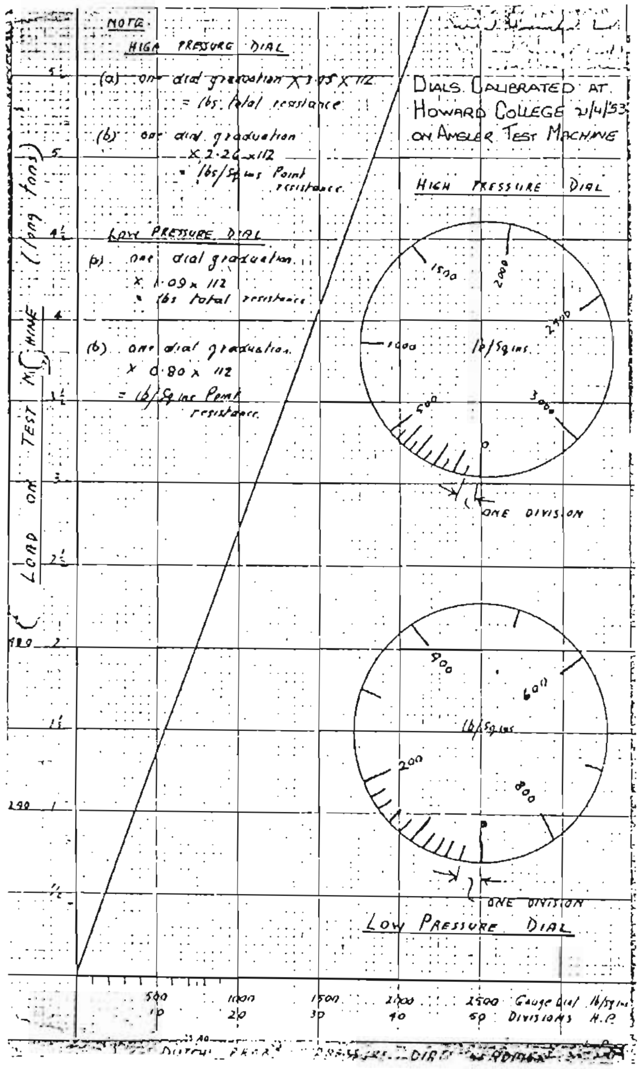
C2 SOUTH AFRICAN MECHANICAL CONE PENETRATION TESTING (1950 - 1975)

There have been few publications by South Africans on cone penetration testing and hence it is relatively simple to trace the history of development of the method locally since the author has been directly involved since 1965. This does not suggest that the system has had little application locally, since the merits of cone penetration testing were appreciated in the early 1950's by Kantey (1951) who advocated its use and was instrumental in introducing the system to South Africa.

In the early years the local system was essentially very similar to that in use in Europe with the exception that smaller diameter equipment, casings and cones, were used. This was because the drilling industry manufactured as standard, E size drilling rods of 33 mm diameter and cones were made to this size, hence the cross section area was smaller than the standard European 1000 mm^2.

The rigs used in South Africa were purpose made locally and similar in concept but not in detail to European machines. The local machines generally used hydraulic rams and pumps from earth moving equipment. No automatically controlled penetration rate was possible and the rate was determined by the judgement of the operator. Although not sophisticated this was adequate for the purpose, which was primarily for pile design. However a less desirable feature which persisted for a number of years was that the load required to push the cone and rods was usually determined by measuring the hydraulic pressure in the main operating ram and not by a separate load measuring system. The accuracy was very poor, particularly at low loads, and uncalibrated pressure gauges exacerbated the problems.

The primary purpose of cone penetration testing at the time was as an economic and relatively accurate alternative to boreholes with Standard Penetration Tests (SPT) for determining the depth and density of sandy subsoils for piles. Such strata would probably require SPT-N values of a minimum of say N = 15, for piling, which is equivalent to a cone pressure q_c of about 7 MPa. A 100 kN CPT rig would generally be limited to a maximum of say 40 kN load on the cone, ie 40 MPa, but the pressure gauge would have to measure the full load pressure on cone and rods of 100 kN, which, with a suitable safety margin, leads to a load measuring requirement of 150 kN. The hydraulic rams were usually about 100 mm diameter and hence maximum gauge pressures of about 20 MPa (3000 lbs/in^2) were common. For firm to stiff clays, or loose sands where cone pressures are less than 2 MPa, giving gauge pressures of less than 0,25 MPa, ie about 1%, or 1 division, of the full gauge reading, the accuracy of the system was poor. For soft clays, where cone pressures of one tenth of these could be expected, the system was totally inadequate and in soft to firm clays zero readings were often recorded. Figures C2.1 and C2.2 are records from 1953 showing calibration of



DUTCH PROBES CONVERSION TABLES

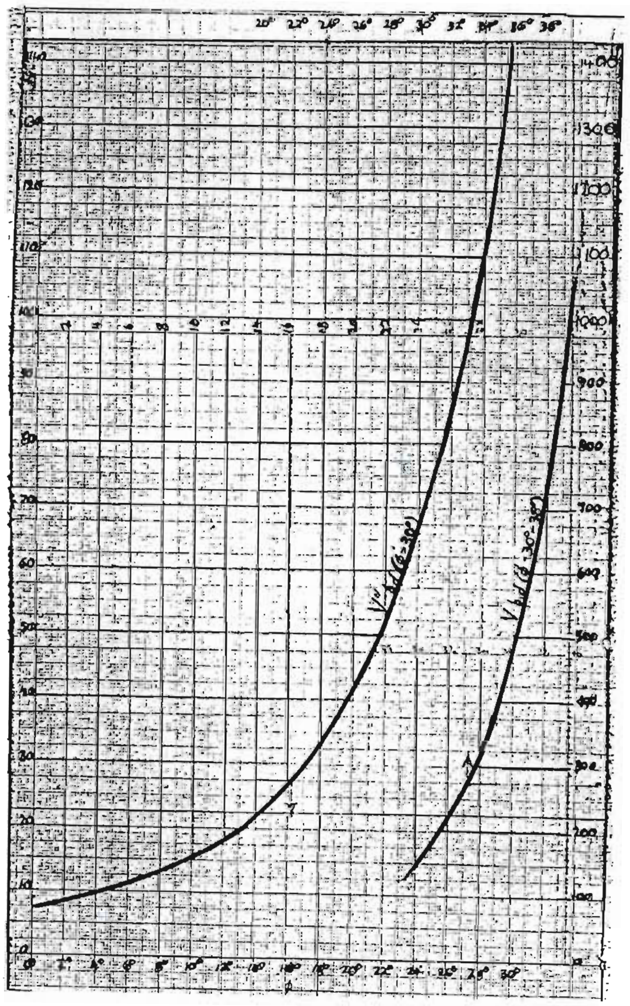
DIAL READING TO LBS/SQ INS.

POINT RESISTANCE

DIAL READING	LOW PRESS. GAUGE FACTOR 86	HIGH PRESS. GAUGE FACTOR 215	DIAL READING	LOW PRESS. GAUGE FACTOR 86	HIGH PRESS. GAUGE FACTOR 215	DIAL READING	LOW PRESS. GAUGE FACTOR 86	HIGH PRESS. GAUGE FACTOR 215
1	86	215	26	2240	5580	51	4390	10900
2	172	430	27	2320	5800	52	4480	11100
3	258	645	28	2410	6000	53	4570	11400
4	344	860	29	2500	6220	54	4660	11600
5	430	1080	30	2580	6430	55	4740	11800
6	516	1290	31	2670	6650	56	4830	12000
7	602	1500	32	2750	6860	57	4920	12200
8	688	1720	33	2840	7080	58	5000	12500
9	775	1930	34	2930	7300	59	5080	12700
10	860	2150	35	3010	7510	60	5170	12900
11	946	2360	36	3100	7720	61	5260	13100
12	1030	2580	37	3180	7940	62	5340	13300
13	1120	2800	38	3270	8150	63	5430	13500
14	1200	3000	39	3360	8360	64	5520	13700
15	1290	3220	40	3440	8600	65	5600	13900
16	1370	3430	41	3530	8800	66	5680	14100
17	1460	3650	42	3620	9010	67	5760	14300
18	1550	3860	43	3700	9220	68	5850	14500
19	1640	4080	44	3780	9450	69	5940	14800
20	1720	4290	45	3870	9660	70	6030	15050
21	1810	4510	46	3960	9880	71	6110	15270
22	1890	4720	47	4050	10100			
23	1980	4930	48	4130	10300	FOR	1/4 ADD 21	1/4 ADD 51
24	2060	5150	49	4220	10500		1/2 " 43	1/2 " 107
25	2150	5360	50	4310	10700		3/4 " 64	3/4 " 158

CALIBRATION CHART AND CONVERSION TABLE

Fig. C·2·1 (a & b)



ANGLE OF FRICTION Ø (degrees).

V_{bd} VALUE TO Ø.

V_{bd}	Ø (degrees)	V_{bd}	Ø (degrees)
7	0	75	25
7·5	1	81	25·5
8·2	2	86	26
9·0	3	93	26·5
9·6	4	99	27
10·5	5	106	27·5
11·4	6	112·5	28
12·2	7	121	28·5
13·2	8	128	29
14·2	9	139	29·5
15·2	10	145	30
16	11	165	30·5
17	12	180	31
20	13	200	31·5
22	14	235	32
24	15	250	32·5
26·8	16	275	33
30	17	285	33·5
33	18	330	34
37	19	360	34·5
41	20	380	35
45·5	21	435	35·5
51·5	22	475	36
54	22·5	535	36·5
58·5	23	600	37
62·5	23·5	685	37·5
66·5	24	740	38
71·5	24·5	825	38·5

V_{bd} = ANGLE OF FRICTION.

Q = DEGREES.

Ckd = TOTAL POINT PRESS.

P_b = OVERBURDEN

$V_{bd} = \frac{Ckd}{P_b}$

Q from Curve.

CHART AND TABLE TO DERIVE ø FROM C_{kd}/P_b (V_{bd})

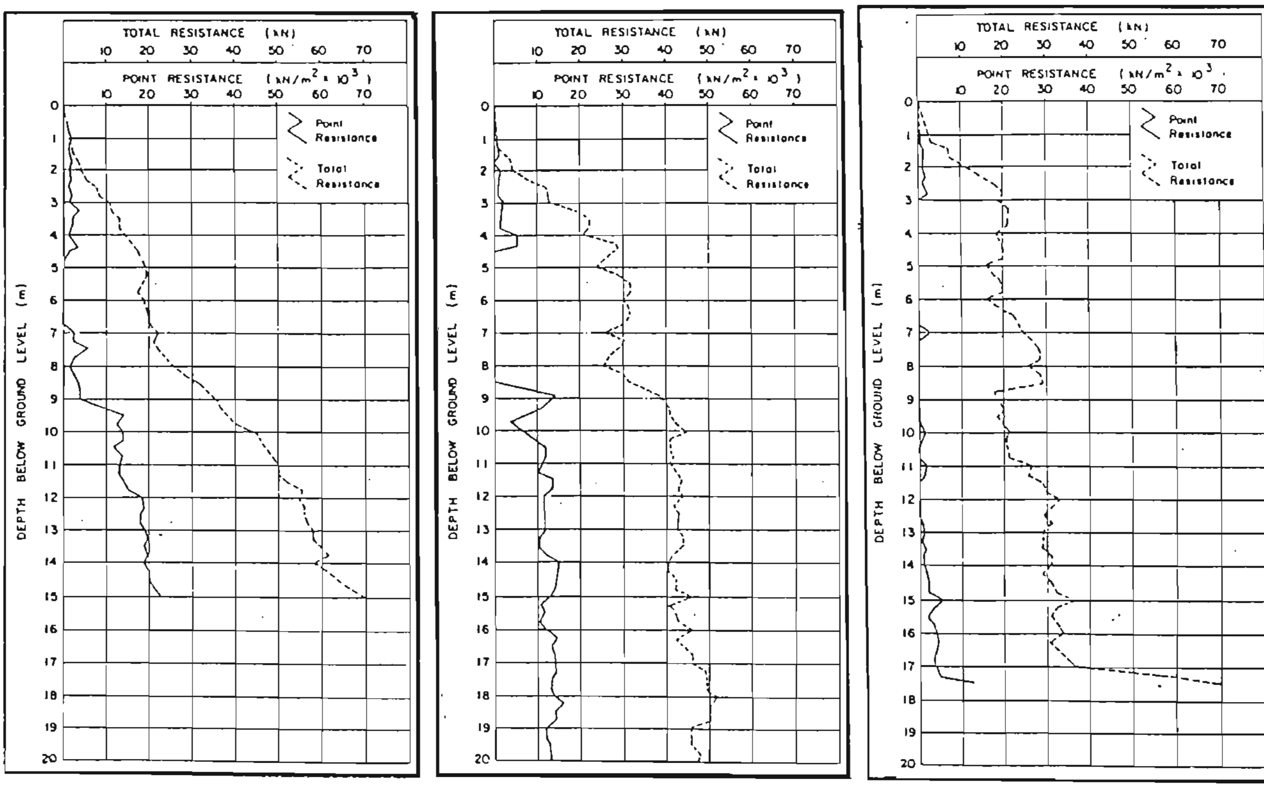
the gauges, (at Howard College); conversion tables from gauge readings to cone pressures; and tables and graphs for evaluating friction angles from $V_{bd} = C_{kd}/P_b$. Typical CPT logs from the mid to late 1960's period are given in Figure C2.3 : those in (a) are for loose to medium silty and clayey sands, and in (b) for soft to firm clay and loose to medium dense sand layers.

The operating system did not change from the mid 1950's to the mid to late 1960's and indeed it is probable that in this period some of the initial care given at the introduction of cone penetration testing had diminished as the application became more routine.

The author, who had previously used Dutch CPT machines in the United Kingdom in 1961 - 1963 whilst working for Soil Mechanics Ltd, joined Kantey and Templer in Durban in early 1965 and was therefore able to participate in this early South African CPT work, which took place primarily in Durban. A milestone in this was a project for the City Engineer, Durban at the Dalbridge Flyover in Durban. The author claims no credit for initiating or supervising this extensive investigation, but was involved in some detail in the interpretation of the results and contributing to the project report. For this reason, and because the work was a well documented example of the fairly early use of the CPT in South Africa, the project is briefly summarized.

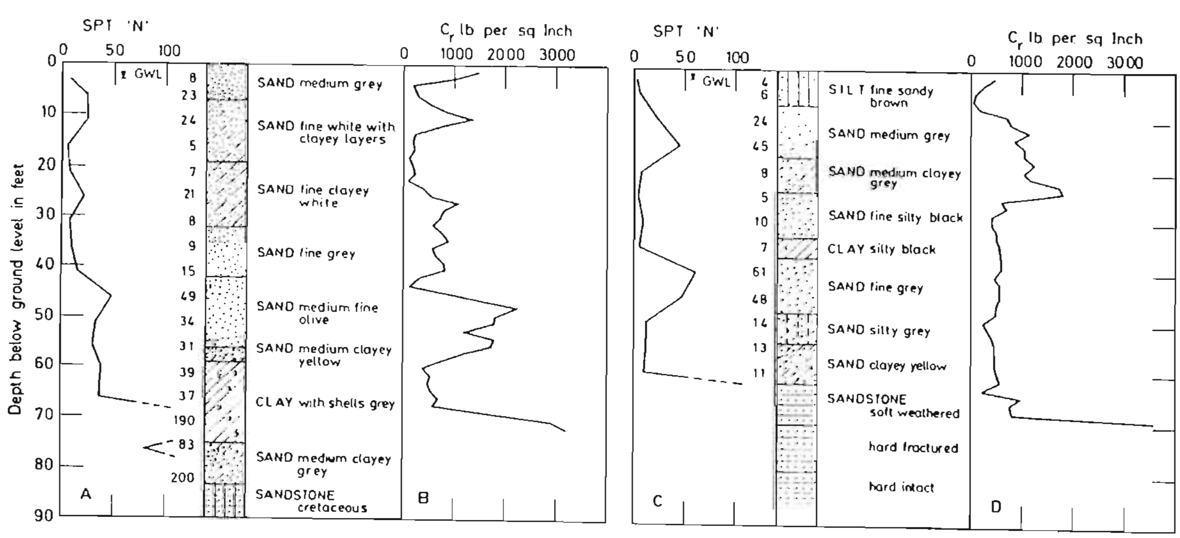
The Dalbridge Flyover is towards the eastern end of the Durban Southern Freeway and carries the latter over numerous railway lines which serve the harbour in that area. It was necessary to elevate the freeway and the method chosen was to place it on a fill which, because of space restrictions, required retaining walls.

The subsoils comprised about 25 m of very loose to medium dense fine sands, clayey sands and soft clays, overlying about 6 m of cretaceous sediments over the Ecca shale bedrock. The reinforced concrete retaining structure had two lines of walls, about 30 m apart, enclosing the fill. The walls rose from ground level to 7 m high over a 150 m length and maintained that height for a further length of 450 m. The walls consisted of individual units each 9 m long (30 feet) and of widths proportional to the heights. The fill retaining faces were at 45° since this provided an economical structure with the most uniform bearing pressure. Piling the units, or any other wall structure, would have been unacceptably costly and because significant settlements were expected, minimizing



TYPICAL LAYERED SOFT TO FIRM CLAY AND LOOSE TO MEDIUM DENSE SANDS

Fig. C·2·3 (b)



Freeway Interchange site at Dalbridge Durban

Tank Installation sites near Airport Durban

TYPICAL LOOSE TO MEDIUM DENSE SILTY AND CLAYEY SAND

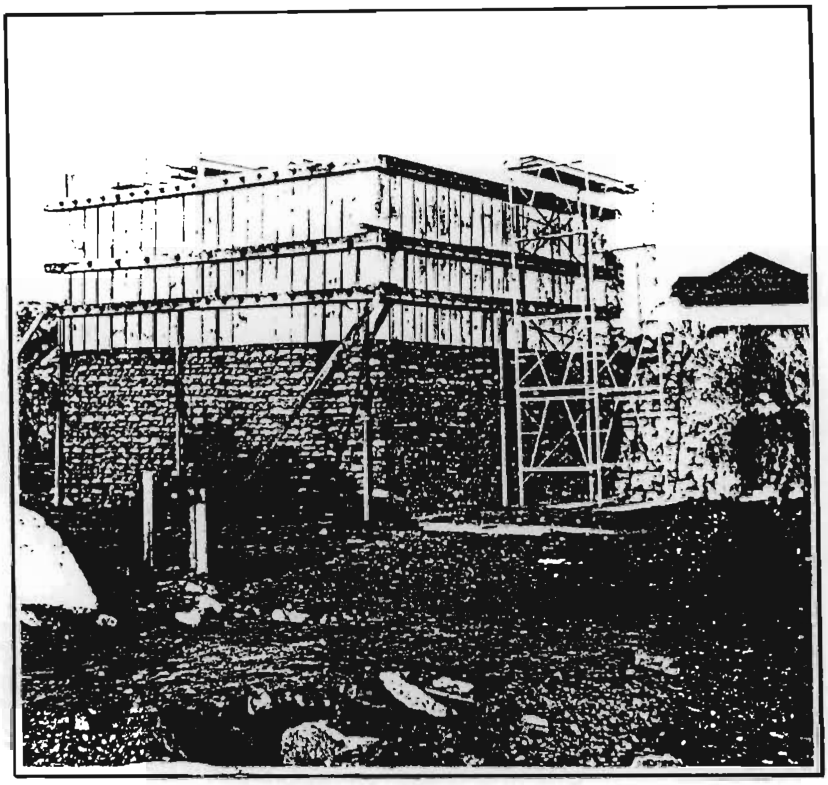
Fig. C·2·3 (a)

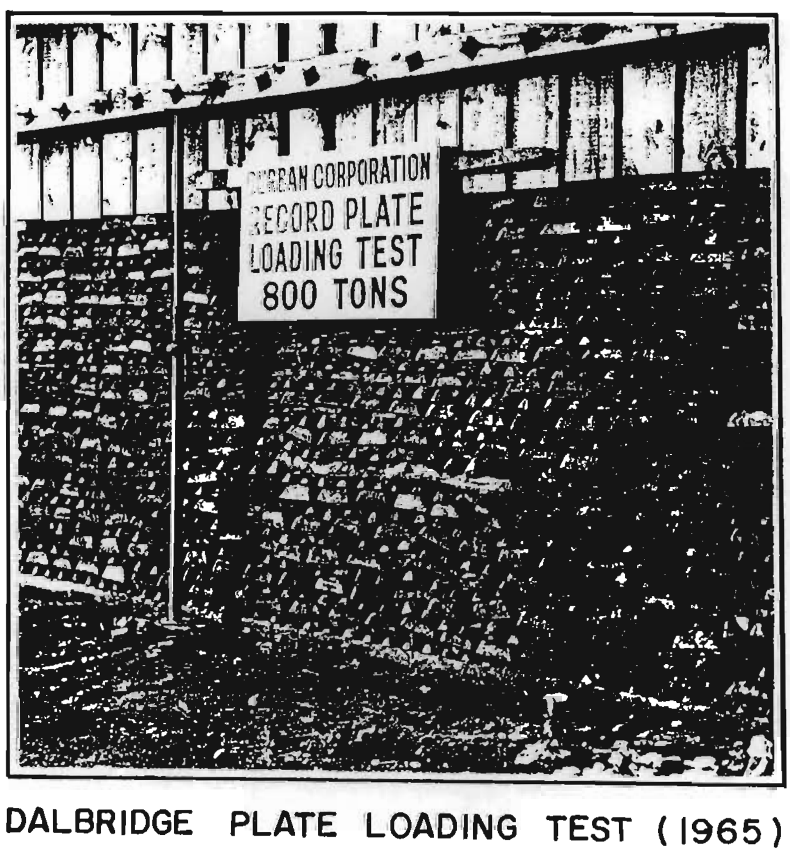
differential settlements of the wall units was an important factor in the design. Due to financial restrictions the investigation for the project took from 1961 to 1965 and in this period thirteen boreholes and fifteen CPT's were put down as well as other sampling holes and inspection shafts.

It was recognized that as valuable as SPT's and CPT's were to provide in situ assessements of compressibility, it was necessary to calibrate these against other tests. Although undisturbed sampling of the more clayey soils, followed by laboratory testing was feasible, most of the subsoil comprised cohesionless soils which precluded this approach. The City Engineer's Department and Kantey and Templer, assisted by the Building Research Station of the Council for Scientific and Industrial Research, conducted a very large plate loading test - Figure C.2.4 - and small diameter screw plate tests. The description of the tests given in the following two paragraphs is condensed from the project report in which the original photographs appeared.

The plate was a reinforced concrete 6,1 m square slab which was loaded with 800 tons of pig iron giving a pressure of approximately 200 kPa. CPT's were put down before and after the test at positions immediately surrounding the plate and through holes left close to the centre of the slab. Precise levelling of the corners and centre of the plate was carried out during the loading, for 5 weeks after loading, during the subsequent unloading and for 6 weeks after the removal of the load. It was intended to leave the load in place for a longer period to give a better measure of longer term settlement but the pig iron load was required for export. Nevertheless, the rate of settlement data indicated that the settlement after full load was only about 4 mm in 5 weeks compared with 75 mm during the loading stage - Figure C2.5.

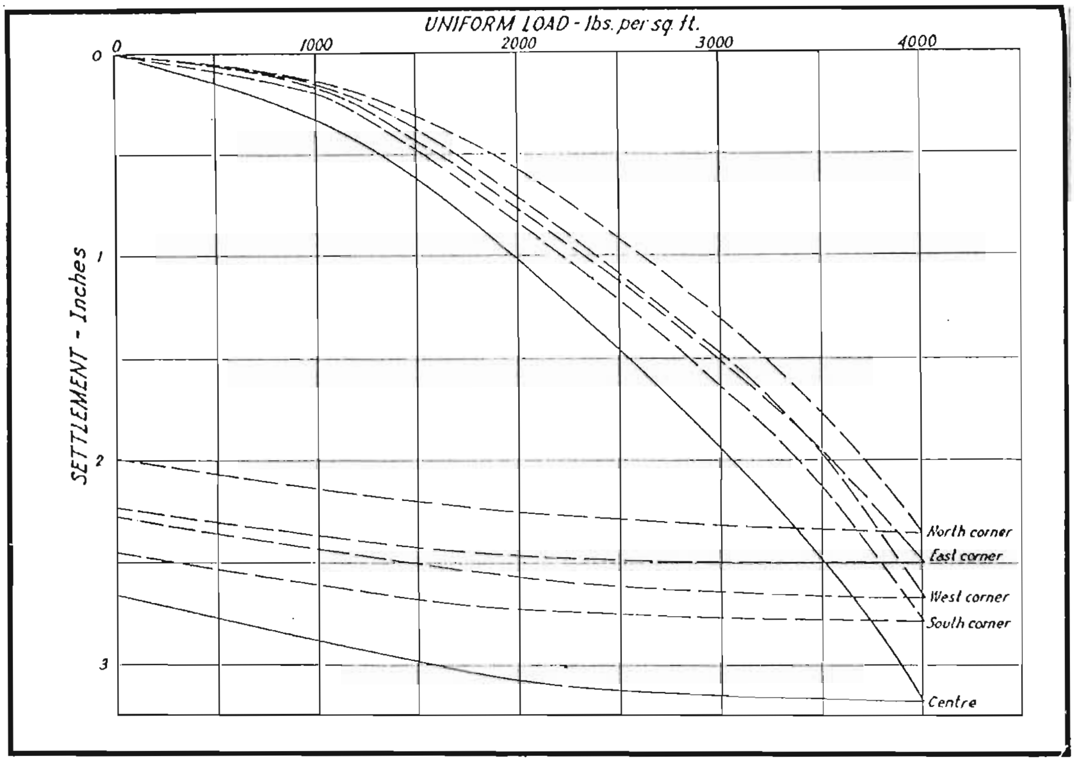
The screw plates were 6, 9 and 15 inch diameter; the tests were at depths of up to 18 m (60 feet) and loaded to pressures of 1,7 MPa. Settlements varying from 2 mm to 70 mm were measured at various stages of loading. At the time the method was generally perceived to be very promising in that it allowed a practicable means, in materials that were difficult to sample, of measuring in situ compressibility which fundamentally was more satisfactory than cone penetration testing. However, reasonably practicable though it may have been, screw plate testing could not compete with cone penetration testing for convenience, economy and simplicity of operation and had, until a recent revival, (eg Bergado et al, 1991), more or less vanished from the scene of site investigation techniques.





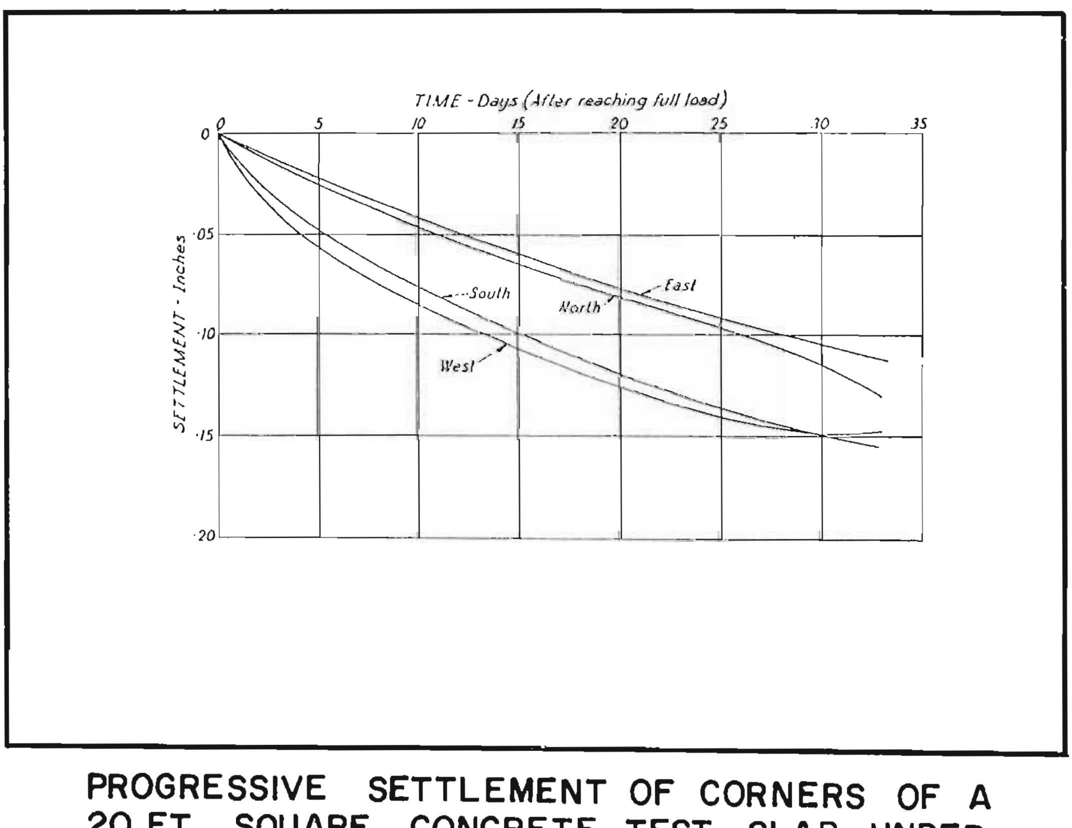
DALBRIDGE PLATE LOADING TEST (1965)

Fig. C·2·4



LOAD SETTLEMENT CURVES FOR 20 FT. SQUARE CONCRETE SLAB

Fig. C·2·5 (a)



PROGRESSIVE SETTLEMENT OF CORNERS OF A 20 FT. SQUARE CONCRETE TEST SLAB UNDER A UNIFORM FULL LOAD OF 4,000 LBS PER SQUARE FOOT

Fig. C·2·5 (b)

The Dalbridge Flyover project, and particularly the very large plate loading test, enabled correlations to be made between measured settlements and predicted settlements from both laboratory and field testing.

In 1961 Schultze and Menzenbach published correlations between compressibility and Standard Penetration Test N values derived from sampling and laboratory testing. These correlations were expressed in regression equations in the form:-

$$1/m_v = 71 + 4{,}9\ N \qquad \text{C2.1}$$

where N is in blows/ft

and m_v is in cm^2/kg

The above was for fine saturated sands and similar expressions were quoted for other materials.

This form of equation was used by the author for the Dalbridge results and modified to generate a local correlation for clayey sands :

$$1/m_v = 18 + 4{,}4\ N \qquad \text{C2.2}$$

In the Dalbridge report SPT N values and CPT q_c values were directly correlated, viz q_c ($tons/ft^2$) = 3N, but no equivalent equations to the N value ones given above were stated. Cone pressures were correlated with compressibility using the then conventional relationship

$$1/m_v = 3/2\ q_c \qquad \text{C2.3}$$

The SPT equation gave settlement predictions which fitted the measured settlements better than those using the CPT data and equation C2.3 to derive compressibilities. The report stated that at this site predictions of settlement using the SPT values were more reliable than those using the CPT data. This general conclusion, however, was not justified since the SPT equation was fitted to the settlement data by adjusting the constants, whereas the CPT equation used only the unmodified q_c and m_v relationship in equation C1.3. Kantey (1965) referred to this work at Montreal.

Webb (1969), again described the Dalbridge Flyover work and that at other sites, and quoted two correlation equations for compressibility and cone values which were derived by the author from the Dalbridge results viz:

$$E(m^2/kN) = 5/2\ (q_c + 3000) \quad \text{for fine to medium sand} \qquad C2.4$$

$$E = 5/3\ (q_c + 1500) \quad \text{for clayey sands} \qquad C2.5$$

It was noted that the correlation of N values and cone pressures at one of sites which was an oil storage tank farm, gave:-

$$q_c = 2{,}2\ N \quad (q_c\ \text{tons/ft}^2). \qquad C2.6$$

Webb concluded this paper by remarking that "more reliable results are obtained from the deep sounding test," a different emphasis from that given in the Dalbridge report. The change can be ascribed to there being both more direct local experience gained in the intervening time and to more international experience being available through the literature.

Webb and Hall, (1969) described the use of cone penetration testing to monitor the efficacy of vibroflotation at a number of sites in the Durban area, viz the Durban Sugar Terminal Silo, a Factory Site and an Oil Tank Site. The author was intimately involved in this work and both advocated and controlled the cone penetration testing which determined the pattern of the vibroflotation.

At the Sugar Silo Site a total of about 50 CPT's were carried out. Initially CPT's were at varying distances from trial vibroflots to assess the increase in density so that the required spacing of the virbroflots could be designed. The CPT's were then used as a control test on the vibroflotation during construction. Similar but less vibroflotation using the CPT as a design and construction control system was carried out at the Factory and Oil Tank Sites.

For the Dalbridge site and for the Sugar Silo work the procedure suggested by de Beer was used for the intepretation of the CPT results, ie they were expressed in terms of the friction angle ø which was calculated from the cone resistance and the assumed overburden pressure - see Figure C2.2. The friction angle was often shown on the logs and was seen as the definitive result of the CPT. Vibroflotation was then specified by requiring a minimum value of ø. However, the procedure for estimating ø was recognized to be approximate and valid only for sands. Where clayey strata or lenses were encountered in the sands the cone resistance, and hence calculated ø, were much lower so that specifying a minimum envelope for ø was impractical since it would require different values for each material. The procedure was therefore changed to specifying a minimum cone resistance and accepting lower values in clayey layers.

The work at these sites in the Durban area in the 1960's gave considerable impetus to a more general acceptance of cone penetration testing for subsoil investigation not only for pile design but for the estimation of settlements.

In the period 1969 - 1973 the author was primarily involved in the geotechnical investigations being conducted for the development of the national road system in Natal which is described in Part A.

Since it was apparent that highly significant problems would be encountered with embankments over the estuarine deposits, the author decided that improved geotechnical investigations would be necessary and that cone penetration testing could fulfil an important role. The conventional investigations then, as now, consisted of boreholes with undisturbed sampling for cohesive materials, followed by laboratory testing, and boreholes with Standard Penetration Testing. The latter, in the softer deposits gave N values in the range 0 to 5 and it was apparent that the results were very dependent on the operator and equipment, particularly that for raising and dropping the SPT sliding hammer.

To overcome the operator and equipment problem for the SPT the author introduced the use of automatic trip hammers. The method previously in vogue was based on American practice and consisted of a rope wrapped around a winch on the drilling machine. As has often been recorded, Fletcher (1965), the method suffered the twin drawbacks that the hammer did not necessarily fall freely and that the height of the drop depended on the operator's skill and diligence. Automatic trip hammers worked on the principle of a mechanical latch which was released when it passed over a larger diameter section of the guide rod thus dropping the hammer. A disadvantage of these systems is that if the lifting cable is co axial with the drill rods, which is mechanically ideal, then head room above the anvil and hammer assembly may be very restricted unless a tall mast or tripod is used with the drilling rig. If, on the other hand, lifting non axially is accepted to reduce head room problems, then lateral loading on rods causes other problems. The author developed, and used successfully for a number of years, an electro-magnetic trip hammer. Figure C2.6 shows the device which worked off the drill rig 12 volt battery. Different switch arrangements were used with various degrees of success : (a) in which the current was interrupted by contacts passing over an insulated section of the hammer guide, and (b), in which an industrial switch was operated by passing over a smaller diameter section of the hammer guide rod. The system worked well and undoubtedly played a part in changing the site investigation industry to using automatic trip hammers, although of the mechanical and not electromagnetic type.

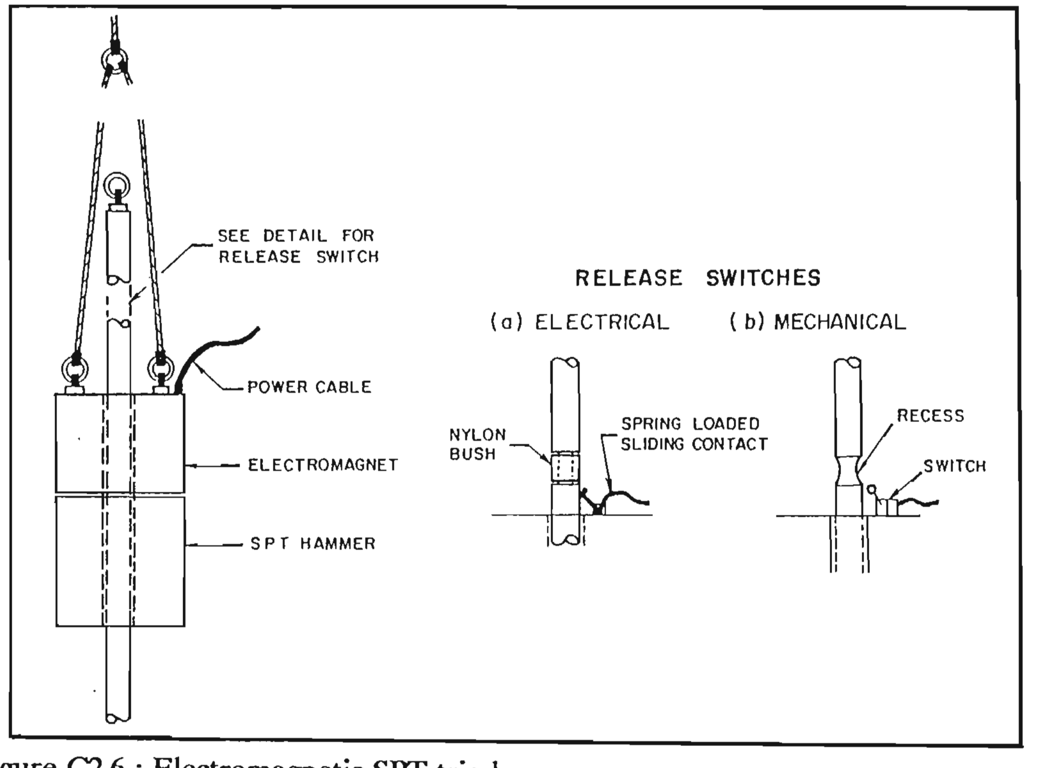


Figure C2.6 : Electromagnetic SPT trip hammer

In the course of developing mechanical and electro-magnetic trip hammers numerous tests were made to compare the automatic trip hammer with the rope over a cathead system. It was found that the blow counts for the two methods were significantly different, there generally being about 25% and sometimes 50% more blows for the manual system than for the automatic.

However, despite the overall improvement in Standard Penetration Testing, the method was of little use in the more clayey estuarine deposits other than as an indication of which strata required more sophisticated investigation.

The author adapted a diamond drilling machine so that it could also be used for cone penetration testing. This entailed fitting an improved hydraulic flow control valve so that the penetration rate could be more finely controlled and an improved hydraulic pressure measurement system comprising a high and low pressure gauge. The drilling machine was also equipped with screw augers which could be drilled in to act as holding down anchors.

A further improvement was the importation from Holland of standard 60^0 mantle cones (35.7 mm diameter) and the use of stronger EW (35 mm diameter) rods with a taper screw thread pattern instead of the lighter EX rods.

Whilst these improvements accentuated the importance of standardised techniques there were still deficiencies. The penetration rate control was very much improved - and it was then generally thought that penetration rate was more critical than has been subsequently shown to be the case - but the system continued to rely on measuring penetration resistance by reading the hydraulic ram pressures albeit through a high quality double gauge system.

The author therefore developed a method based on that used in Europe of a closed circuit hydraulic load cell. In essence this was simply a commercially available hydraulic jack connected to a twin gauge pressure measuring unit. Adaptors were made for the jack so that it could be fitted to the cross head of a drilling machine or to the ram of a penetration test rig. In the former case the adaptor was a socket end of a standard (N) sized drill rod which could then be screwed onto a rod and held in the drill chuck.

The jack and connected gauges were calibrated in the laboratory and the calibration checked from time to time. The system worked well and was assembled from readily available pieces of equipment. As a result it was specified for all investigations for the Natal Roads Department and has subsequently become normal practice in South Africa and at about the same time it was specified that the internationally recognized cone size ($10\ cm^2$) should be used.

In 1974, Webb reporting on South African practice to the First European Symposium on Penetration Testing, ESOPT I stated that the older E-rod equipment was still in general use and the system of measuring pressures from the main hydraulic ram. This, however, although a fair description at the time it was written, was already outdated in some respects by the time it was published with regard to the equipment being used. The comments on the usage of cone penetration testing are nevertheless valid and when these are compared with reports by other international contributors at ESOPT I it can be seen that South African practice, compared favourably with that anywhere else in the world other than in western Europe where it originated.

Despite this however it was abundantly clear that the available cone penetration equipment both internationally and locally was inadequate for the investigation of the softer clays found in the estuaries along the Natal Coast because the load measuring systems were unable to measure the low cone pressures required. For example a subsoil with an undrained shear strength of 15 kPa would be expected to give a cone resistance of about 200 kPa. The usual dual gauge measuring units consisted of a 0 - 100 MPa high load gauge and a 0 - 10 MPa low load gauge. A cone resistance of 200 kPa is therefore less than 1% of the full scale low gauge reading - depending on the area of the load cell. Since a field operating gauge of this nature is unlikely to be more accurate than say 1% of full scale reading, the accuracy of assessment of shear strength was hardly sufficient to allow any design decisions to be made other than that further investigation by some other means is essential.

Nevertheless the inhomogeneity of the estuarine deposits meant that a relatively inexpensive near continuous testing method such as cone penetration testing was in many respects ideally suited for these subsoils. This, and the fact that the problems of road embankments on soft soils were recognized as a major difficulty for the design and construction of roads, led to the author being invited to the National Institute for Transport and Road Research in Pretoria early in 1974 to develop cone penetration testing.

C3 MECHANICAL CPT EQUIPMENT AND INTERPRETATION DEVELOPMENTS IN SOUTH AFRICA

During the period at NITRR (1973 - 1977) the author published a number of reports and papers incorporating the use of cone penetration testing and this section comprises a summary of these and of the research which provided the information.

C3.1 Methods of Estimating Embankment Settlements using CPT

The first of these was research report RS/6/74, Jones (1974) Methods of Estimation of Settlement of Fills over Alluvial Deposits from the results of Field Tests. It is a description of international cone penetration testing at that stage and a compendium of the methods of settlement estimation based on correlations of cone resistance with compressibility. These correlations are taken both from international and South African experience and include correlations of CPT q_c values with the Standard Penetration Test, which was then much more familiar in South Africa.

Essentially two approaches were adopted for the estimation of settlements. The de Beer and Martens (1957) method, and the Terzaghi based consolidation equation using the coefficient of compressibility, m_v; these are briefly described in the following subsections.

C3.1.1 de Beer and Martens

The subsoil is divided into an appropriate number of strata on the basis of material type or ranges of cone resistance. The average cone resistance for each layer is estimated from the CPT log; the overburden pressure at mid layer depth is calculated, usually from an assumption of subsoil densities, both above and below any water table, and the increase in pressure at the mid layer depth due to the imposed load (embankment) is calculated using a Boussinesq stress distribution method.

$$s/H = 1/C \ln (\sigma_{vo} + \sigma_z)/\sigma_{vo} \qquad \text{C3.1}$$

where s = settlement of layer

H = thickness of layer

C = compression modulus

σ_{vo} = overburden stress at mid layer

σ_z = embankment stress at mid layer

The compression modulus, C, is given by :

$$C = 3/2\, q_c/\sigma_{vo}$$

where q_c = average cone pressure in layer.

de Beer and Martens specifically referred to upper limits of settlement primarily so that decisions could be made regarding the need for piling of bridge abutment. This and subsequent experience on sands, for which the method was derived, led to Meyerhof (1965) and Schmertmann (1970) suggesting that it should be modified to

$$C = 1{,}9 \text{ or } 2\, q_c/\sigma_{vo} \qquad \text{C3.2}$$

C3.1.2 Coefficient of compressibility, m_v

This method uses a direct relationship between cone pressure, q_c, and the coefficient of compressibility, m_v and the conventional Terzaghi compression equation :

$$s/H = \sigma_z\, m_v \qquad \text{C3.3}$$

and $m_v = 1/\alpha_m q_c$

where α_m = constrained modulus coefficient

The constrained modulus coefficient α_m depends on the material type. In this method the material type would be defined either from boreholes and sampling or from friction ratios obtained from the cone penetration testing.

The method is otherwise similar to that in C3.1.1 the subsoil is divided into layers and the settlements for each layer are calculated on the basis of the average cone pressure and embankment pressure within each layer and summed to give a total. The constrained modulus coefficient requires to be assessed for each layer.

The publication RS/6/74 described the use of the friction sleeve in detail and the purpose of this was to encourage the use of the standard cone together with the friction sleeve. The data given for Mtwalumi - south coast Natal - where the site work was conducted - reflects the first published use in South Africa of the friction sleeve and hence of friction ratios and materials identification by cone penetration testing. It had become practice in South Africa for CPT readings to be taken at 0,5 m or even 1,0 m depth intervals. Whilst this may well have been satisfactory when assessing sand densities for piling, it was inadequate in multilayered sands, silts and clays. The document therefore recommended that the depth interval should be not greater than 0,25 m and that this was necessary and convenient for assessment of friction ratio. This is so because the sleeve is approximately 250 mm above the cone so the calculation of friction ratio should take this depth difference into account.

$$\text{F.R.} \quad = \quad \text{Sleeve Pressure /cone pressure \%}$$

However since the sleeve cannot operate independently but only in conjunction with the cone, the sleeve pressure is obtained by subtracting the cone gauge pressure G_{cz} from the cone plus sleeve gauge pressure G_{csz}. This value is then related to the cone reading at the previous depth interval, ie 0,25 m higher taking account of the areas of the cone and sleeve (1000 mm^2 and 15000 mm^2)

$$\text{F.R. \%} \quad = \quad 6{,}7\ (G_{csz} - G_{cz})/G_{c(z\ -\ 0,25)} \qquad \text{C3.4}$$

Manual recording of gauge readings at 0,25 m depth intervals was tedious and the author believed that automatic systems could be used. It was common locally to record the pressure required to advance the string of rods both with and without the cone so that even without the friction sleeve each depth increment required three gauge readings.

The report recommended that even with good CPT data settlement estimates should be expressed in such a way as to reflect the confidence in the accuracy of the estimate ie

$$\text{Mtwalumi embankment settlement} \quad = \quad 0{,}7 \pm 0{,}2\ \text{m}$$

No data existed in South Africa for the correlation of friction ratios with material type, hence it was recommended that a simplified version of Begemann's (1953) correlation for Europe should be used as given in Table C3.1.

Table C3.1 : Material description from friction ratios

FRICTION RATIO %	MATERIAL DESCRIPTION
0 - 2	sand
2 - 2,5	silty sand
2,5 - 3,2	sandy silty clay
3,2 - 4,0	silty clay
4,0 >	clay

From these derived material descriptions, the cone pressures and published relationships (Bachelier and Parez, 1965; Gielly et al, 1970) the constrained modulus coefficients, α_m are obtained. Alternatively the South African correlations of q_c directly with m_v can be used, equations C2.4 and C2.5.

RS/6/74 also described the derivation of undrained shear strengths for clays using the conventional equation:

$$c_u = N_k q_c + \sigma_{vo} \qquad \text{C3.5}$$

and notes that at that stage no evaluations of N_k for South African clays were available but that the internationally generally accepted value of N_k = 15 for normally consolidated clays appeared to be satisfactory. Thus, for initial conservative assessment of stability, it was recommended that the undrained shear strength should be given by:

$$c_u = 20\, q_c \qquad \text{C3.6}$$

The report noted that general relationships between coefficients of compressibility and undrained shear strength have been postulated for clays from normally consolidated to overconsolidated, (Skempton, 1951)

$$1/m_v = (25 \text{ to } 80)\, c_u \quad - \quad \text{normally consolidated} \qquad \text{C3.7}$$

$$= (70 \text{ to } 120)\, c_u \quad - \quad \text{overconsolidated} \qquad \text{C3.8}$$

and that if comparisons are made of derived m_v values using the various approaches then a large range of values may be obtained from the same cone, q_c, data.

The remainder of document RS/6/74 describes in detail settlement from cone penetration test results for a particular site in order to demonstrate the method.

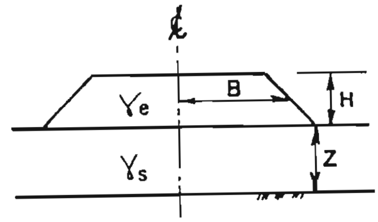
A settlement estimation chart was devised by the author to allow the rapid estimation of settlements - Figure C3.1. This is based on the de Beer and Martens method with an α_m of 1,5; other values of α_m may be selected including those resulting from equations C2.4 and C2.5. The chart assumes a fill density of 20 kN/m^3 but if the density is different an adjusted fill height can be used. The settlement of layers can be individually estimated by subtracting the settlement to the top of the layer ie all material above it assumed to have the same properties, from the settlement to the base of the layer.
A secondary purpose of the chart was to illustrate the marked dependence of settlement estimation on the values of α_m, hence the inappropriateness of detailed calculations.

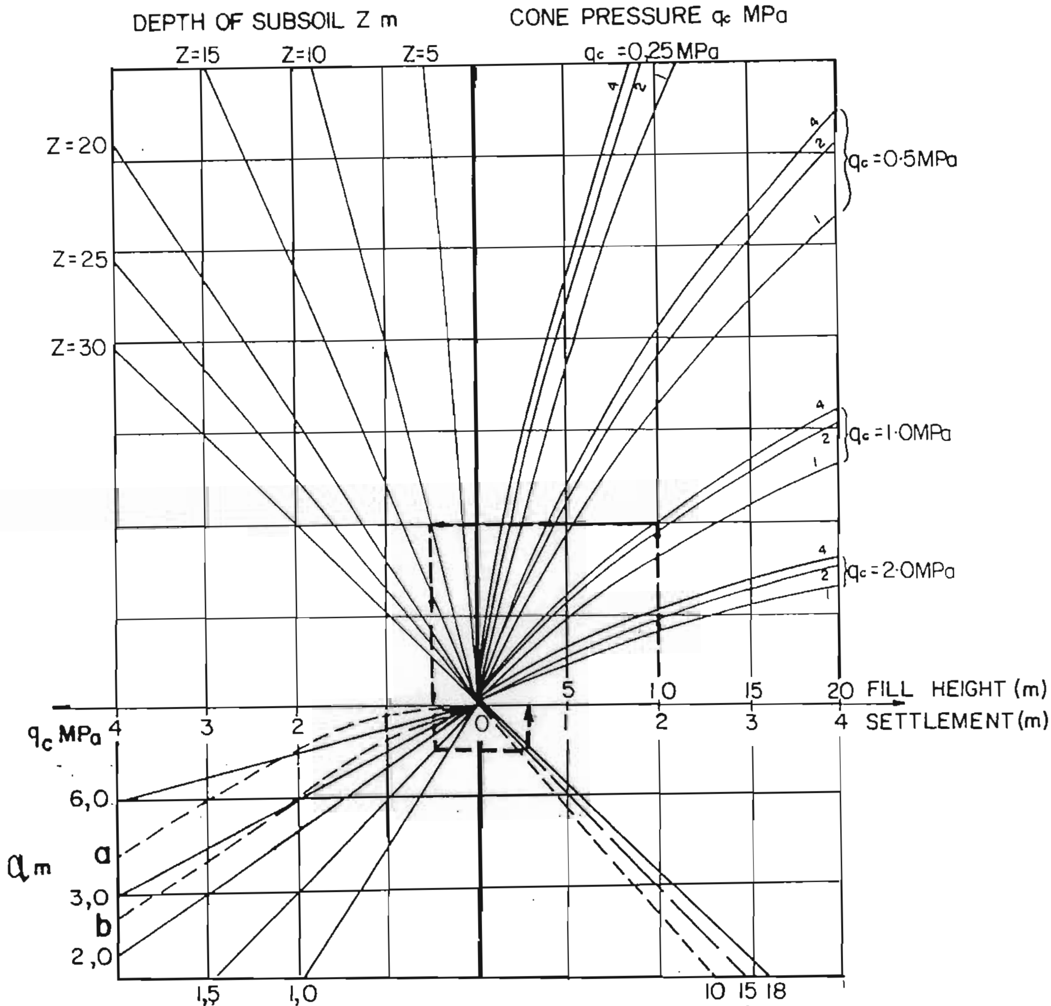
C3.2 Improvements to CPT Equipment - Vane Shear

In view of the progress being made with cone penetration testing the NITRR purchased a CPT rig from Goudsche Machinefabriek B.V. of Gouda, Holland, who had been the principal developers of CPT equipment over the previous 40 years.
The availability of the new equipment in 1974 enabled specific research projects to begin.

The first aspect of this concerned the use of the friction sleeve and this led to the second aspect of improvements to the load sensing system. Both of these are reported in a paper by the author (Jones, 1975) to the Sixth Regional Conference on Soil Mechanics and Foundation Engineering held in Durban, September 1975.

The first aspect discussed in the paper was that in the soft materials encountered in the Natal estuaries, cone penetration testing was very useful but there were limitations. As previously pointed out the system is essentially semi-empirical and relies on correlating cone pressures, or friction ratios, with other soil parameters so that locally applicable correlation factors can be established. The paper refers to field research conducted to correlate cone pressures with undrained shear strength measured by vane testing. In





NOTE: CONE PRESSURE LINES 4, 2 and 1 INDICATE $\frac{2B}{Z}$ VALUES

a --- $1/m_v = 5/2\ (q_c + 3000)$ FINE TO MEDIUM SAND.

b --- $1/m_v = 5/3\ (q_c + 1500)$ CLAYEY SAND.

EXAMPLE: H = 10m; B = 20; Z = 10m; q_c = 1,0MPa

ASSUME; α_m = 1,5; γ_e = 20; γ_s = 15 kN/m^3

∴ SETTLEMENT = 0,60m

CPT SETTLEMENT ESTIMATION CHART

Fig. C·3·1

order to do this vane shear equipment was manufactured which could be used with the CPT rig. Figure C3.2 shows a diagram of the vane shear apparatus. It consisted of a retractable vane mounted in a nose cone attached to a string of CPT rods which had been modified to have square sockets at one end of each inner rod and matching square plugs at the other end. Two sizes of vanes were built to enable a range of shear strengths to be measured. A torque measuring spanner was manufactured for the head since commercially available torque wrenches had inadequate measuring sensitivity. The torque measuring system, which comprised a strain gauged bar connected to a chart recorder system, was calibrated in the laboratory.

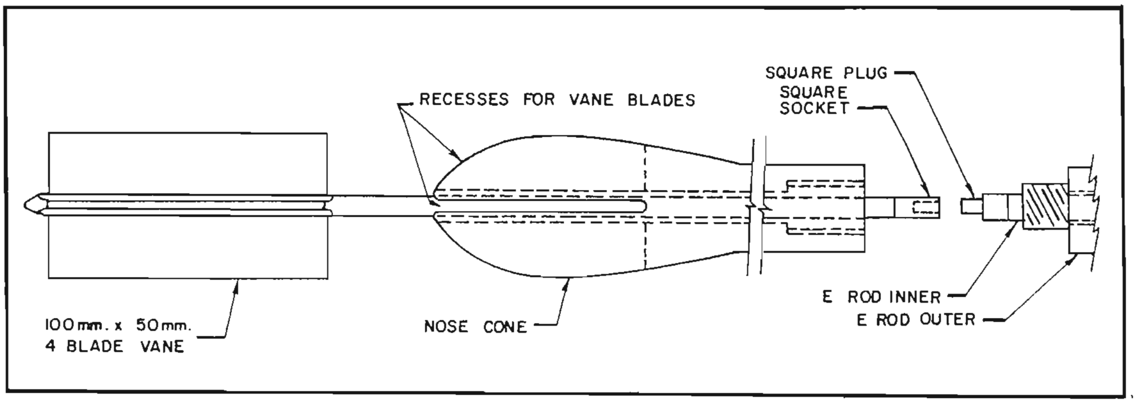


Figure C3.2 : Vane shear apparatus

Field testing was similar to cone penetration testing; the vane, the CPT casing and inner rods were pushed into the soil using the CPT rig. Penetration was stopped at 0,5 m intervals, the thrust transferred from the outer casing to the inner rods to push the vane out of the nose cone housing. The torque wrench was then inserted into the top rod socket and the vane rotated. Since the CPT rig prevented complete rotation of the torque bar a device was used which allowed an offset position for the bar. The initial position of the vane was carefully noted so that after rotation the outer casings could be advanced so that the vane finished in the protected position in the nose cone. Post peak residual shear strengths were easily measured and the sequence recorded on chart. A dummy vane comprising only the shaft without the vane blades was also made and pushed into the subsoil to measure a calibration zero for the system.

The complete system worked extremely well and took only a little longer than conventional CPT measurements. It had advantages over measuring undrained shear strengths by CPT since no soil dependent factors were involved. However the promise, or at least potential, for measuring other parameters, viz m_v, with the CPT mitigated against further development of the vane equipment which was intended only for calibrating the CPT. Conventional CPT's were made in a soft clay at Umhlangane (Sea Cow Lake) and vane shear tests conducted at positions approximately 0,5 m away to allow direct comparisons of the two sets of data. These indicated that N_k = 18,4, if the overburden pressure term σ_{vo} is ignored and N_k = 15,8 if it is included (Jones et al, 1975) - Figure C3.3. The line shown is the linear regression through the origin and gives a correlation coefficient of 0,80 which is considered to be satisfactory.

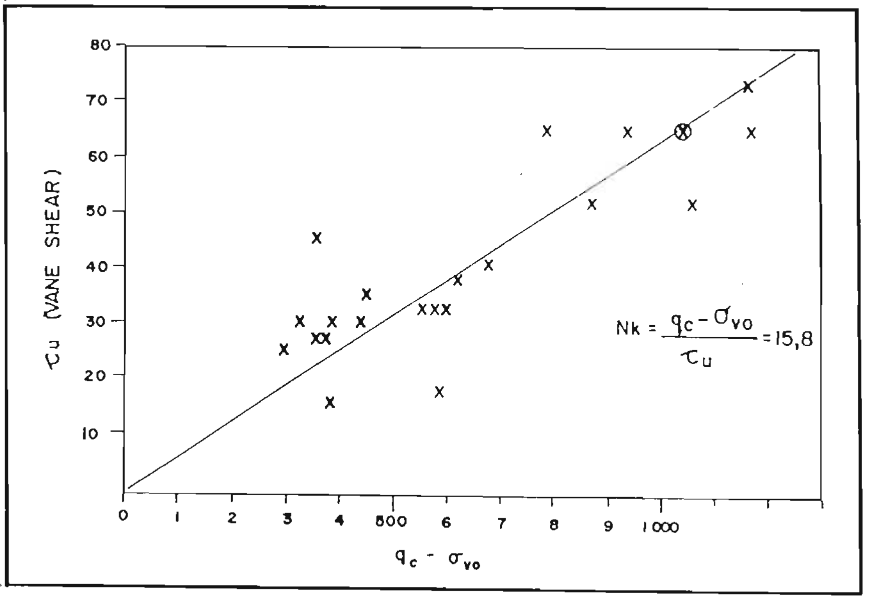


Figure C3.3 : Vane shear against cone pressure

C3.3 Improvements to CPT Equipment - Friction Ratio

The second aspect of the Jones (1975) paper concerned the use of friction ratios to determine material type. The results of 25 CPT's adjacent to boreholes to obtain samples were analysed and a correlation between friction ratios and material type for South African estuarine deposits was established. Begemann's chart whilst invaluable in setting out the approach was considered impracticable for the very soft materials being encountered locally. Alternative correlations were drawn up on the basis of comparisons of friction ratios and percentage passing 0,075 mm nominal aperture sieve (BS Sieve No 200), the percentage passing 20 μm and the plasticity indices, I_p.

Although all three showed some positive correlations they were disappointing. There was no reason to suppose that the relationships between friction ratio and any other indicator of material type should be linear, but the scatter of points suggested that any correlation at all would be difficult to justify. This was contrary to Begemann's experience in firmer materials and the explanation was believed to be that the method of measuring friction ratios was unsuitable for soft variable material.

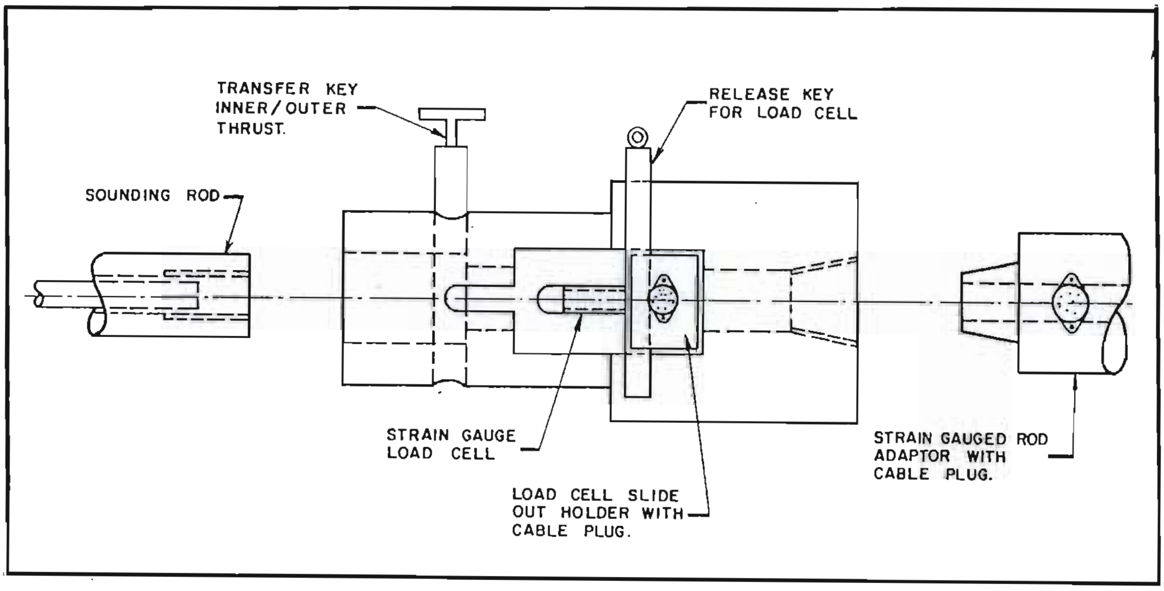


Figure C3.4 : CPT strain gauge load cell

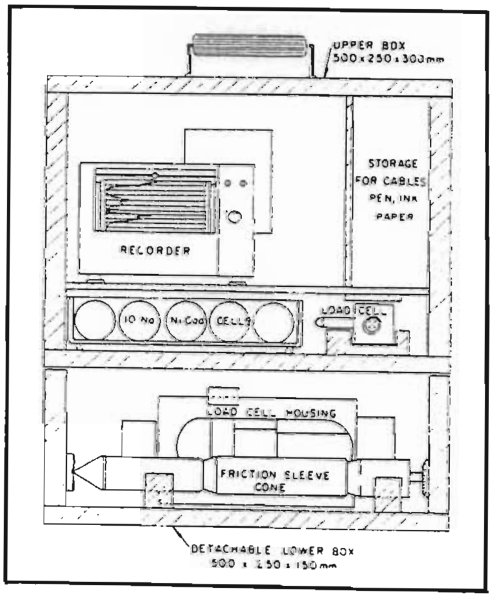


Figure C3.5 : Chart recorder for vane and CPT measurements

For this reason and for the reasons stated in C2 that cone pressure measurements in very soft materials were inaccurate due to the limitations of the load measuring system, it was decided to build an improved system.

Figure C3.4 shows the load measuring system and Figure C3.5 shows the chart recorder system which was also used for the vane shear apparatus.

The essential feature of the load measuring system was that the load on the inner rods was measured by a strain gauge load cell operating directly on the inner rods. A separate strain gauge load cell in the drill rod adaptor was used to measure the load to advance the whole rod string. Both electrical outputs were led to a two channel chart recorder. The cone pressure load cells were made in three sizes to provide a range of operating loads and were laboratory calibrated against proving rings and subsequently checked after field use. An accuracy of ± 10 N could be achieved in the appropriate load range and the limiting factor was reading off the relatively small chart. Cone pressures could then be determined to an accuracy of ± 10 kPa within the soft material range, and hence undrained shear strengths within 1 kPa.

The equipment worked extremely well in the field and represented an important advance in cone penetration testing in soft materials. A disadvantage was that overloading and damage to the load cell could occur if the cone entered a high resistance zone. Spare load cells were carried and were easily fitted should this happen.

Further field tests were carried out with the new equipment and these showed a marked improvement in the correlations of friction ratios with plasticity index or percentage passing 20 μm compared the results from earlier investigations using the previous measuring system. This is discussed in the following section.

C3.4 Improvement in Interpretation of CPT Results - Friction Ratio

Since the Begemann relationship between friction ratio and material type did not operate well in the very low cone pressure range, an alternative relationship between friction ratio and material description was given by the author based on the results of site and laboratory testing : this is shown in Figure B4.18. In addition to the material description, percentage particle size smaller than 20 μm and plasticity index I_{pw} (whole sample) are also shown. The latter additional information on particle size and plasticity index obviously implies a correlation between these two material properties. This was confirmed by the laboratory testing using a linear regression analysis which gave the following relationship with a correlation of 0,88 (see Figure B4.19) :-

$$I_{pw} = 0{,}55\ (\% < 20\ \mu m) - 2{,}36 \qquad \text{C3.9}$$

Whilst this has no specific relevance to cone penetration testing it is a useful relationship for a wide range of estuarine deposits. From Atterberg Limits, it is therefore possible to estimate clay and silt contents with a reasonable level of confidence for the Natal coastal deposits.

The primary purpose of concentrating on material description via friction ratios was to be able to assess the probable order of magnitude of time for consolidation of subsoils under embankment loadings.

Assessment of shear strengths (undrained) from cone penetration testing was well established hence obtaining information for total stress stability analyses presented no major problem. Similarly, particularly if local correlations were available, coefficients of compressibility, and hence settlements could be estimated.

What remained to be found was a method to provide a reasonable estimate of consolidation time. Undisturbed sampling and laboratory consolidation testing were available and conventionally used for this purpose, but in highly variable deposits sufficient representative testing and the selection of the appropriate parameter values was not easy. An alternative approach was desirable, not as a substitute, but as a complementary method.

A potential approach using cone penetration testing was that if the coefficient of compressibility, m_v could be estimated from the cone pressure, q_c and if the permeability, k, could be assessed from the friction ratio through a relationship between particle size distribution and permeability, then the coefficient of consolidation, c_v, could be estimated from : -

$$c_v = k/m_v\gamma_w \qquad \text{C3.10}$$

Although this approach seemed feasible and merited further examination, the difficulty arose from the fact that published relationships between material description, through particle size distribution, and permeability, are approximate and could barely be considered as defining permeability by any closer than an order of magnitude. Dividing such values by an estimated m_v would not improve the overall accuracy and this suggested that a stalemate was reached unless better correlations of k with material type could be obtained. There was no evidence in the literature of such correlations and therefore the only possibility was to actually determine them by appropriate field and laboratory testing. Although it may be fundamentally preferable to determine permeabilities, in practice one might just as well measure the coefficients of consolidation, c_v, directly in the laboratory and relate them to material descriptions if the purpose is to obtain c_v from simple field testing ie friction ratios. The literature gives numerous examples of soils where both c_v and a material descriptor eg I_p, are given and therefore this approach would have the possibility of producing, on a local basis, reasonable correlations of friction ratios with coefficients of consolidation. The end result could not however, be expected to be significantly better than determining c_v to the correct order of magnitude as indicated in Table C3.2.

For the particular problem of the prediction of embankment settlement times the appropriate order of magnitude, ie 1 year, 10 years or 100 years is helpful for basic planning but is insufficient for detailed design. At that stage one requires predictions within say - 50% to + 100% range of the predicted value eg 2 years with limits in the range 1 year to 4 years, or 10 years in the range 5 years to 20 years.

Table C3.2 : Relationship between soil description, friction ratios, particle size, plasticity and coefficients of consolidation

Description	Friction Ratio	% < 20 µm	I_{pw}	c_v m^2/year
clay	5 +	70	35	0,1 - 1,0
silty clay	4 - 5	50 - 70	25 - 35	1,0 - 10
sandy clay, clayey silt	3 - 4	30 - 50	15 - 25	10 - 100
silty sand	2 - 3	5 - 30	2 - 15	100 - 1000
sand	0 - 2	- 5	N.P	1000 +

Thus the CPT in its form as then existed was considered to have reached the limit of its capabilities for the purpose of investigating the consolidation rate characteristics of alluvial deposits.

The attractions of the overall system, however, remained, - viz economy, convenience, continuous or near continuous record of subsoil profile, - and the idea of utilising the equipment in an altered form or mode was explored. This aspect of the research is described in C4.

C4 CPT AS IN SITU CONSOLIDOMETER

C4.1 Introduction

Having arrived at the situation of wishing to utilize the CPT equipment for measuring consolidation rate characteristics it was necessary to envisage a mode in which this could be done. The author conceived the idea of using the cone in a constant stress mode similar to conventional consolidometer testing and this concept formed the basis of the research described in this section. The method was termed the consolidometer-cone test.

C4.2 Research Project 1973 - 1976

The idea, shown diagrammatically in Figure C4.1 was to view the cone as a plate loading test at depth in the soil. The concept was that if penetration was stopped at any depth, the inner rods could then be loaded to induce consolidation of the subsoil. If this consolidation took its expected course the rate would decrease with time and the amount and rate of settlement could be measured, and these could be related to the conventionally defined coefficients of compressibility, m_v, and coefficients of consolidation, c_v. It was accepted that such correlations could be difficult to justify in theoretical terms since adequate theories on cone penetration werer not available, and that the testing envisaged could at the most be expected to give semi-empirical correlations which would be useful in practice provided they could be obtained fairly easily.

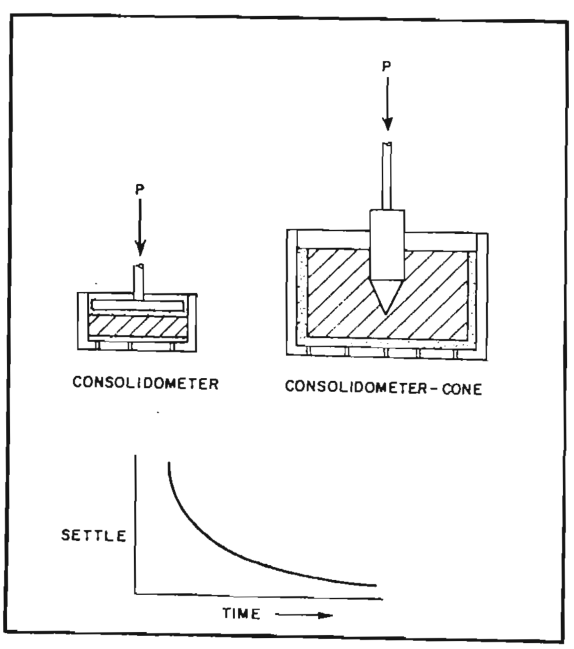


Figure C4.1 : Consolidometer-cone schematic

The research project comprised a series of steps of constructing a number of laboratory prototypes and testing these, and finally developing and testing a field model.

C4.2.1 Prototype 1 (July 1973)

The first prototype was intended to demonstrate whether the idea was practicable. The first laboratory prototype of the apparatus is shown in Figure C4.2.

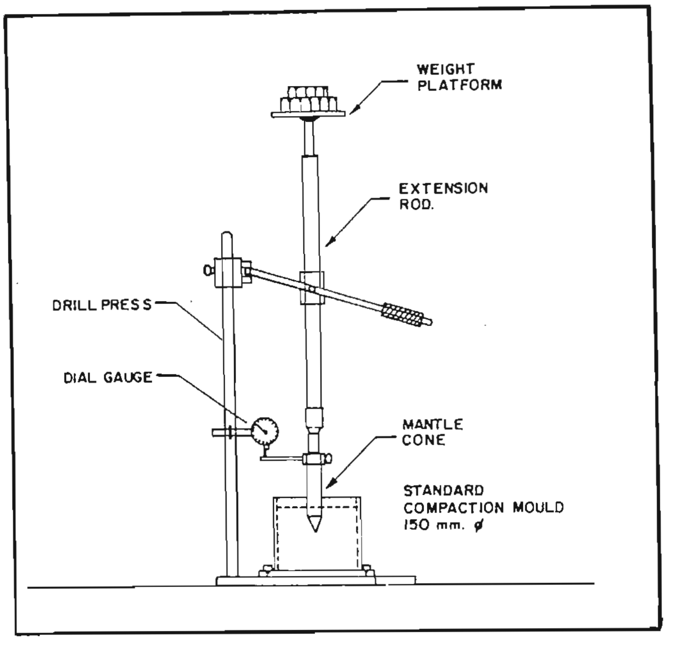


Figure C4.2 : Consolidometer-cone apparatus : Prototype 1

A standard mantle cone, without friction sleeve, was held vertically in a workshop drill press over a CBR mould containing the sample. A moderately clayey soil sample was obtained from the CSIR site and lightly compacted in layers into the mould to simulate a moderately soft natural soil condition.

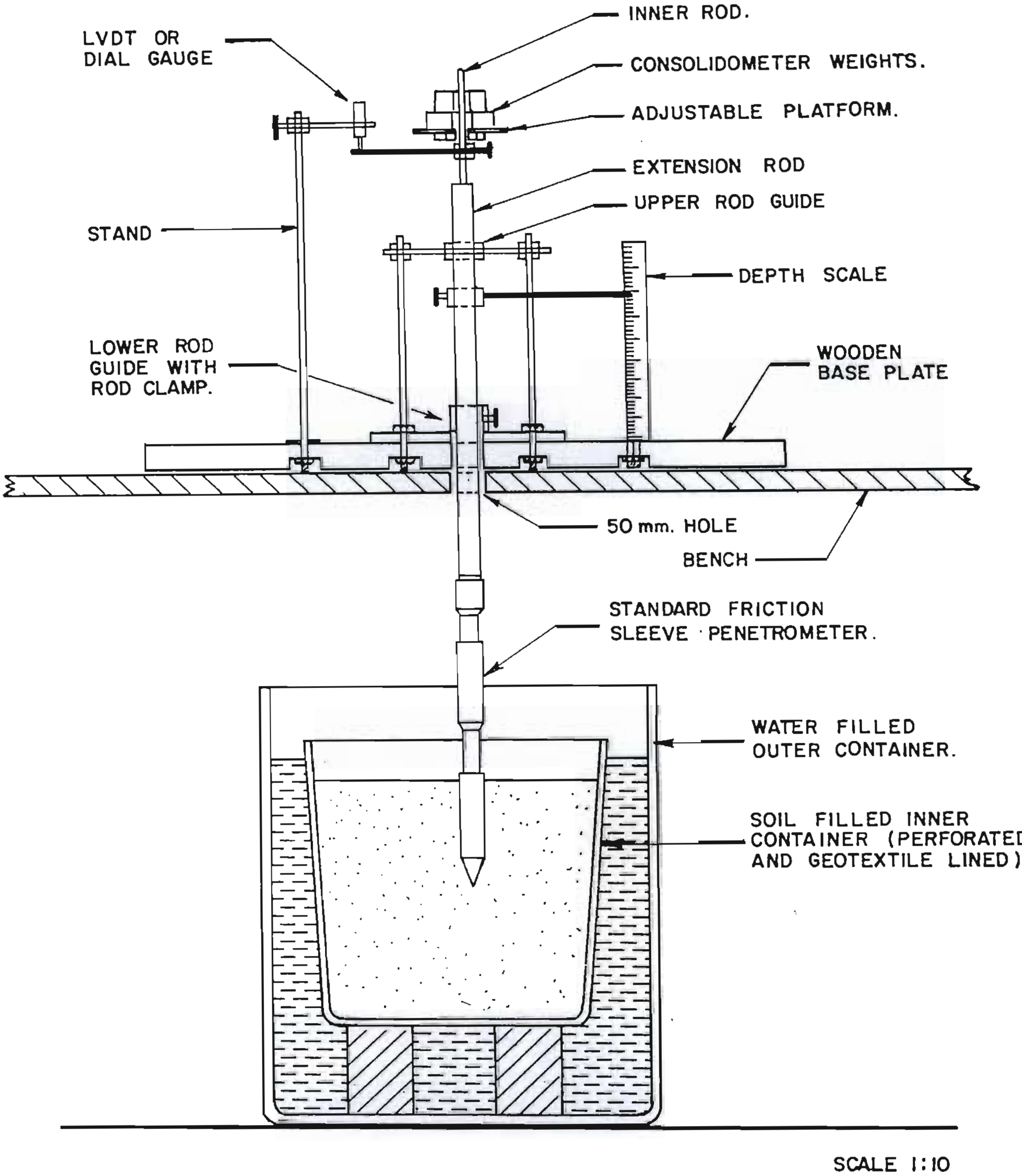
The penetrometer was advanced into the soil and clamped in position so that the cone could be advanced separately. The standard cone inner rod was replaced with a longer inner rod to which a weight platform was fixed. Consolidometer weights were placed concentrically on the load platform and the test thus begun. Standard dial gauges measured movement of the cone. Settlement times were taken in the same manner as for normal consolidometer testing.

It was found that some experience, skill and/or luck was required to select a load which would be sufficient to produce a reasonably measurable movement, but not so much as to induce failure. Readings of settlement against time were taken and plotting these showed that a decay in rate of settlement occurred which was similar to that observed in a consolidometer test.

The equipment was crude and the sample preparation erratic, but the purpose was simply to examine in qualitative terms whether the idea of using the penetration of a cone in a constant stress mode was workable in the laboratory. The initial tests with Prototype 1 showed that it was indeed possible and that some of the results were encouraging. This led to the construction of an improved version of the apparatus.

C4.2.2 **Prototype 2 (Dec 1974 - Jan 1975)**

The apparatus, shown in Figure C4.3 was an improved version of the earlier model. The improvements were more robust ways of clamping the sounding rods to ensure verticality of the casings and inner rod. A secondary depth measuring system was incorporated so that the initial level of the cone could be recorded at the beginning of each test of a series, ie in any soil sample in the container (bucket) a number of tests were conducted at different depths. This measuring system comprised simply a pointer on a vertical scale which did not move during a test since it was clamped to the outer casing. The initial settlement measuring system was by dial gauge, but subsequently an LVDT was incorporated and connected to a chart recorder. This allowed a test to be conducted with very little supervision after successfully being started.
The CBR mould sample container (150 mm diameter) was far too small and was replaced by a much larger plastic container. The soil samples were prepared by compacting layers in the container in which slots had been cut in the sides and drainage holes in the base. The bucket was lined with a filter fabric. During compaction the bucket was circumferentially strapped, using masking tape, to prevent excessive horizontal deformation of the sample and bucket. The full sample height was approximately 400 mm and this was compacted in three or four layers using a standard Proctor moisture-density test hammer. The moisture content was as high as possible during compaction to ensure complete saturation. The number of blows per layer were varied to suit the requirement of the particular test and the soil. Generally, however,



CONSOLIDOMETER – CONE APPARATUS
PROTOTYPE 2

Fig. C·4·3

the compaction effort was relatively small since the purpose of the test was to simulate normally consolidated alluvial soil conditions.

After compaction the strapping was removed by cutting it away at the drainage slots. The bucket was then placed on a stand in a partly filled outer water container. The water level was adjusted to be at the level of the top of the soil in the bucket and the sample left for at least 24 hours for the moisture conditions to stabilize. The assembled soil container and outer water containers were a little lower than laboratory bench height. The upper section of the apparatus was placed in position on the bench centrally over the bucket. A friction sleeve cone which was attached to a half metre standard CPT casing, was clamped in position above the soil surface and the guide yoke with level indicator clamped to the top of the casing. The special long inner rod was clamped to prevent movement of the cone relative to the outer casing.

The cone and sleeve (retracted) and the outer casing were advanced after releasing the lower clamp by manually pushing on the cross yoke so that the cone penetrated into the soil to a distance of about 50 mm, which was to be the first test position. The lower clamp was then tightened.

The load platform was placed over the inner rod and clamped in place at a convenient height for the penetration measuring LVDT and the latter set so that the chart recorder start of test position could be set.

Consolidometer weights were placed on the platform and the test was ready to start. This was achieved by releasing the inner rod clamping screw and simultaneously starting the chart recorder. Penetration was generally allowed to continue until it stopped by itself or until the full travel of the cone, which was 40 mm, was reached. At this point the cone and sleeve mechanism was such that the sleeve engaged and this extra resistance prevented further penetration. The chart was stopped and the LVDT removed. The upper inner rod clamp was tightened preventing movement through the yoke thus holding the cone in the end-of-test position, and the lower clamp released so that the outer casings and sleeve could be advanced to catch up to the cone. The lower inner rod clamp was then tightened, ie with the cone and sleeve then retracted, and the yoke manually pushed to advance the cone sleeve and casing to a new test position. This was generally chosen as 50 mm lower than the first position and then the sequence was repeated viz clamp the outer casing; set the LVDT and chart; note the depth scale reading; release the inner rod clamp, start the chart and check to see if any movement had occurred. It was observed that generally as the depth increased, a higher load was required to initiate a satisfactory amount of penetration for the load test.

Tests were continued to close to the full depth of the sample, generally 5 or 6 tests at 50 mm intervals on each bucket sample. On completion of the tests the upper section of equipment was removed after first unloading and unclamping all the rods and casings and withdrawing them.

The outer container was drained to the base of the sample and the bucket removed. Consolidometer samples were cut from the soil sample in the annular space about half way between the disturbed centre and the outside edge, by digging to the appropriate depth and pushing standard consolidometer cutting rings. Consolidometer tests were carried out on all these samples and the coefficients of consolidation obtained in the usual way.

The LVDT chart records were processed by measuring the chart displacements at various time intervals. In this way data sets were obtained of compression at various times to a final value which was defined as that at which settlement appeared to have stopped on the chart. The time for this varied with the type of soil and was in the range of an hour to twenty four hours, ie similar to the times required for conventional consolidation testing for each load increment.

C4.2.3 Analysis of cone settlement

The modelling of cone penetration is complex in a steady state situation. To attempt to model penetration at a decaying rate and relate this to the conventional consolidation parameters for the soil was considered to be inappropriate for a prototype test method. Some form of interpretation of the test data was however necessary to evaluate the potential of the method. The overall principle adopted, as implied in the foregoing description of the test procedure, was to directly compare the rate of consolidation in the consolidometer with the rate of settlement in the cone test.

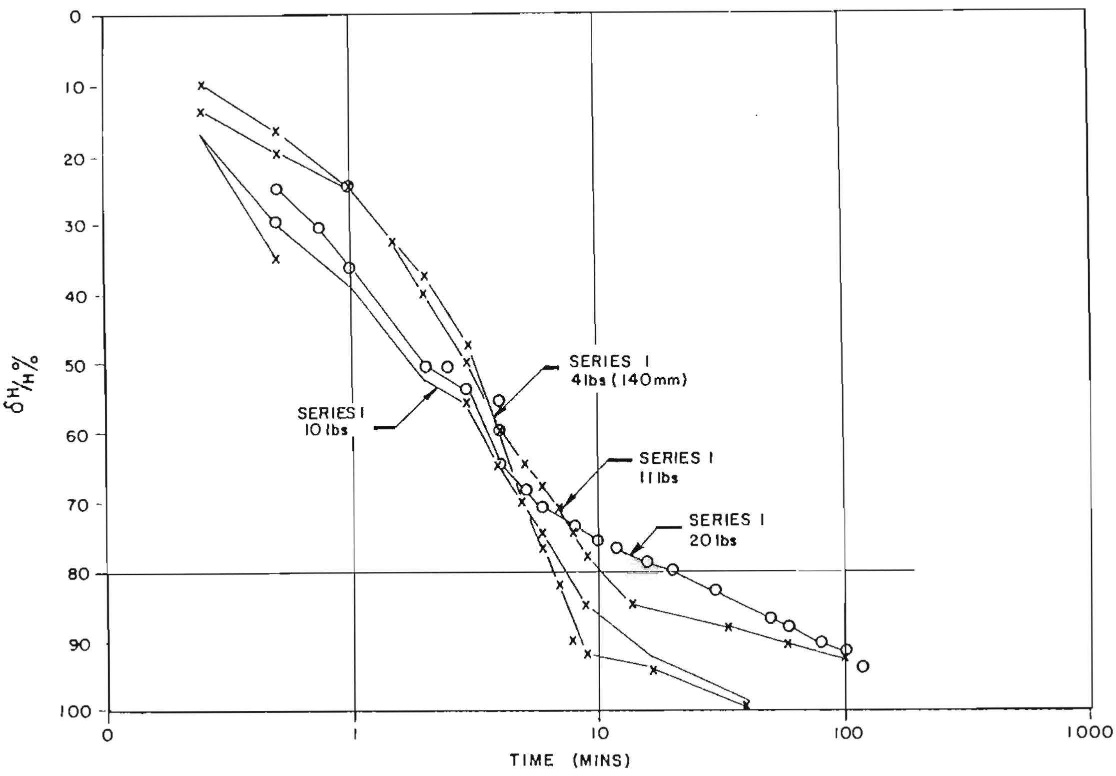
The rate was defined as the ratio of the settlement after any time interval, δH, to the final settlement, H and plotted against time on a logarithmic scale.

Typical results of the early cone penetration tests are shown in Figures C4.4 and C4.5. As with laboratory consolidation test data there is some difficulty with defining both the beginnings and ends of the plots and it is necessary to use a consistent method to determine these. The same graphical construction as that given by Casagrande and described by Taylor (1948) for consolidation tests was utilized, and from this, end of primary settlements, and hence degrees of consolidation could be calculated. These are plotted as degree of consolidation against log time on Figure C4.6 It can be seen that the results appear to be similar to those for laboratory consolidation tests for which theoretical curves for a range of coefficients of consolidation, c_v, are shown in Figure C4.7.

An overall way of describing the consolidation characteristic for the laboratory tests and the settlement characteristics for the cone tests is to use the time at 50% consolidation t_{50}.

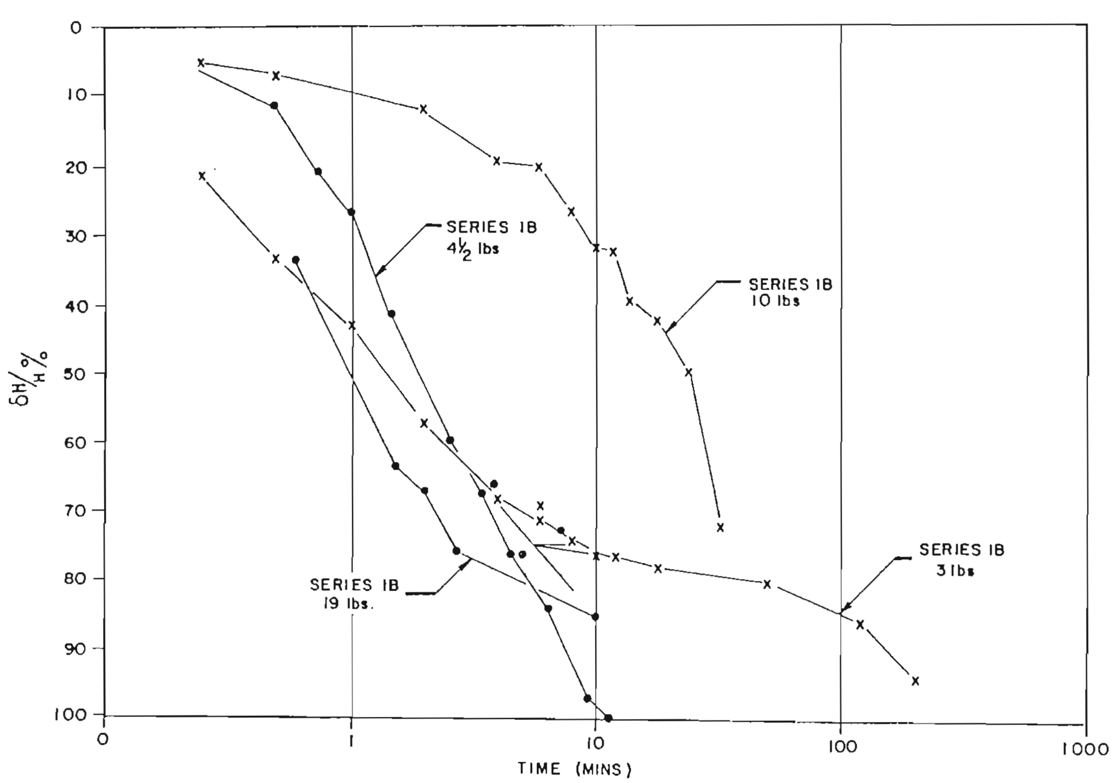
The results for the cone tested samples showed an increase in t_{50} with increasing clay content, and also that the t_{50} values for the cone tests were similar in magnitude to those for the laboratory consolidation tests.

At the time, being conscious that the consolidometer-cone test only superficially resembled a laboratory consolidation test, the outcome of the early cone testing shown in Figures C4.6 and C4.7 was most gratifying, and was sufficient to justify building a third laboratory prototype test rig.



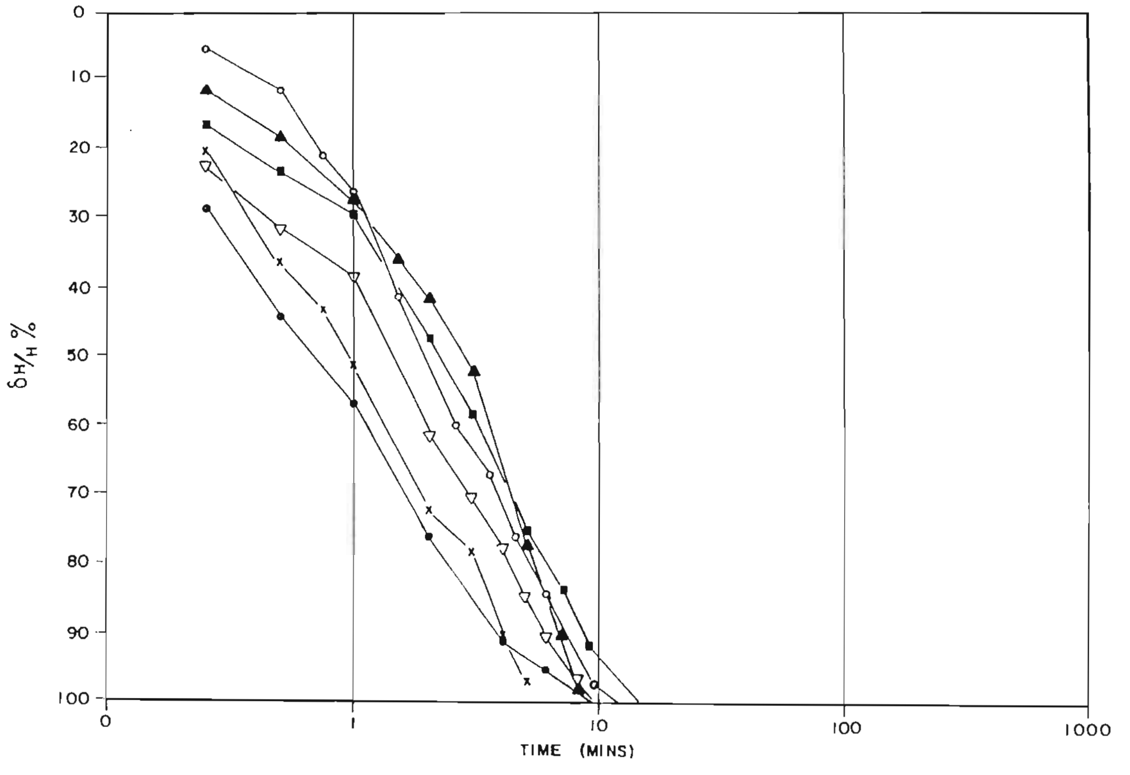
LABORATORY CONSOLIDOMETER- CONE
TIME-SETTLEMENT TESTS
LPC SERIES 1.

Fig. C·4·4



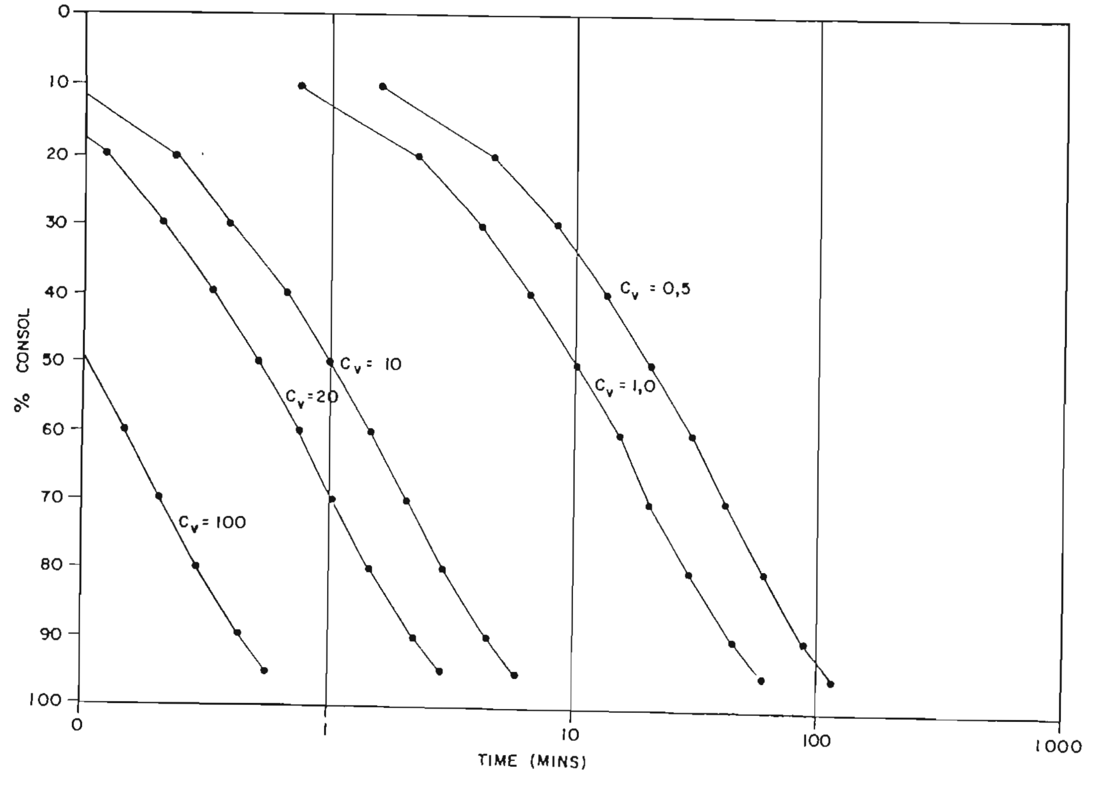
LABORATORY CONSOLIDOMETER- CONE
TIME -SETTLEMENT TESTS
LPC SERIES 1B

Fig. C·4·5



LABORATORY CONSOLIDOMETER- CONE
TIME-SETTLEMENT TESTS
LPC

Fig. C·4·6



LABORATORY CONSOLIDOMETER THEORETICAL
TIME-SETTLEMENT CURVES

Fig. C·4·7

C4.2.4 Prototype 3 (May 1975 - June 1976)

The test equipment was a development of Prototype 2 as shown by Figure C4.8. The improvement was primarily that the cone could be advanced to a new testing position in a controlled way by a screw jack since the earlier equipment necessitated manually pushing the cone and penetrometer assembly which was difficult to control.

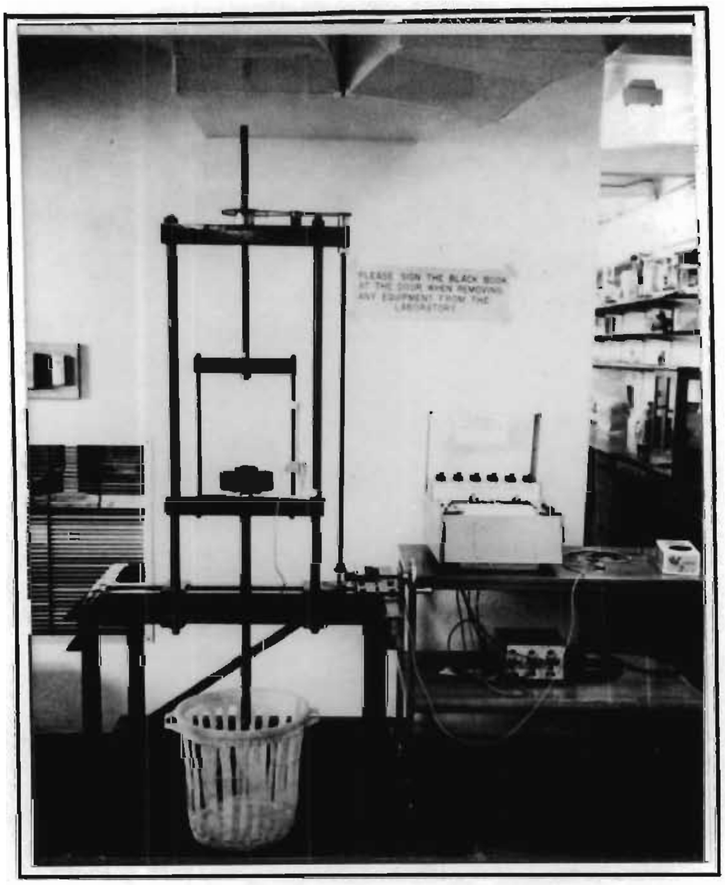


Figure C4.8 : Consolidometer-cone apparatus : Prototype 3

The modifications also had the advantage of simplifying the procedure since once the outer casings were clamped in the main yoke no further unclamping of them was required. The inner rods however needed clamping when the loads were placed on the load platform.

The system was built early in 1975 and was used until the middle of the following year. Numerous test were conducted on a range of materials. The method of preparation of the soil samples was not changed from that described for Prototype 2, and the same type of bucket and outer water container were used. It is noted that the purpose of the testing was to show that the cone in the proposed constant total stress mode of operation measured characteristics that could be qualitatively related to consolidation. It was fully appreciated that quantitative correlations would be limited by the sample size and preparation. It was the intention that if this initial project was successful then field equipment would be justified. If this too was satisfactory, then field testing would be necessary. This would include undisturbed sampling and laboratory consolidation testing so that correlations with the field consolidometer-cone test data could be made.

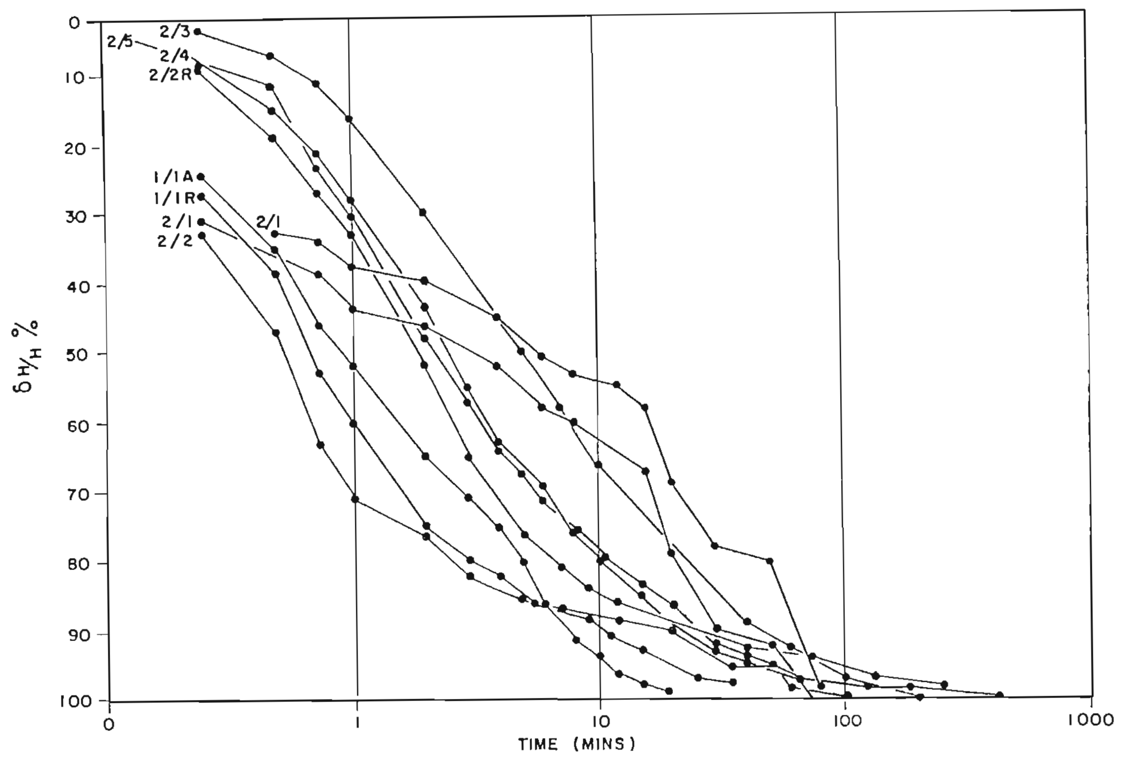
C4.3 Analysis and Discussion of Test Results

Time-settlement data was recorded on a chart recorder and from these records settlements against the logarithm of time were obtained. End of primary consolidation time was assessed and settlements were expressed as degree of consolidation, against time to a logarithmic scale. The cone data was directly compared to laboratory consolidation test data on undisturbed samples from the soil container.

A range of sample types was tested and results are given in Figures C4.9, C4.10, C4.11, C4.12 and C4.13. These varied from laboratory remoulded samples of silty sands (TSPC) and clayey sand (TSSH) to large intact clay samples. The latter were obtained by carefully excavating with backactor large clay samples from a site at Umhlangane in the Sea Cow Lake area of Durban. The site was chosen because a major site investigation was taking place there for embankments for the proposed Durban Outer Ring Road (Jones et al, 1975). The samples were wrapped in plastic sheeting and filter fabric, supported in boxes, carefully transported to the laboratory (NITRR Pretoria) and stored in a humid room.

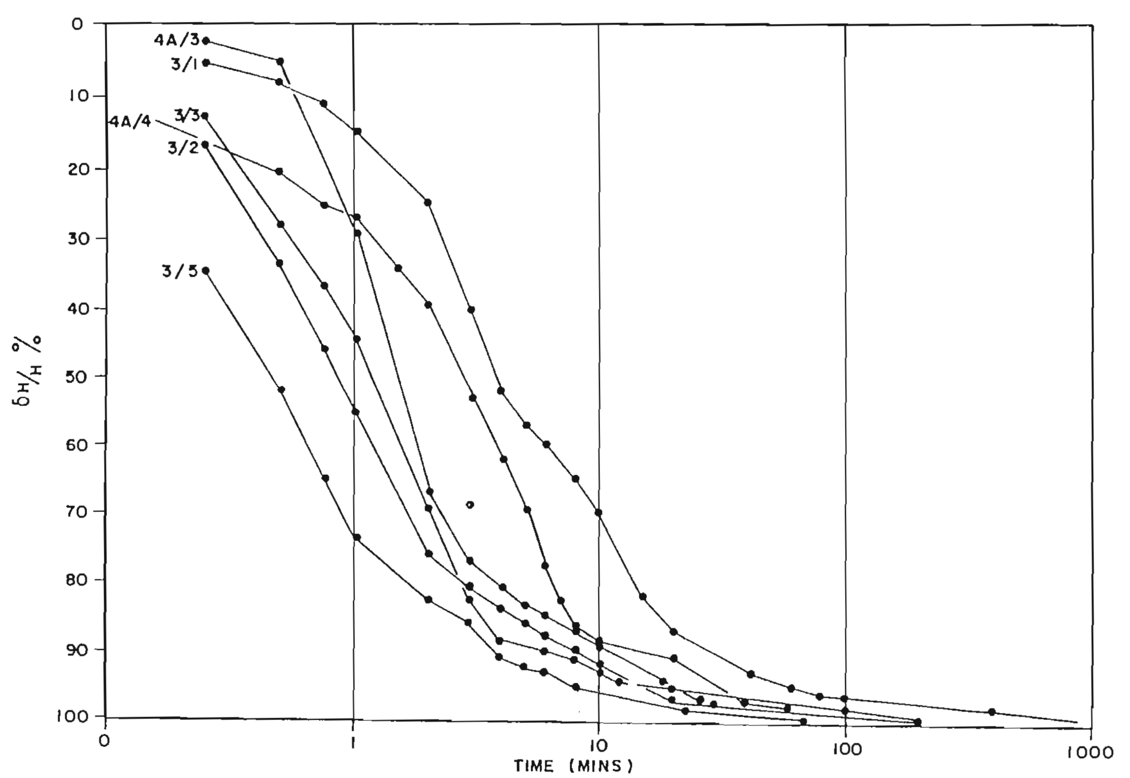
Samples were hand trimmed to bucket shape, the filter fabric-lined inverted bucket placed over them and re-inverted. To minimize drying out the sample was only trimmed to correct level when testing was due to start. Consolidometer samples were subsequently taken from the bucket and a check on disturbance was possible through detailed examination of the consolidation test data.

Figures C4.13 and C4.14 show the results of consolidometer-cone and consolidation tests carried out on the Umhlangane Sea Cow Lake clays. The latter showed no unusual characteristics and the compressibility and consolidation parameters were similar to those obtained in the testing for the site investigation.



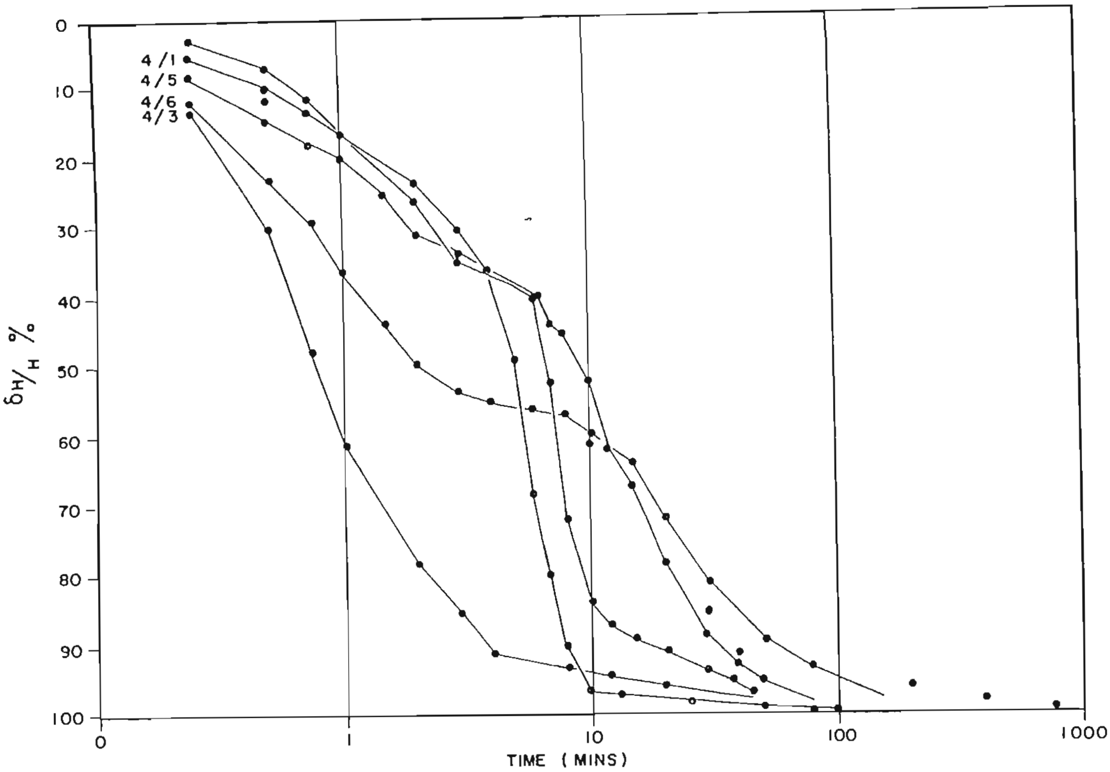
LABORATORY CONSOLIDOMETER-CONE TIME-SETTLEMENT TESTS FOR TSPC SERIES I AND 2

Fig. C·4·9



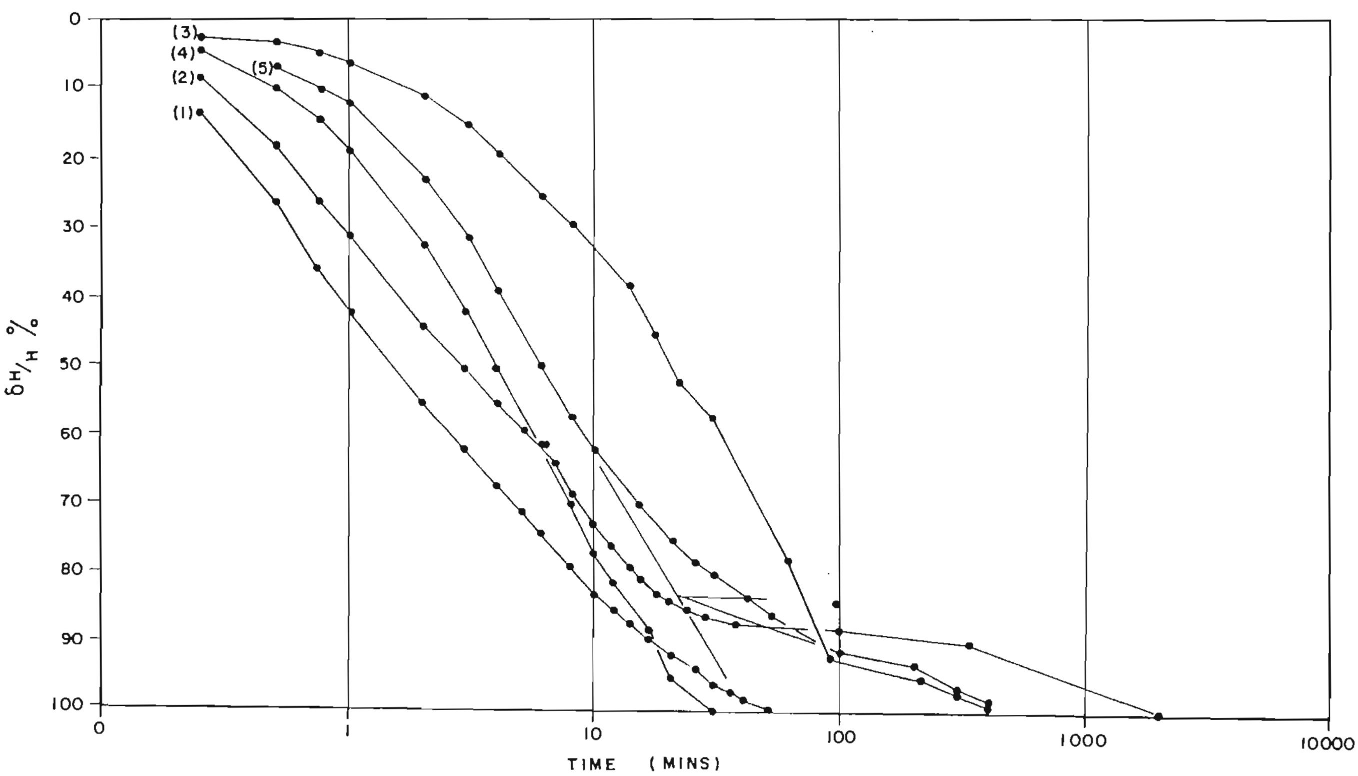
LABORATORY CONSOLIDOMETER-CONE TIME-SETTLEMENT TESTS FOR TSPC SERIES 3 AND 4A

Fig. C·4·10



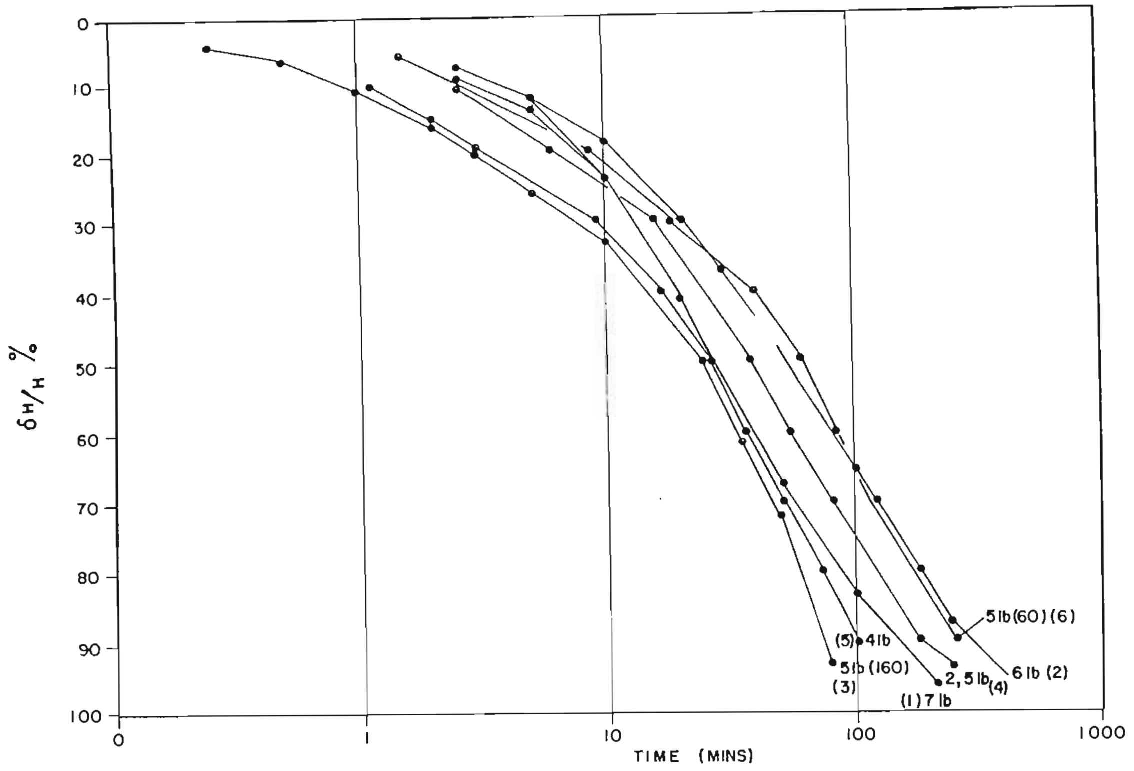
LABORATORY CONSOLIDOMETER-CONE TIME-SETTLEMENT TESTS FOR TSPC SERIES 4

Fig. C·4·11



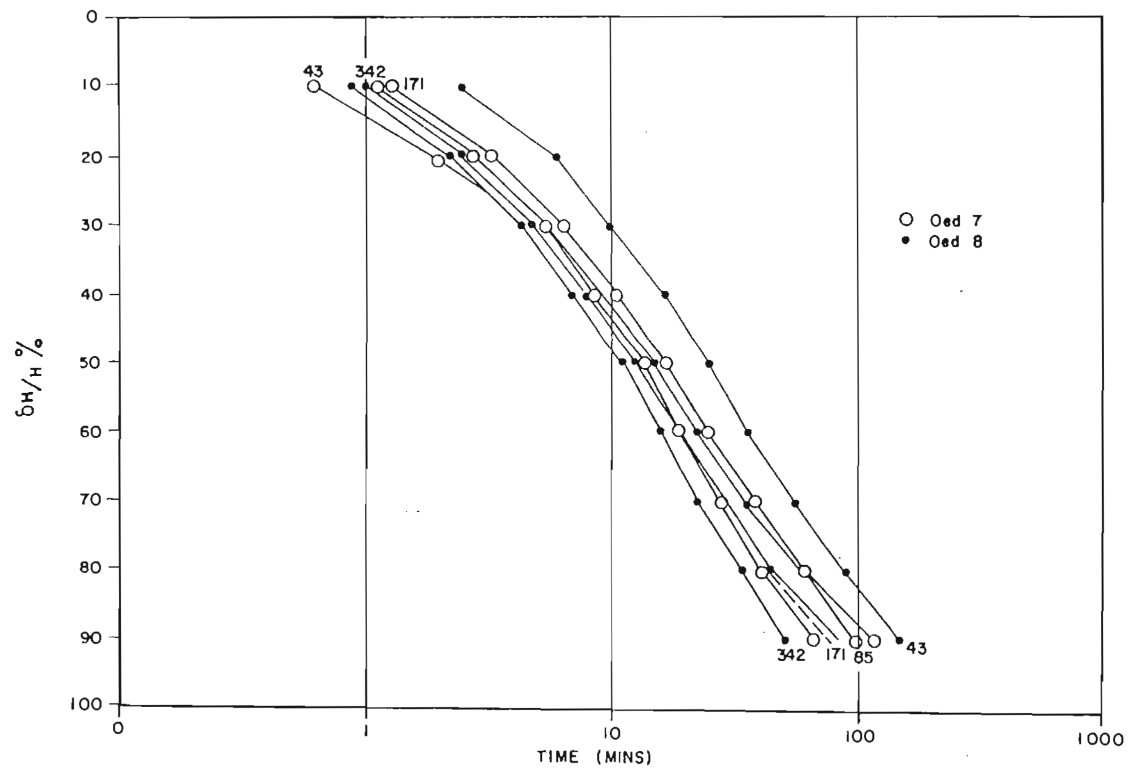
LABORATORY CONSOLIDOMETER-CONE TIME-SETTLEMENT TESTS FOR TSSH

Fig. C·4·12



LABORATORY CONSOLIDOMETER-CONE TIME-SETTLEMENT TESTS FOR SEA COW LAKE

Fig. C·4·13



LABORATORY CONSOLIDOMETER TIME-SETTLEMENT TESTS FOR SEA COW LAKE

Fig. C·4·14

It should be noted that the end of primary consolidation was determined from the test results graphicallly in the usual way (Taylor, 1948) and defined as H : Figures C4.13 and C4.14 are plotted as settlements expressed as a percentage of primary consolidation, H, ie up to 90% and therefore do not show marked changes in slope at the tails.

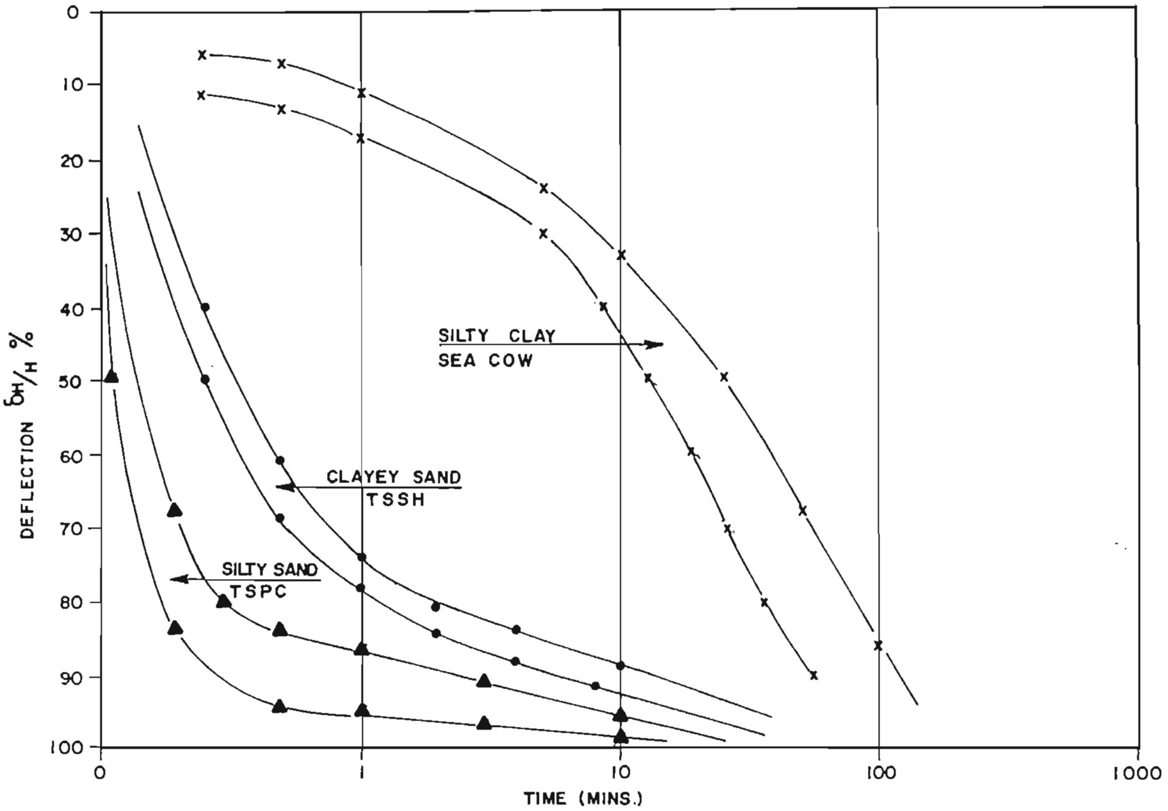
The data sets for the laboratory consolidation tests and for the cone tests for the three soil types are shown in Figures C4.15 and C4.16 as degree of consolidation against log time. It is clear that there is a marked similarity in the form of the results for the consolidometer and consolidometer-cone tests. This was more than encouraging although not unexpected since the earlier prototype testing had led to this position.

The results can be expressed as t_{50} for both consolidometer-cone and consolidation tests after correcting the time-settlement results to eliminate creep or secondary consolidation. The final t_{50} are given in the Table C4.1, together with the grading, hydrometer and Atterberg Limits.

Table C4.1 : Particle size, Atterberg limits and cone t_{50} for consolidometer-cone tests

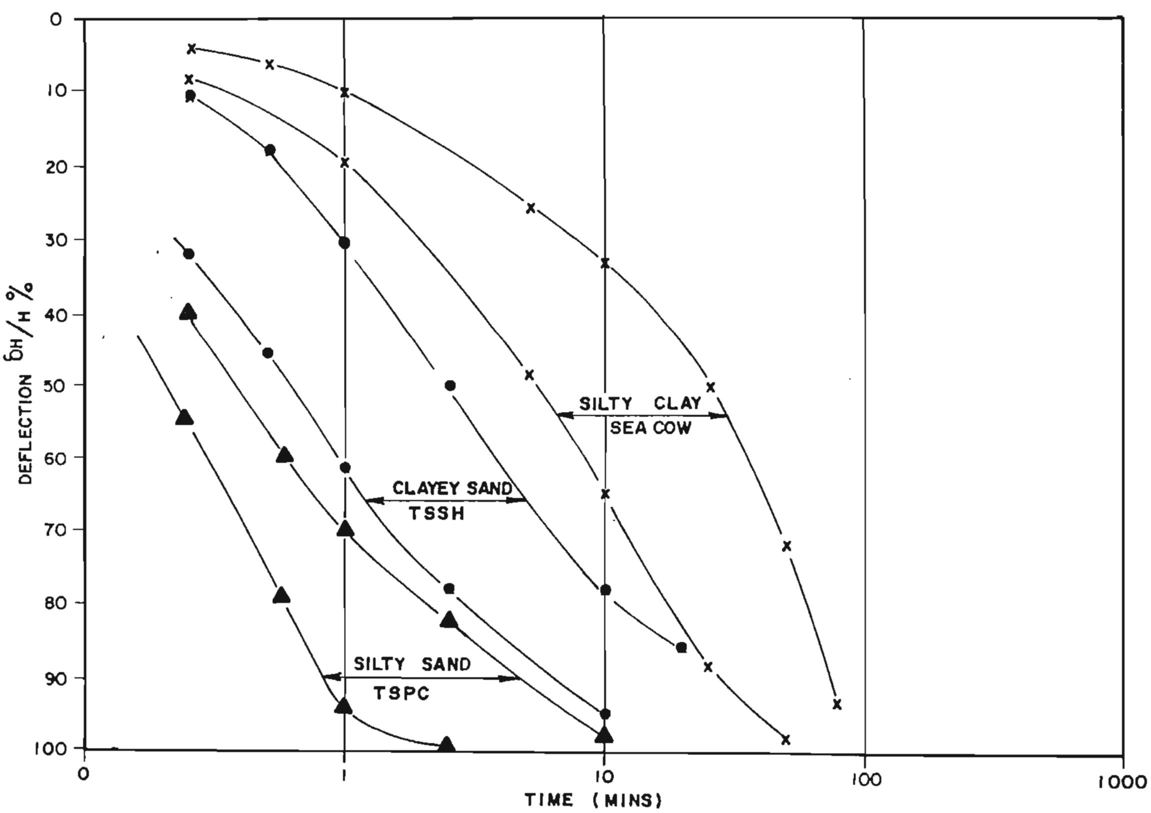
Test Series	Particle Size μm			Atterberg Limits		Cone t_{50} Averages mins
	< 2	2 - 60	> 60	w_L - w_p	I_p	
Sea Cow	52	35	13	56-35	21	39
TSSH	23	19	58	40-18	22	3,6
TSPC	17	16	67	32-20	12	2,5
LPC	-	7	93	NP		2,5 ?

From Table C4.1 and Figures C4.15 and C4.16 it can be seen that there was a qualitative relationship between the consolidometer-cone t_{50} and the consolidometer t_{50} and between the former and the soil type. The derivation of a more closely defined relationship on the basis of the evidence would however, not be justified since there was a large gap in the data in the mid range. Nevertheless the correlation for the Sea Cow Lake clay (and it is the more clayey materials which give rise to embankment problems) was sufficiently close to warrant further investigation.



LABORATORY CONSOLIDOMETER TIME-SETTLEMENT TESTS FOR RANGE OF SOIL TYPES

Fig. C·4·15



LABORATORY CONSOLIDOMETER-CONE TIME-SETTLEMENT TESTS FOR RANGE OF SOIL TYPES

Fig. C·4·16

As shown in Figure C4.1 the consolidometer-cone could be seen as a plate loading test approximately half the diameter of a conventional consolidometer. Whatever the shape of the volume of soil significantly stressed under the cone, ie sphere, hemisphere or cylinder, it is apparent that the drainage path length for consolidation is a function of the radius of the cone (17,5 mm). Since the consolidometer drainage path length for double drainage is 10 mm, it could be expected that consolidation times for the consolidometer-cone would be about triple those for the consolidometer. Also a significant part of the soil stressed, ie that next to the cone, has a much shorter drainage path length so that consolidation times of less than triple those for the consolidometer might be expected.

For the clay sample the average values for the t_{50} and t_{90} times for both the consolidometer and the consolidometer-cone are given in Table C4.2.

Table C4.2 : Consolidometer and consolidometer-cone t_{50} and t_{90} times for Sea Cow Lake

	t_{50} mins	t_{90} mins	av t_{90}/t_{50}
Consol Sample 7	10; 15; 13; 10; 11	70; 105; 65; 45; 45	5,52
Sample 8	21; 26; 17; 17; 13	100; 140; 90; 100; 65	5,26
Cone	24; 25; 26	74; 100; 150	4,5
	37; 52; 60	180; 250; 290	
Cone/Consol	2,44	2,11	

The consolidometer results for the Sea Cow Lake clay in Table C4.2 are shown in Figure C4.14. Since the ratio of the t_{90} and t_{50} times should be the same as the ratio of the time factors T_v ie 0,848 to 0,197 = 4,30 this would suggest that the estimated t_{90} times for the consolidometer tests given in the Table are probably too high and include some secondary compression.

Unquestionably the consolidation test results indicate a significant secondary compression component and because the test times were limited to 24 hours for each load increment, the estimation of c_α, the coefficient of secondary compression, and hence t_{100} is relatively inaccurate. Selection of different slopes for c_α on the detailed laboratory test results gives a range of t_{90} from 100 to 150 minutes and the corresponding range for t_{50} is 21 to 23 minutes, hence the t_{90}/t_{50} changes from 4,7 to 6,5. Therefore although the average measured ratio in the Table is about 5,4 compared with the theoretical value of 4,30, this difference is not believed to be significant, since it is within the range of accuracy of the measurements from the consolidometer data.

The corresponding ratio for the cone tests is 4,7 which suggests that the slope of the settlement - log time plots matches closely the slope for the theoretical one dimensional consolidation. Since consolidation around the cone must be three dimensional the ratio of T_v at δ_{90} to that at δ_{50} would be expected to be somewhat higher; for example if Blight's (1968) solution for the consolidation of a sphere is considered applicable to the cone, then the ratio of T_{v90} to T_{v50} is about 1,35 to 0,21 viz 6,4; however, no significant inference was drawn from this anomaly.

For the clay the cone tests give t_{50} and t_{90} about 2,4 and 2,1 times longer than those for the consolidometers - Table C4.2.

Since the time factors, T_v for the consolidometer T_{vo} and the cone T_{vc} as a sphere are approximately the same at t_{50} (0,197 and 0,21) then from :

$$C_v = \frac{T_v\, a^2}{t} \qquad \text{C4.1}$$

$$\frac{T_{vo}\, d^2}{t_{50}} = \frac{T_{vc}\, a^2}{2{,}4\; t_{c50}} \qquad \text{C4.2}$$

$$\text{then } a = \left(2{,}4 \times \frac{0{,}197}{0{,}21} \times 100\right)^{\frac{1}{2}}$$

$$= 15 \text{ mm}$$

ie the equivalent sphere has a radius 15 mm, a little less than the radius of its generating cone, 17,5 mm.

This analysis demonstrates that the volume of soil consolidating under the cone is about the same as a sphere of the same radius as the cone. Since the cone t_{50} and t_{90} times are about two and a half times longer than the equivalent consolidometer times then :

$$\text{for } t_{50} \quad c_v = \frac{0{,}2 \times 0{,}01^2 \times 60 \times 24 \times 365 \text{ m}^2/\text{yr}}{t_{50}/2{,}5}$$

$$c_v = \frac{26}{t_{c50}}\text{m}^2/\text{yr} \qquad \text{C4.3}$$

where t_{c50} = time in minutes for cone half settlement

$$\text{Similarly for } t_{90} \quad c_v = \frac{110}{t_{c90}}\text{m}^2/\text{yr} \qquad \text{C4.4}$$

where t_{c9} = time in minutes for cone 90% settlement

The foregoing results were very encouraging and tended to confirm that the cone in the constant stress mode could be used to measure the consolidation properties of clay.

However when the same approach was adopted for the tests on the other soils a less encouraging picture emerged.

The soil type TSSH results are examined first because this series was conducted in the Prototype 3 apparatus which was significantly superior to the previous prototypes.

The relevant test results are given in Table C4.3.

Table C4.3 : Consolidometer and consolidometer-cone t_{50} and t_{90} times for TSSH

Sample No	Depth mm	Consol		Cone		Run No	Load
		t_{50} (min)	t_{90} (min)	t_{50} (min)	t_{90} (min)		
1		0,19	0,68	1,5	17	1	1
2	90	0,17	0,94	-	-	-	-
3	150	0,17	0,66	20	30	2	2
4	190	0,17	0,62	20	80	3	3
5	210	0,20	1,04	4,0	17	4	7
6	250	0,18	0,62	6,0	80	5	11
		0,18	0,77	3,6	36		
t_{90}/t_{50}		4,3		10			
cone/consol		t_{50} = 20		t_{90} = 47			

It is apparent that the various ratios given in the Table are very different from those for the clay samples (Table C4.2). The consolidometer showed very short times for settlement and in almost all cases the t_{90} times were less than 1 minute. Since measurements had only been started at 0,5 min the estimates of t_{50} can only be considered as very approximate, nevertheless the $t_{90} : t_{50}$ ratio of 4,3 is the same as the theoretical ratio for one dimensional consolidation and suggests that the samples have undergone consolidation.

For the consolidometer-cone the corresponding $t_{90} : t_{50}$ is 10. Re-examination of the cone plots, Figure C4.12, however, shows that some of the results exhibit significant secondary compression or creep and this has a marked influence on the $t_{90} : t_{50}$. The cone results were modified to determine a revised t_{100} and the t_{50} and t_{90} times are given in Table C4.4.

Table C4.4 : Modified consolidometer-cone t_{50} and t_{90} times for TSSH

	t_{50} min					t_{90} min				
Run No	1	2	3	4	5	1	2	3	4	5
Load	1	2	3	7	11	1	2	3	7	11
Original	1,5	3,0	20	4,0	6,0	17	30	80	17	80
Revised	1,2	1,9	17	3,6	4,5	9	13	80	15	20
Average t	2,8 (excl Run 3)					14,25 (excl Run 3)				
t_{90}/t_{50}	7,5	6,8	47	4,2	4,4	Average 5,5				

It can be seen that the revision makes a considerable difference particularly to the cone t_{90} and hence the ratios in Table C4.3 should be modified.

The cone $t_{90} : t_{50}$ becomes 5,5 (excl Run 3); the cone : consol t_{50} ratio becomes 16 and the t_{90} ratio becomes 18. The modification to the cone results therefore leads to consistent ratios for the TSSH soil, but despite this improvement these ratios are dissimilar to those for the Sea Cow Lake samples. These are compared in Table C4.5.

Table C4.5 : **Comparison of cone / consolidometer ratios for t_{50} and t_{90} for TSSH and Sea Cow Lake**

Soil	Cone/Consolidometer Ratios	
	t_{50}	t_{90}
Sea Cow Lake	2,4	2,1
TSSH	16	18

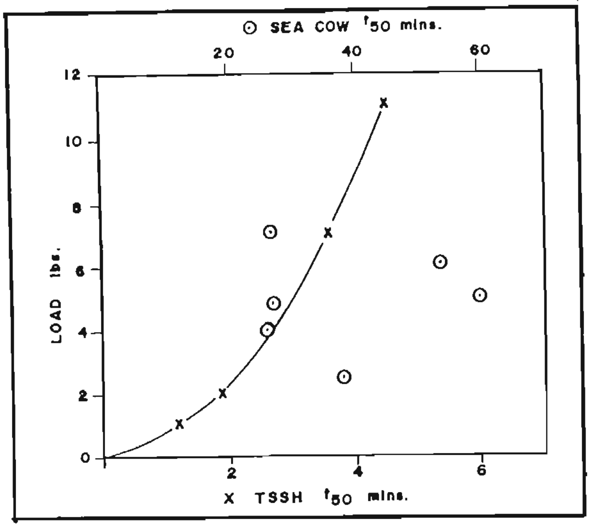
It was argued previously that the cone/consolidometer Sea Cow Lake soil ratios of about 2 - 2,5 were what might be expected from the relative drainage path lengths for the consolidometer-cone and consolidometer viz 17,5 mm and 10 mm. The TSSH soil gave very different values and by the same line of argument as before the implied drainage path length for the cone was :

$$a = \left(16 \times \frac{0{,}197}{0{,}21} \times 100\right)^{\frac{1}{2}}$$

$$= 39 \text{ mm}$$

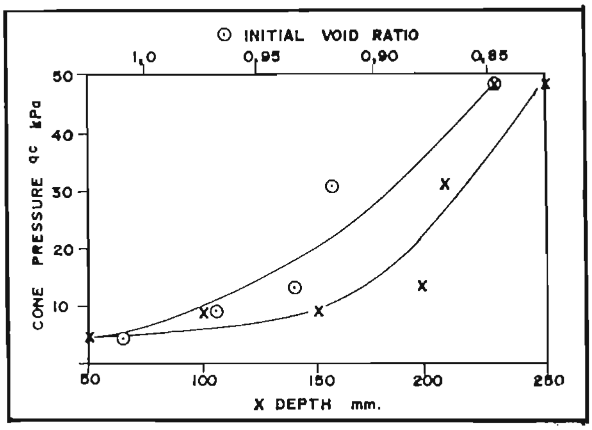
There was no obvious explanation for this discrepancy; the stress levels in both the Sea Cow Lake and the TSSH series of tests were similar, the former loads being in the range $2^1/_2$ - 7 lbs and the latter 1 - 11 lbs.

The Sea Cow Lake tests showed no correlation between stress and t_{50} whereas the TSSH showed a consistent, although non linear, relationship - Figure C4.17. Tests on the latter also showed - Figure C4.18 - that the required cone pressure q_c appeared to be a function of the test depth and a similar, but somewhat less marked, trend was apparent for the q_c and void ratio relationship. However the initial voids ratios for the TSSH samples were also a function of depth - Figure C4.19 - since the samples were prepared in compacted layers in the container. It would be expected that for this laboratory prepared clayey silty sand the permeability would be directly related to the initial void ratio. This is shown in the laboratory consolidometer test results when void ratio is plotted against t_{50}, Figure C4.20.



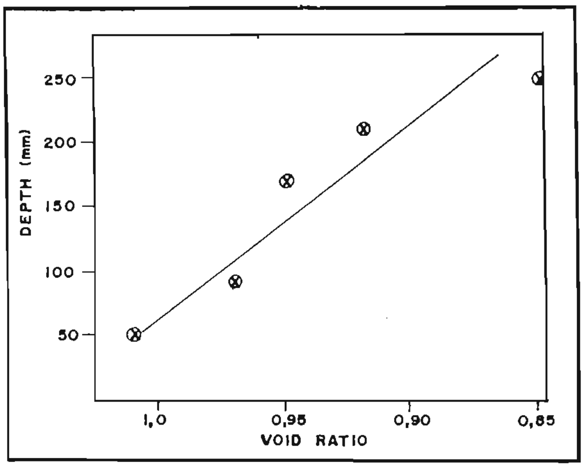
LOAD AGAINST t_{50} FOR SEA COW AND TSSH.

Fig. C·4·17



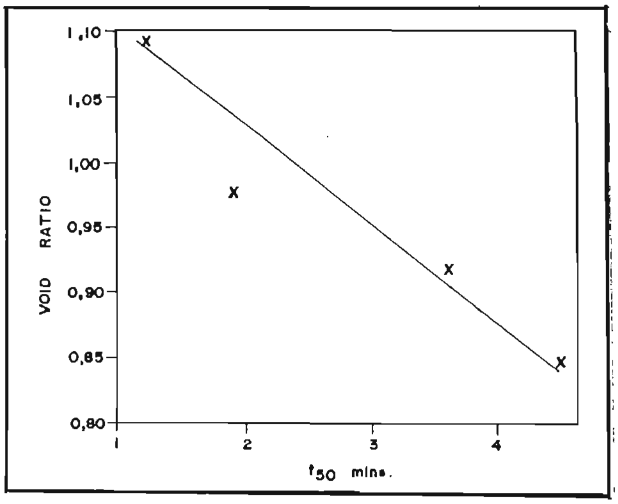
SAMPLE TSSH:
q_c AGAINST VOID RATIO AND DEPTH.

Fig. C·4·18



SAMPLE TSSH:
VOID RATIO AGAINST DEPTH.

Fig. C·4·19



SAMPLE TSSH:
VOID RATIO AGAINST t_{50}

Fig. C·4·20

Detailed consideration of test series, LPC and TSPC, show that the results were similar to those for the TSSH series.

The LPC was a non plastic silty sand which had initial laboratory void ratios in the range 0,48 - 0,62; the consolidometer t_{50} were in the range of about 0,10 - 0,20 minutes and t_{90} from 0,5 - 1,0 minutes. The cone t_{50} were in the range of about 1 - $2^{1}/_{2}$ minutes and t_{90} in the range 5 - 10 minutes. The consolidometer-cone testing equipment was in the initial stages for the LPC series and was relatively crude, nevertheless the results are similar to those for TSSH. The required load to induce controlled settlement appeared to be a function of sample depth but from the data it could not be ascertained whether the t_{50} or t_{90} times were also dependent on depth.

Sample TSPC was a slightly clayey silty sand. The initial laboratory void ratios were in the range 0,72 - 1,07 and were a function of depth. The consolidometer t_{50} were in the range of about 0,2 - 0,5 minutes and the t_{90} from about 0,5 - 2 minutes. The consolidometer-cone t_{50} were in the range 0,5 - 9 minutes with an average of 2,5 minutes and the t_{90} in the range 4 - 50 minutes with an average of 20 minutes. The latter was significantly reduced by correcting the plots for excessive creep as was done for the TSSH results, resulting in an average cone t_{50} of 2 minutes and t_{90} of 12 minutes.

The LPC and TSPC results are similar to those for the TSSH series and the more pertinent ratios are given in Table C4.6.

Table C4.6 : Consolidometer and consolidometer-cone times and ratios for all samples

Test		Average Time (Minutes)			
		LPC	TSPC	TSSH	Sea Cow
Consol	t_{50}	0,15	0,2	0,18	15
	t_{90}	0,75	0,80	0,77	81
	t_{90}/t_{50}	5,0	4,0	4,3	5,4
Cone	t_{50}	1,5	2,0	2,8	37
	t_{90}	7,0	12	14,2	174
	t_{90}/t_{50}	4,7	6,0	5,1	4,6
Cone/consol	t_{50}	10	10	16	2,4
Cone/consol	t_{90}	9	15	18	2,1

The overall picture from the comparison of the results of the silty and clayey sands with those of the Sea Cow Lake clay is that the clay results appear to be consistent with the idea of the cone acting as a form of consolidometer and thus measuring consolidation characteristics, whereas in the sandier materials the same simplistic model does not apply since the cone/consolidometer time ratios are much higher than the expected value of about 2 to 2,5.

Re-examination of the results showed that although the applied pressures were similar for all the tests, the resulting strains were very different.

Typical measured chart displacements together with the actual LVDT settlements are given in Table C4.7.

Table C4.7 : Measured strains for consolidometer-cone tests

Sample	No of Runs	Chart Distance mm		Cone Pressure kPa	Settlement Cone mm
		Range	Average		
LPC	8	97-280	170	40	10,6
TSPC	21	23-224	88	55	11,0
TSSH	5	133-288	229	20	14,3
Sea Cow	7	12-49	27	21	2,7

The measured consolidometer settlements for the pressure range equivalent to the cone test pressures; the coefficients of volume compressibility, m_v, (in the pressure range 50 150 kPa); the compression indices C_c, (in the 50 - 100 kPa range) and the initial void ratios are given in Table C4.8.

Table C4.8 : Consolidometer test results

Sample	Initial Void Ratio	m_v^* MN/m^2	δH mm	C_c
LPC 1 + 2	0,55	0,08	0,16	0,025
LPC 3	0,60	0,11	0,21	0,034
TSPC	0,88	0,35	0,65	0,16
TSSH	0,97	0,57	0,75	0,20
Sea Cow	2,72	0,61	0,6	0,50

* for equivalent pressure range to cone tests

A comparison of the settlements in the consolidometer tests, for the pressure range equivalent to the cone tests, with the settlements for the corresponding consolidometer-cone tests is shown in Table C4.9 as the ratio of consolidometer-cone to consolidometer settlements.

Table C4.9 : Cone : consolidometer settlement ratios

Material	Settlement Ratio Cone/Consol
LPC	66,2
TSPC	16,9
TSSH	19,1
Sea Cow	4,3

It was clear that the cone settlements in the more sandy materials were very much larger than the equivalent consolidometer settlements, whereas for the clay the ratio was about 4,3. The latter was reasonable if the cone was considered to influence a sphere of approximately the same diameter as the cone ie 35 mm compared to the consolidometer sample thickness of 19 mm and that the lateral constraint conditions for the cone and consolidometer were different.

For all the other samples, however, the relative strains were inordinately high and there could be only one explanation. It was that for these materials the consolidometer-cone test was not measuring consolidation. The erratic nature of many of the earlier tests in particular had suggested that a failure situation was being developed. With the improved equipment, however, the results for the TSSH soil were consistent and during testing gave rise to the hope that they could be usefully interpreted in terms of consolidation. To paraphrase the words of the French general witnessing the Charge of the Light Brigade, "C'est magnifique, mais ce n'est pas la consolidation (guerre)". It is worthwhile giving some consideration to these results; they must represent a bearing capacity failure of a buried foundation and Figure C4.18 shows the ultimate bearing pressures against depth, as well as against initial void ratios of samples subsequently taken at or close to the cone test depths. Figure C4.19 shows the correlation of void ratio with depth the form of which is to be expected since the sample was compacted in layers and it is apparent that the governing factor for the bearing capacity is not only the depth, but also the initial void ratio of the sample.

In other words cone penetration is increasingly resisted by the increasing in situ stresses with depth and by the increasing soil density with depth, and the overall result is a decreasing rate of failure. The actual cone movements in these materials is about 10 - 15 mm which is very large compared with the diameter of the cone, viz 35 mm.

Although this re-appraisal of the proposed consolidometer-cone system produced mixed conclusions, namely that in sands no useful results could be obtained, but in clays the results were encouraging, the latter justified the continuation of the work. The ultimate purpose was for in situ testing of clays in which the times for embankment consolidation were expected to be large. In such materials the values of the coefficients of consolidation, c_v, at which practical problems of slow settlements would be expected to start becoming significant, would be in the range of about 5 - 20 m^2/year. The sandy materials which were tested in the laboratory have coefficients of consolidation of about 50 m^2/year and the Sea Cow Lake clay has a c_v of about 0,5 - 1 m^2/year. The range of materials selected for the laboratory programme was too wide and intermediate materials were required. The results, however, emphasized the more significant problem of whether or not the cone test was measuring consolidation settlement or shear strain, or a variable combination of both.

In order to test this the author conceived the idea of installing a piezometer in the cone to measure the rate of dissipation of pore pressure during settlement of the cone.

The following section, C5, describes the laboratory piezometer cone.

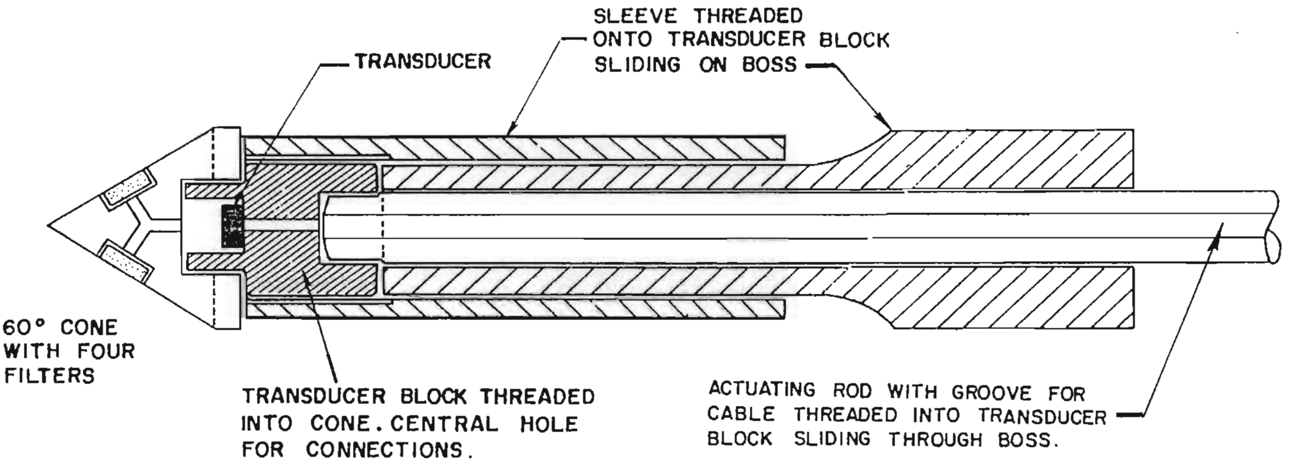
C5 LABORATORY PIEZOMETER CONE

The laboratory piezometer cone was designed by the author in early 1975 to incorporate a pore pressure measuring system into a standard mechanical cone which was being used for the laboratory consolidometer-cone testing. The previous consolidometer-cone used the standard friction sleeve Dutch mechanical cone modified only to the extent that the inner rod had been replaced with a longer rod to accommodate the weight platform. The standard cone, however, was extensively modified to fit in the piezometer and this is shown in Figure C5.1. Fortuitously the pressure transducers which were available were eminently suitable for the purpose and the same type continues to be used in piezometer cones fifteen years later. The particular transducer used, Kyowa PS - 1 KA had a pressure rating of 1 bar or 100 kPa. The transducer was a thin disc of 5 mm diameter and hence could very conveniently be fitted into the cone. The filter elements were set into the cone face as shown. The four elements consisted of ground to shape ceramic sections from conventional consolidometer drainage discs.

The pressure transducer sensitivity was 1 mV/V at full load (100 kPa) and the excitation voltage was 3 volt. The output was led directly to a chart recorder and resulted in the full chart width, 250 mm, representing a pressure change at 1 mV chart setting of 0,03 mV for 1 kPa or 1 mm representing 0,13 kPa.

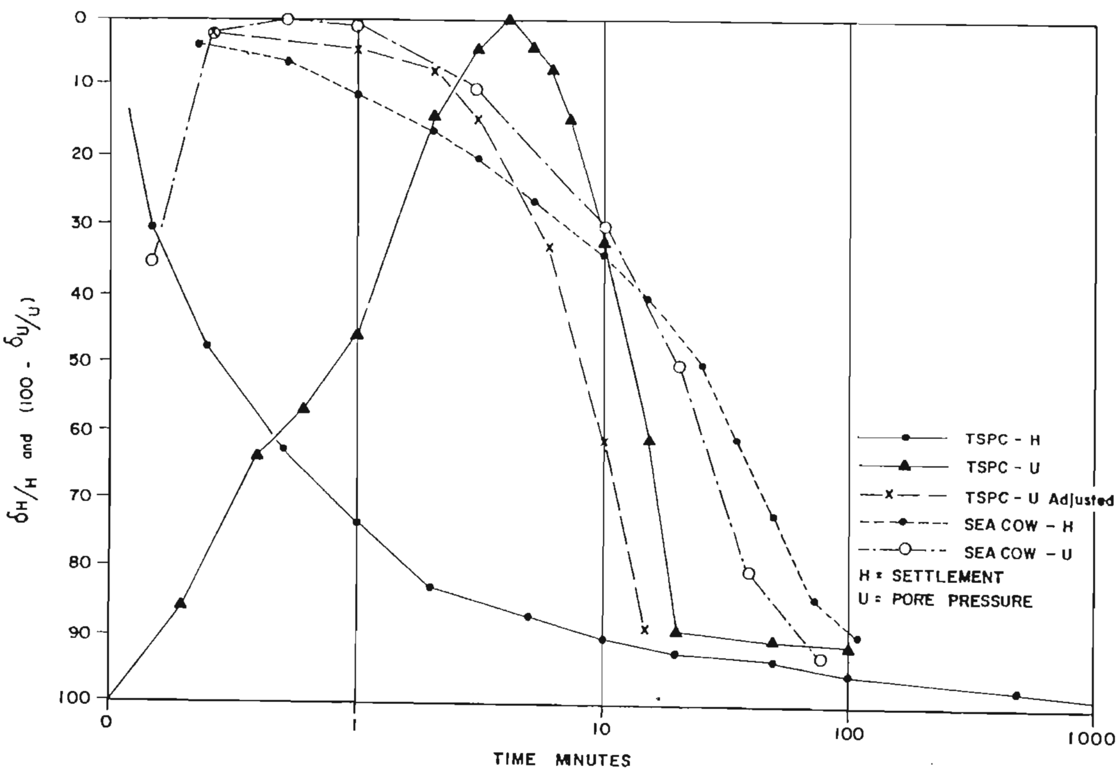
The results of the first test in a soil (TSPC) are shown in Figure C5.2. The shape of the cone settlement curve appeared to be a good example of the previous series of similar cone tests, but that of the pore pressure curve was puzzling. It showed an increase in pore pressure from the initial zero value for 4 minutes until a peak was reached at a pressure of 1,82 kPa. The cone pressure for this test was 30,8 kPa (7 lbs load) so the maximum recorded pore pressure was only about 5% of the cone pressure. The pore pressure subsequently dropped very rapidly, but then, even after a long time, did not essentially change and remained higher than the initial value.

Experience of operating the consolidometer-cone in sand had led to the procedure of adding the load in increments until sufficient was in place to promote a significant settlement. For this particular test the initial 4 lb load was supplemented by a further 2 lb then a further 1 lb within about one minute.



LABORATORY PIEZOMETER CONE

Fig. C·5·1



LABORATORY PIEZOMETER CONSOLIDOMETER-CONE
TIME- SETTLEMENT AND TIME- PORE PRESSURE
DISSIPATION FOR TSPC AND SEA COW LAKE.

Fig. C·5·2

Despite this delay in applying the full load there was still a significant lag before the peak pressure was registered. This was believed to be due to the measuring system itself being relatively soft, ie large volume, and lack of any de-airing procedures.

The peak pore pressure recorded was very much lower than the cone total pressure (viz 5%) and this low response was assumed to be because the peak pore pressure under the cone had already largely dissipated in the time taken for the instrument to respond.

The subsequent rate of dissipation was masked because of the lag in response; if the lag is assumed to be constant after the peak at 4 minutes, then correction should be possible by shifting the dissipation curve by 4 minutes to the adjusted position shown. Although the total time for 90% dissipation was about 20 minutes, the same as that for the cone to travel 90% of its total settlement, (Figure C5.2) the shape of the pore pressure dissipation curve was different from both the laboratory consolidometer-cone and consolidometer settlements Figures C4.15 and C4.16. This, however, was not unexpected because as already discussed in Section C4 it had been shown that in the silty sands the settlement measured and hence the pore pressure dissipation measured was not due to consolidation.

The pore pressure dissipation curve for sample TSPC in Figure C5.2 did not return to the original zero and this was because the depth of the cone had increased by, in this test, about 20 mm. Since the increase in pressure in the saturated sample was hydrostatic, it would have been expected to be 0,2 kPa which was very close to that measured.

It was concluded from this initial testing with the piezometer cone that the instrument itself required improvement, together with the de-airing procedure to ensure that the response time was minimized, but that the system was satisfactory.

Modifications were made to the cone to decrease the volume around the transducer to about one third of the original volume and shaped to minimize the possibility of trapping air bubbles. The holes from each of the filter element recesses were made finer to minimize the volume of the system. A method of de-airing and calibrating the piezometer cone was devised as shown in Figure C5.3. The perspex block was clamped over the cone and tightened in place with clamping screw; the water leads were connected to the laboratory triaxial equipment, which used de-aired water, and a pressure difference set up to cause flow through the system. After allowing an initial period to de-air, calibration of the system was accomplished by setting a series of pressures and measuring the corresponding chart displacements. It was observed that at changes of pressure the piezometer cone response was very rapid.

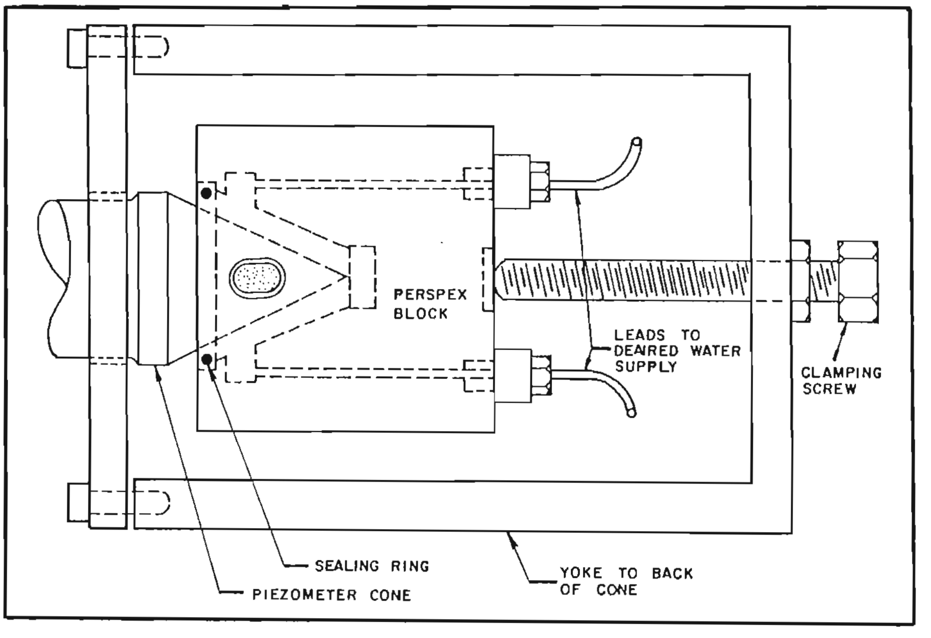


Figure C5.3 : De-airing and calibration adaptor

Few such tests were carried out and the records of these are only available in fragmentary form. A typical pore pressure record for the Sea Cow Lake clay is shown in Figure C5.2 together with the simultaneous cone settlement record.

In order to minimize initial loading increment problems the selected load was placed as one load. In some cases this induced failure and these results were discarded after cursory inspection. Where an apparent consolidation type settlement took place the pore pressure were generally as shown in Figure C5.2; some however showed erratic fluctuations of pore pressure.

Subsequent examination of the large block sample showed open fissures and small sandy inclusions and it was surmised that these had caused the pore pressure variations.

The typical result shown in Figure C5.2 for the Sea Cow Lake clay confirmed that the rate of settlement was closely matched by the rate of excess pore pressure dissipation, and therefore that the measured settlement was predominantly primary consolidation.

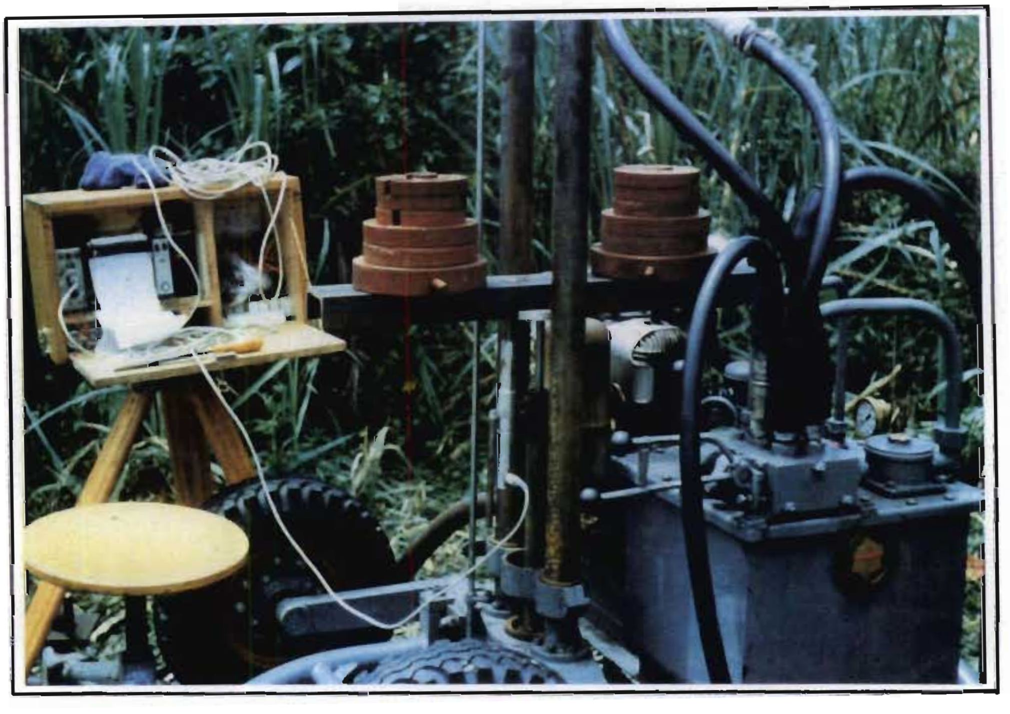
There was still some initial lag in the pore pressure response but subsequently dissipation stopped before completion of settlement. The maximum pore pressures were about 10% - 20% of the cone pressures and in some cases a residual pressure would continue which could not be explained by the change in depth since this was only about 3 mm. The pore pressures however, were very low, say up to 5 kPa, compared to full load of 100 kPa and the small residual pressure was assumed to be due to the system.

The overall conclusion was that in clays in the laboratory the consolidometer-cone was shown by the piezometer results to satisfactorily model consolidation. It was therefore necessary to test the consolidometer-cone (without piezometer) in the field.

During this period, May - June 1975, the opportunity arose to carry out cone penetration testing along a section of the proposed alignment for the National Road at Umkomaas, South Coast Natal. The National Institute for Road Research 100 kN Goudsche Machine Fabriek rig was used, with the modified strain gauge load measuring system described in C3.3. The standard mechanical friction sleeve penetrometer was used in the consolidometer-cone mode.

The approximate position of the proposed alignment then closely followed the coast and involved an embankment about 800 m long across a low lying swampy area. The mechanical cone penetration test results indicated extremely poor materials and it was obvious that embankment construction would have been difficult and the future performance would have been poor. As a result the proposed road was subsequently realigned to the present route, which is further inland.

During this investigation the field version of the consolidometer-cone was tried for the first time. The equipment is shown in Figure C5.4. It consisted of entirely standard cones and rods except for a longer than standard final inner rod to which the weight platform was fixed. The procedure was that a normal mechanical CPT was carried out to a suitable clay layer depth judged from the cone and friction sleeve measurements. The upper inner rod was removed and replaced with the special rod and load platform. A LVDT was positioned and connected to the chart recorder. Loads were added to the load platform until settlement was induced and then the time-settlement data recorded on the chart. The results were dissapointing; the dividing line between insufficient



FIELD CONSOLIDOMETER - CONE TESTING SHOWING LOAD PLATFORM, SETTLEMENT MEASURING LVDT TO CHART RECORDER. Fig. C·5·4

Fig C·5·4

weight to cause significant movement and sufficient to cause failure was so fine that very few consolidation curves were obtained. A large amount of weight was necessary (over 100 lb) even in a soft clay; this was expected, since cone pressures in this material were about 500 - 700 kPa - ie undrained shear strength of about 30 to 40 kPa - which is equivalent to a load of about 150 lbs and a significant proportion of this was expected to be necessary to cause settlements. The amount of weight was not in itself a major problem; the critical issue was that a relatively small increment could cause the cone to plunge down to the full extent of its travel. The loads and platform were removed, the cone advanced to a new position and a further test attempted. The success rate was low and the time taken for full settlement was long, being two or three hours and it became clear that the prognosis for the consolidometer-cone test as a routine field procedure was limited.

The operation, however, of the chart recorder and strain gauge load cell with the mechanical cone proved to be very successful in the field. The enhanced accuracy and convenience compared with the standard pressure gauges and manual recording of large numbers of readings was readily apparent. These experiences overcame the initial reluctance to operate this type of equipment in the field, bearing in mind both the rugged conditions and the relatively low level of sophistication of both operators and equipment then current in site investigation in South Africa.

Further important factors were that the success of the laboratory cone piezometer, the earlier experience of designing and building load cells to measure cone pressures, and the confidence of operating such equipment in the field had all led to the idea of building a field electric piezometer cone. The purpose of this was to measure in situ pore pressure dissipations and hence directly obtain the consolidation characteristics of the alluvial deposits, which was one of the ultimate objectives of the research.

The development of the field piezometer cone is described in Section C6.

In summary, at the end of the research on the use of the consolidometer-cone ie 1977, the overall standard of cone penetration testing in South Africa had been raised very greatly from that reported by Webb (1974) :

- the standard Dutch mechanical cone with friction sleeve was in common use,

- hydraulic load cells with high and low pressure gauges operating independently of the penetrometer rig hydraulic ram were mandatory,

- appreciation of the value of the results obtained from cone penetration testing had been significantly raised.

The author was instrumental in achieving these through advocating the technique and the improvement of the equipment during this period in consulting engineering, at the Roads Department of the Natal Provincial Administration and later at the CSIR. The latter not only allowed the research and general development of the equipment to be conducted, but through discussions, lectures and publications (Jones, 1974), and close ties with the road authorities, the awareness and knowledge of the technique was extended.

By 1974, it was clear that the standard cone load measuring equipment lacked the sensitivity for meaningful results to be obtained in the very soft alluvial materials, although the CPT did at least give a very definite indication of the presence of such materials. The author, nevertheless, was convinced that the inherent advantages of cone penetration testing, viz, rapid and relatively inexpensive testing of deep multilayered materials, should be exploited. A statement by Heijnen (1974) on penetration testing in the Netherlands was not considered by the author to be universally applicable, "Very seldom the cone resistance values are used for the prediction of settlements of compressible soil layers. As to be expected the employed relation is not very reliable".

The author's view was that not only was the cone penetration test suitable for the prediction of settlements, but that the research then being conducted would enable not only settlements to be adequately predicted on the softer materials, but would also allow satisfactory estimates of consolidation times to be made.

C6 DEVELOPMENT OF SOUTH AFRICAN FIELD PIEZOMETER CONE

Part C5 described the laboratory piezometer cone which had been developed by the author to determine whether the laboratory consolidometer-cone settlements conincided with pore pressure dissipation and hence measured consolidation settlement. It was concluded that in the clay tested the rate of dissipation of excess pore pressure was similar to the rate of settlement and therefore that the settlement was due to consolidation. The experience of using a piezometer in a cone had led to the belief that direct measuring of pore pressure dissipation was the preferred route for estimating consolidation characteristics.

The laboratory mechanical cone fitted with a piezometer had shown a number of aspects of the pore pressure response to loading in a clay; viz :

- the pore pressure increased immediately on loading
- at constant load (ie in the consolidometer-cone mode) the pore pressures gradually decreased
- the pore pressure increased immediately on loading but instantaneously decreased on unloading to some intermediate value
- if the cone was clamped in place before unloading the instantaneous pore pressure decrease was avoided and dissipation continued.

What was particularly interesting, was that the pore pressure dissipation did not require ongoing penetration. It was about this time, mid 1976, that the 1975 papers by Torstensson and Wissa became available. These addressed the rate of penetration around piezometers and together with the author's experience were sufficient to confirm that a cone for simultaneously measuring cone and pore pressures was a necessary development.

Initially the author considered measuring cone loads through the standard mechanical system of inner rods to the surface, and measuring pore pressures as in the laboratory piezometer cone. This was abandoned because of the problem of accommodating both inner rods and a cable within the conventional sounding casings which had an internal diameter of 16 mm, with the inner rods having a diameter of 15 mm, hence there was no space for a cable.

Some thought was given both to threading a continuous cable through the inner rods and to having a discontinuous cable through the inner rods with connectors at each end of the rods. Both of these would have been difficult; the former would have seriously weakened the inner rods - but was a possibility if the equipment was only to be used in softer materials - and the latter would have necessitated a large number of waterproof connectors which would have been a major source of problems.

If the inner rods were drilled to accommodate cables then in dense materials rod pressures of 200 MPa would result. This is not an excessive pressure for high quality steel rods for simple compression and the reduction in area would not be intolerable even at full load. In practice, however, the inner rods, unless the ends are very carefully machined and maintained in perfect alignment with one another, will be subject to high eccentric loads and buckling and overstress of the ends would inevitably result. Despite these potential problems the proposed system had some benefits, provided some restriction on the cone loads could be maintained. The obvious benefit would have been the use of the existing mechanical cone systems, together with the strain gauge load cells for measuring cone and friction sleeve loads.

The alternative was to dispense with the mechanical cone system altogether and change to cones incorporating load cells and hence eliminate the inner push rods.

Electrical cones had been in use elsewhere for some years, but not in South Africa, and de Ruiter's 1971 ASCE paper illustrated such strain gauge penetrometers - Figure B2.5.

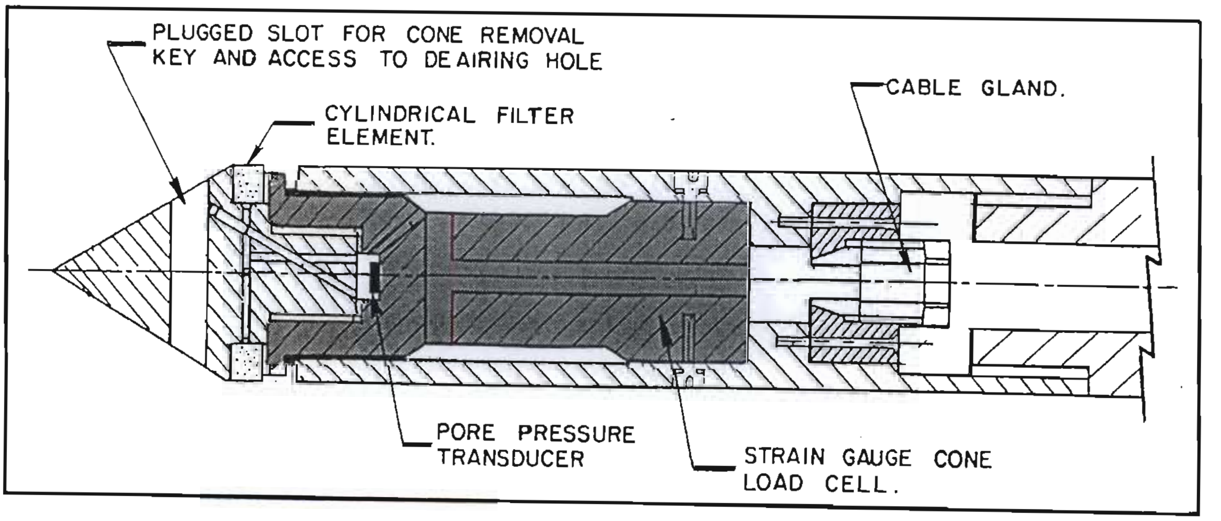


Figure C6.1 : Field piezometer cone

	LITHOLOGY	LITOLOGIE
Q	Q	Alluvium, sand, calcrete Alluvium, sand, kalkreet
Q	Qb	Aeolianite, sand, clay, limestone Eolianiet, sand, klei, kalksteen
T	T-Qa	Limestone, clay, conglomerate Kalksteen, klei, konglomeraat
T	T-Qb	Limestone, sandstone, conglomerate Kalksteen, sandsteen, konglomeraat
T	T-Qk	Sand, limestone Sand, kalksteen
T	T-Qn	Aeolianite, dune sand Eolianiet, duinsand
T	Tg	Silcrete Silkreet
T	Tu	Siltstone, limestone, calcarenite Sliksteen, kalksteen, kalkareniet
K	Ki	Limestone Kalksteen
K	Kmb	Conglomerate, sandstone Konglomeraat, sandsteen
K	Kmz	Limestone, clay Kalksteen, klei
K	Ksu	Breccia, tuff, trachyte, melilite basalt, carbonatite Breksie, tuf, tragiet, melilietbasalt, karbonatiet
K	Kma	Sandstone, conglomerate, marl Sandsteen, konglomeraat, merrel
K	Kmg	Conglomerate, sandstone Konglomeraat, sandsteen
K	Ks	Sandstone, mudstone, shale Sandsteen, moddersteen, skalie
K	Kz	Siltstone, sandstone, conglomerate Sliksteen, sandsteen, konglomeraat
J	J-K	Mudstone, sandstone, conglomerate Moddersteen, sandsteen, konglomeraat
J	Jb	Rhyolite, syenite, basalt, tuff, breccia, conglomerate, sandstone Rioliet, siëniet, basalt, tuf, breksie, konglomeraat, sandsteen
J	Jdr	Basalt Basalt
J	Jj	Rhyolite Rioliet
J	Jm	Basalt Basalt
J	Js	Basalt, tuff, breccia Basalt, tuf, breksie
J	Jd	Dolerite Doleriet
J	Je	Conglomerate, sandstone Konglomeraat, sandsteen
J	Jl	Basalt Basalt
J	Jp	Breccia, agglomerate, tuff Breksie, agglomeraat, tuf
J	Jt	Granophyre Granofier
Ꞧ	Ꞧb	Mudstone Moddersteen
Ꞧ	Ꞧm	Sandstone, mudstone, shale Sandsteen, moddersteen, skalie
Ꞧ	Ꞧny	Mudstone, sandstone Moddersteen, sandsteen
Ꞧ	Ꞧc	Sandstone, siltstone Sandsteen, sliksteen
Ꞧ	Ꞧmc	See Ꞧc, Ꞧe, Ꞧm Kyk Ꞧc, Ꞧe, Ꞧm
Ꞧ	Ꞧt	Mudstone, sandstone Moddersteen, sandsteen
Ꞧ	Ꞧe	Mudstone, sandstone Moddersteen, sandsteen
Ꞧ	Ꞧnt	Sandstone Sandsteen
P	P-Ꞧ	Shale, sandstone, mudstone, coal Skalie, sandsteen, moddersteen, steenkool
P	P-Ꞧi	Mudstone, sandstone Moddersteen, sandsteen
P	P-Ꞧsk	Shale, mudstone, sandstone Skalie, moddersteen, sandsteen
P	Pa	Mudstone, sandstone Moddersteen, sandsteen
P	Pf	Shale Skalie
P	Pp	Shale Skalie
P	Ps	Shale, sandstone Skalie, sandsteen
P	Pw	Carbonaceous shale Koolstofhoudende skalie
P	Pc	Sandstone, shale Sandsteen, skalie
P	Pk	Shale Skalie
P	Ppr	Shale Skalie
P	Pt	Shale Skalie
P	Pwa	Sandstone, shale Sandsteen, skalie
P	Pe	Shale Skalie
P	Pko	Sandstone, shale Sandsteen, skalie
P	Ppw	See Ppr, Pw Kyk Ppr, Pw
P	Pv	Sandstone, shale, coal Sandsteen, skalie, steenkool
P	Pem	Mudstone, shale, sandstone Moddersteen, skalie, sandsteen
P	Pm	Sandstone, shale, coal Sandsteen, skalie, steenkool
P	Pr	Sandstone, shale Sandsteen, skalie
P	Pvo	Shale Skalie
C	C-Pd	Tillite, sandstone, mudstone, shale Tilliet, sandsteen, moddersteen, skalie
D	Db	Shale Skalie
D	Dc	Shale, sandstone Skalie, sandsteen
D	Dt	Shale, siltstone, sandstone Skalie, sliksteen, sandsteen
D	Dbi	Shale, siltstone, sandstone Skalie, sliksteen, sandsteen
D	Dl	Shale, sandstone, diamictite Skalie, sandsteen, diamiktiet
D	Dw	Quartzitic sandstone, shale Kwartsitiese sandsteen, skalie
S	Sn	Quartzitic sandstone, shale, tillite Kwartsitiese sandsteen, skalie, tilliet
O	O-S	Quartzitic sandstone, arkose, shale Kwartsitiese sandsteen, arkose, skalie
O	Op	Quartzitic sandstone, shale Kwartsitiese sandsteen, skalie
O	Ope	Quartzitic sandstone Kwartsitiese sandsteen
€	€k	Sandstone, conglomerate, shale Sandsteen, konglomeraat, skalie
€	€t	Granite, syenite, foyaite Graniet, siëniet, foyaïet
N	N-€	Biotite granite Biotietgraniet
N	N-€k	Biotite granite Biotietgraniet
N	Nnt	Conglomerate, mudstone, limestone, schist Konglomeraat, moddersteen, kalksteen, skis
N	Ntu	Amphibolite, gneiss, schist Amfiboliet, gneis, skis
N	Nmp	Gneiss, granulite Gneis, granuliet

LITHOLOGY

SCALE : 1:1000000

FIGURE A.3.1c

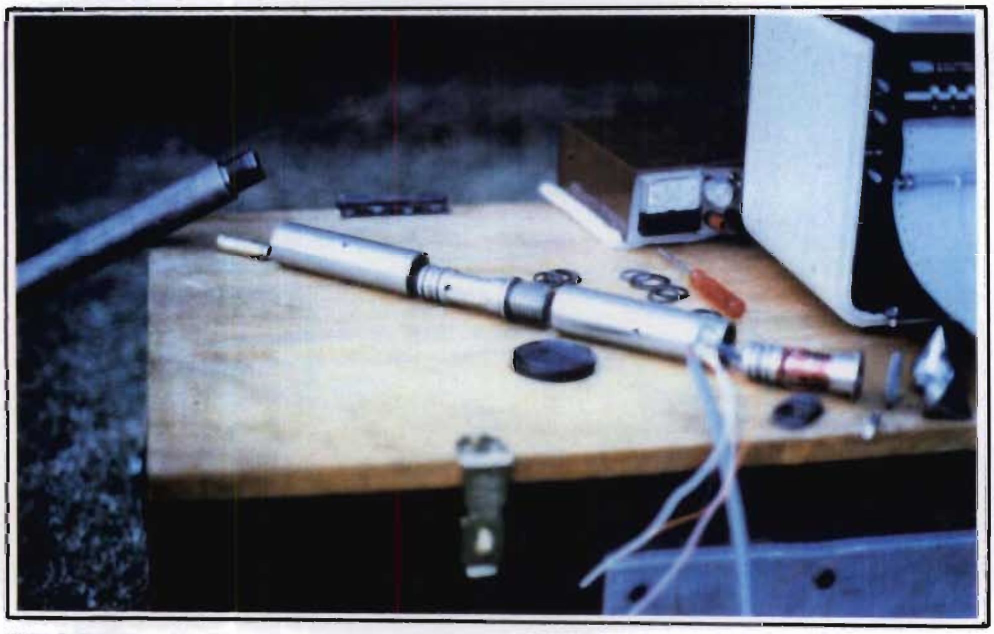
The first prototype electrical piezometer cone was designed by the author and built in 1976; design drawing of the piezometer cone is reproduced in Figure C6.1. The technology was developed from experience with the laboratory piezometer cone and from the earlier strain gauge load cells systems which had been used both for vane shear testing and for the enhanced cone load measuring systems for the mechanical cone testing. The piezometer cone was calibrated in a modified triaxial cell - with the top plate of the cell having a 36 mm diameter hole and sealing rings - and tested in the cone penetration loading frame previously described - Figure C4.8.

The combined cone load and pore water measuring penetrometer, called the piezometer cone or piezocone, performed extremely well in the laboratory trials and both cone loads and pore pressures could be accurately calibrated through the triaxial test equipment.

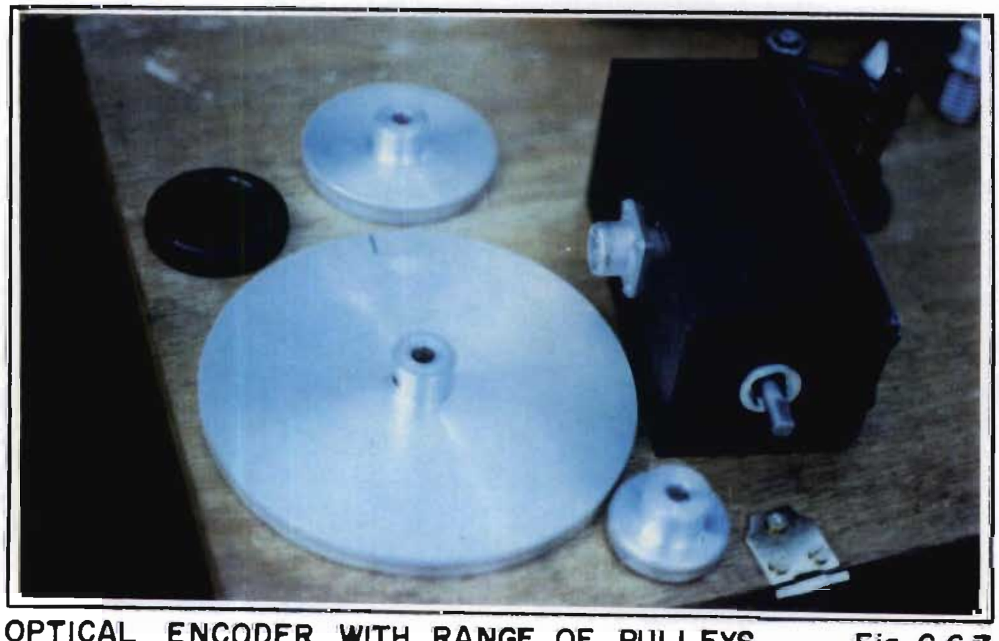
The pore pressure system worked almost perfectly. The resolution at the full range of pressures tested was more than adequate. The pressure transducer was upgraded to the Kyowa PS - 2 KA (200 kPa), instead of the 100 kPa transducer used in the initial laboratory cone, despite the fact that even the upgraded transducer would limit the depth capability in soils giving a high pore pressure response. The response time of the pore pressure system was again noted to be variable and dependent on the efficiency of the initial de-airing and subsequent maintenance of saturation. These were not difficult to achieve and gave no cause for concern that there would be practical field problems. The four flush mounted face filter elements appeared to perform satisfactorily, but were difficult to manufacture. The purpose of face mounted filters was that it was assumed that the maximum shear and compressive stresses, and hence pore pressure development, would be at the face of the cone, thus filters half way up the cone would represent an average position. However, from experience with various piezometers, the idea was conceived of having a cylindrical filter element at the base of the cone where it would be less subject to damage during penetration than face or tip mounted filters. It was appreciated that a different pore pressure regime would be generated in this zone compared to the cone face, but since the primary purpose was to generate an excess pressure in order to measure its rate of dissipation, the magnitude of excess pore pressure was largely irrelevant, provided it was sufficient to sensibly measure. The practical simplicity of the cone base position made it the preferred situation. A cone was therefore made in this configuration. The filter element, as in the laboratory piezometer consolidometer-cone, - Figure C5.1 - comprised ceramic filters used in conventional consolidation tests. Figure C6.2 shows the two cone types.

FACE (Left) AND BASE (Right) FILTER CONES (LATTER WITH HOLE AND PLUG FOR UNSCREWING BAR. Fig. C·6·2(a)

FRICTION SLEEVE (Left) AND CONE (Right) LOAD CELLS. Fig. C·6·2(b)



FRICTION SLEEVE PIEZOMETER CONE. Fig. C·6·3



OPTICAL ENCODER WITH RANGE OF PULLEYS. Fig. C·6·7

The load cell system was designed by the author on the basis of the knowledge gained from previous load cells made for measuring cone loads through the mechanical cone system inner rods. It was found that there was more than adequate space within the penetrometer body to provide sufficient cross sectional area for the load cell, so the design decision was to select the desired load range and provide a suitable cross section area. A solid cylindrical section was chosen since this was relatively easy to manufacture; flats were machined on the cylindrical section to ease the problem of cementing the strain gauges - Figure C6.3.

The diameter of the central measuring part of the load cell was 24 mm, which after machining the four flats gave a cross sectional area of 425 mm^2. This allowed a cone load of 85 kN at a steel stress of 200 MPa; with a modulus of 200 GPa the strain at full load would therefore be about 100 micro strain. The electrical strain gauge load cell design allowed for drilling out a central hole to decrease the cross sectional area and hence produce a range of load cells for designed pressure ranges with the same external sizes and fitting.

The original strain gauge system consisted of a simple uncompensated bridge which was later (1977) improved to comprise four 90 degree, 120 ohm Kyowa KFC-2-D15-11 strain gauge rosettes in one fully compensating bridge to measure axial loads only and compensating for eccentric loads and temperature changes.

With this bridge, and using a 5 volt excitation voltage, the maximum output from the load cell was 10 mV. In practice the cone pressures in different materials have a very wide range, varying from about 30 kN for dense sands to about 0,2 kN for soft clays; in addition it was possible by hitting rock or boulders, to develop instantaneous cone loads of about 60 kN before penetration was stopped. The first load cell was therefore a deliberate compromise. The soft clay load of 0,2 kN (equivalent to undrained shear strength of 15 kPa) produced an electrical output of only 0,02 mV which on a 100 division chart at 1 mV full scale gives only 2 divisions. The resolution in terms of undrained shear strength for soft clays was little better than say ± 3 kPa and if the system operated only moderately well this accuracy could be expected to be lower and would be inadequate for the measurement of undrained shear strength of soft clays. The system was, however, experimental and the primary purpose was to check the pore pressure measuring system in the field; the cone load system was deliberately detuned to avoid overloading damage. Nevertheless, even at this detuned level, the sensitivity of the cone load measuring system was considerably superior to the then current standard of mechanical cones.

The laboratory tests carried out in a triaxial test frame revealed a number of mechanical and electrical problems. Water leaked under pressure, despite the cone sealing ring, into the load cell space and caused electrical problems : under repeated loading some shift in the zero load readings was observed, and at the amplifications necessary to give reasonable chart output a considerable extraneous interference or noise was apparent.

The pore pressure measuring system operated without problem.

At this stage, the beginning of 1977, the author left the NITRR and joined the consulting engineering practice of van Niekerk, Kleyn and Edwards (VKE), who, at the time were designing a major national freeway project along the Natal South Coast which involved a number of crossings of rivers and flood plains. The investigations for these and subsequent monitoring is described in Part D. Fortunately, because of the cooperation between the Department of Transport, NITRR and VKE, it was arranged that the research equipment be transferred to the VKE soils laboratory so that the development could continue.

Despite the pressures of the new post, some progress was made with the equipment during 1977. The strain gauge bridge was improved to include eccentric load and temperature compensation; techniques for cementing and waterproofing the strain gauges were developed and the sealing ring and cable gland were improved. The consolidometer-cone frame was utilized for testing prepared samples. These were clayey layers of different thicknesses in silty sands. The tests showed that thin layers, ± 10 mm, could readily be detected by the very marked changes in pore pressure response. Significant pore pressures were recorded in the clay layers and dissipations of these with time were observed. Field tests were conducted in the Pretoria area where softer materials could be found. The systems worked as had been shown by the laboratory tests and the field work was primarily to develop the operating techniques needed to deal with cables through the pushing devices and through the CPT outer rods. The instrumented cone had a short length of 8 core cable emerging from the penetrometer which was connected to the main cable by a threaded plug and socket having eight connectors. This connection gave incessant problems and it proved impossible locally to find a suitable connector that was watertight and could cope with both the tension and bending at the plug and also fit inside the 16 mm internal diameter casings. The connector system was somewhat reluctantly abandoned in favour of having the penetrometer semi permanently attached to a 30 m length of cable. The reluctance was because damage to the cable would entail opening the cone in order to remove the cable. In fact it transpired that opening the cone was more frequently required to repair the cone rather than the cable and until recently subsequent models continued with the integral cable method.

In mid 1978 the first commercial site investigation project using the piezometer cone was undertaken at Bafokeng Impala Platinum mine some 100 km west of Pretoria. The purpose was to establish the hydrostatic conditions within an existing tailings dam, which was then about 25 m high, so that decisions could be made regarding the stability and hence the potential safety for raising the dam. The piezocone was intended to perform as a piezometer by penetrating to selected depths and measuring the ambient pore pressures, but, in addition, it had the advantage of measuring cone resistances.

The expectations were over-modest : the system gave results that more than confirmed all the hopes for it.

At that stage of development the two data sets of cone and pore pressure were fed directly to a two pen TOA chart recorder without amplification. The only other apparatus were regulated voltage supplies working off a 12 volt car battery and a generator to supply power for the chart recorders. Figure C6.4 shows the machine in operation at Bafokeng. Figure C6.5 shows a section of a typical field chart result and Figure C6.6 shows this in the form of cone pressures against depth and also pore pressure against depth.

The implications of the results justify detailed discussion of the chart output. The two pens of the chart recorder are offset so that they may pass one another, which means that the cone and piezometer traces are offset. After each metre of penetration a further rod has to be added and this takes about 45 - 60 seconds before penetration can restart. The chart was normally switched off for this period and restarted simultaneously with restarting penetration. The cone and pore pressure readings, and changes to them, continued in the rod change period, and it was observed that neither necessarily returned to zero.

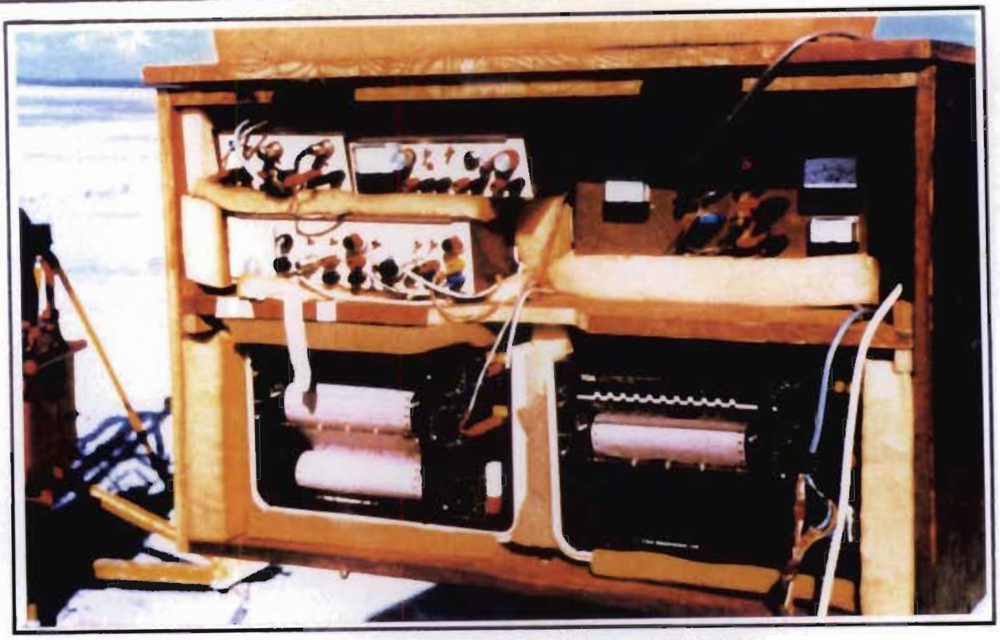


CHART RECORDERS, AMPLIFIERS AND CONTROL BOX.

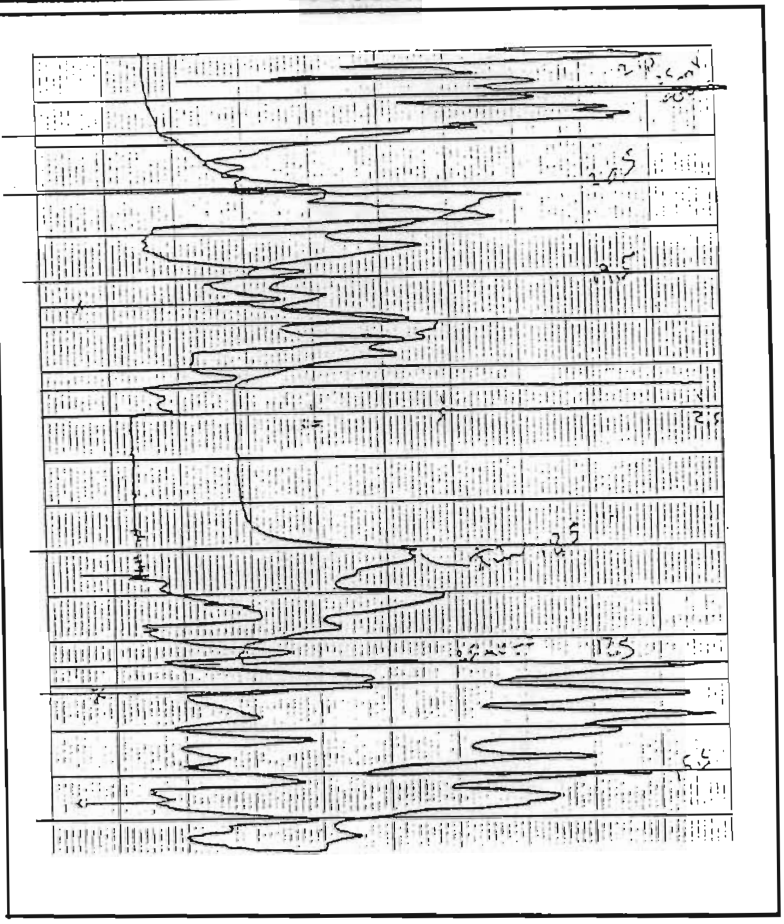
Fig. C·6·4(a)

CPT RIG AT GYPSUM DAM – SHOWING OPTICAL ENCODER ON TRIPOD.

Fig. C·6·4(b)

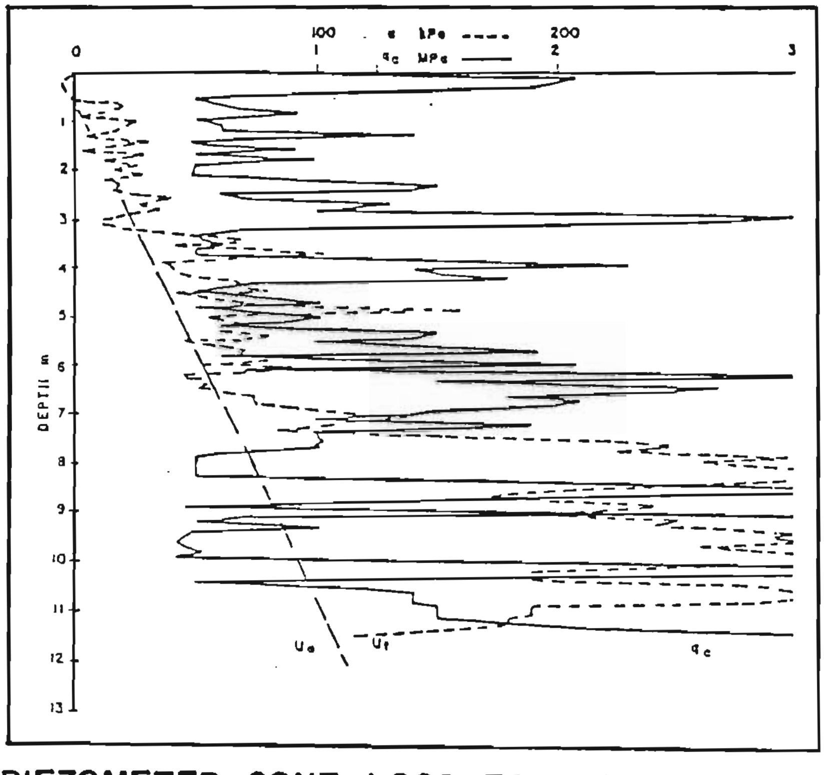
CPT RIG AT PLATINUM TAILINGS DAM – BAFOKENG.

Fig. C·6·4(c)



TEST RESULTS AT BAFOKENG

Fig. C·6·5



PIEZOMETER CONE LOGS FOR BAFOKENG

Fig C·6·6

The following observations were made from the chart readings during penetration :

- Both cone and pore pressure readings fluctuated very markedly presumably indicative of a multi layered tailings of different densities and permeabilities.
- The peaks and troughs of the cone and pore pressure readings were precisely in phase and of opposite sense (noting the offset pen distance)
- The magnitudes of the corresponding cone and pore pressures were inversely proportional to one another viz a small cone reading corresponded to a large pore pressure reading.
- The chart distance for 1 m penetration was about 22 mm and since the penetration rate was intended to be 20 mm/sec ie 50 sec for 1 m, then either the chart speed was inaccurate or the actual penetration rate was slower than it should have been. The latter proved to be the case and was about 15 mm/sec, ie at the limit of the recommended standard rate of 20 ± 5 mm/sec.
- The chart needed to be annotated at rod changes since it was possible to lose track of the depth, particularly if stopping and starting of the recorder was not well synchronised with the penetration.
- It was possible to select high pore pressure layers, stop penetration and allow the chart to continue to record pore pressure dissipations. The cone readings returned to practically zero in these extended periods. Distinct differences in dissipation rates at different depths could readily be seen.
- The pore pressure system with a 200 kPa working range was inadequate even at modest depths and the peak pressures often exceeded 200 kPa.
- The pore pressure readings appeared to be considerably more sensitive than the cone readings in that fluctuations in readings were more rapid.

Not all the observations were positive and the following aspects were considered to be sufficiently problematic to require improvement before the equipment could be used for routine investigations :

- Both cone and pore pressure signals, particularly the former, required amplification if the equipment was to be sufficiently sensitive to measure small changes.

- The cone load cell required a smaller cross sectional area to operate at a lower load range and the fairly marked problem of a shift in the unloaded zero reading had to be eliminated.
- The cable was vulnerable to damage and to electrical interference.
- The filter elements cracked after loading
- The time base for the chart led to difficulties in processing the results, ie the chart movement should be controlled by depth not time.

On balance, however, the results were considered to be overwhelmingly successful; the information available on the layering and pore pressure conditions within the tailings dam was far superior to that obtainable by any other investigation method and, subject to the difficulties described, the operation was straightforward. There was no doubt that it was necessary to improve the mechanical and electrical aspects to a user friendly state. Processing the results was extremely laborious and it took ten times longer to produce a finished CPT log than to carry out the test in the field - no immediate solution was seen for this except, as a starting point, having the chart transport automatically controlled by the penetration depth.

At this stage Eben Rust, a newly qualified civil engineer joined the author as part of the team involved with cone penetration testing. Over the years he became involved, under the author's direction, with the interpretation as well as the equipment development and field work.

The piezometer cone equipment developments in the early years (1977 - 1982) are listed below and are also described in the author's papers given in Appendix I.

- Mechanical components, particularly cone seals and cable gland improved to prevent leaks.

- Load cell range reduced by having central hole; material changed from stainless steel to tempered high tensile steel with higher allowable strain which gave a greatly improved load cell.

- Piezometer transducer changed from Kyowa model PS-2KA to PS-10KA, giving a pressure range up to 1000 kPa.

- Amplifiers separate from the chart recorders were built, together with superior voltage regulators for the cone load cell and pore pressure transducers.

- The chart transport control was changed from time based to depth based. The recorder being used was designed to accept the chart transport rate either from its internal timer or from an external source. Numerous devices were tried such as a long brass rod mounted on the CPT rig with a spring loaded connector to the moving cross head of the rig measuring the changing voltage along the rod, and a similar 1 m long threaded brass rod with a moving on-off switch counting the threads. The final system utilized a commercially available linear optical encoder : literally, a black box which, through an internal rotating shutter system driven by an external pulley, produced 5 volt square wave pulses which were fed to the chart recorder - Figure C6.7. The external pulley was driven by a belt to a pulley fixed to the cross head of the rig and the black box was mounted on a survey tripod - Figure C6.4. The encoder pulley size could be changed so that the ratio of penetration depth to chart advance was controlled and was usually set at 10 mm/metre. This depth linked chart drive system has worked virtually perfectly since originally fitted.

- The filter element was changed from a ceramic to a plastic material used in large diameter consolidometers (150 mm dia Rowe Cells). Both face and base filter position cones were manufactured.

- The piezocone cable was made less vulnerable to damage and interference by threading it through a transparent plastic tube.

- Adaptors for pushing and withdrawing the rods were made which improved the ease of cable handling at the rig.

The equipment was used in practice to the extent that during 1980 a number of site investigations were undertaken not on an experimental basis but for routine investigations and some of these are are discussed in Part D.

However, some problems still existed with the equipment, the primary one being that data reproduction was very tedious. Some zero drift also continued to occur on the load cell and although this could be compensated in the data processing it was not entirely satisfactory.

At this stage (1981) the author moved to the specialist geotechnical consultants Steffen, Robertson and Kirsten for whom many of the site investigations using the piezocone had been conducted. From then improvements were primarily to the electronics of the system which were made under the guidance of the author and designed and built by Rust.

A digital, as opposed to the previous analog, data logger was built which included new amplifiers and a digital tape recorder with a transfer port to connect to an office based computer. An analog chart recorder was used in parallel to the data logger so that a visual output of the cone and pore pressure readings was available during the penetration testing.

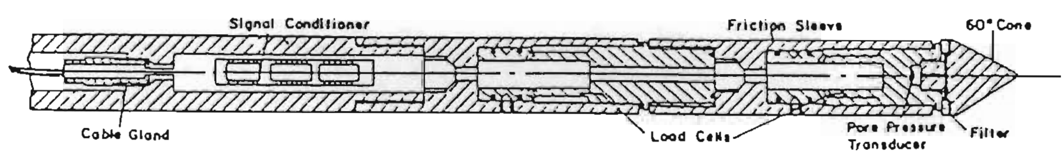


Figure C6.8 : Friction sleeve piezometer cone

A friction sleeve was included in the penetrometer and with the sleeve load cell of the same pattern as that of the cone - Figure C6.8. Mini computers were becoming much more common and with some manipulation were suitable for data logging and processing and a Sharp MZ 80B PC was modified to accept an analog to digital board. An amplifier was made which fitted inside an extension rod above the cone and sleeve. The system worked well, the only significant problems being that the data storage was limited to 15 m of cone penetration for each data file tape cassette and that the floppy drive unit was not suitable for field use because of dust problems.

The down the hole amplifier performed well but because of the ever present danger of losing or damaging a penetrometer, the amplifier was later transferred to be part of the surface equipment where it worked just as well.

Further development progressed as mini computers developed and the Sharp was replaced with an Olivetti M24 in conjunction with the original purpose made data logger. The chart recorder system continued to be used in parallel with the sophisticated data logging and processing, firstly because it provided a useful check and back up for the data logger and secondly because with experience it was possible to interpret the chart during operation and decide whether and where to carry out dissipation tests.

The 1991 piezometer cone equipment included a laptop computer and more sophisticated data processing programs. From these not only can the conventional cone and pore pressure against depth logs be produced instantaneously in the field, but the data can be manipulated to produce pore pressure ratios or other combinations of the measured parameters for subsequent interpretation.

During the development of the piezometer cone equipment, methods of interpretation of the data obtained were also developed. These have been described in Part B in which reference was made to the author's method of determining coefficients of consolidation from pore pressure dissipation tests and soils identification from pore pressure to cone pressure ratios.

The application of piezometer cone penetration at numerous sites is described in Part D together with two research projects for the validation of the parameters used to estimate total settlement and consolidation times from piezometer cone testing.

D SOUTH AFRICAN APPLICATION OF PIEZOMETER CONE PENETRATION TESTING

D1 INTRODUCTION

In the 1975 - 1985 decade the piezometer cone was developed practically to its current level of sophistication and the use of CPT was considerably expanded throughout the world. Despite the fact that no all embracing theoretical analysis is available the value of the CPT, and other insitu testing, is well appreciated particularly for soft alluvial or estuarine deposits. A variety of methods now exist for determining compressibility, consolidation and shear strength parameters for soils from CPT and CPTU data and sets of correlations are available in the international literature; much of this data for South African soils has been obtained by the author and colleagues.

Experience in South Africa, however, indicated that although the methods appeared fairly reliable for the prediction of compressibility of predominantly sandy materials, clayey sands and sandy clays, discrepancies have occurred in predictions for some embankments on soft clays. The author was directly involved in many of the investigations using cone penetration testing, some of which are described in the papers in Appendix I and are summarized briefly in section D2. Further sites have either been described by others in papers or in geotechnical investigation reports and relevant information from these is summarized in section D3. Because of the discrepancies in some of the predictions on soft clays, two research projects were conducted by the author, 1989 - 1990 and 1991 - 1992, to calibrate the methods of settlement prediction for South African soils using cone penetration testing. These two projects are described in Sections D4 and D5.

D2 SITES DESCRIBED IN AUTHOR'S PAPERS (APPENDIX I)

D2.1 Umhlangane - Sea Cow Lake - (Jones, le Voy and McQueen, 1975)

Approximately 6 km of embankments over soft clays were constructed for the national road immediately north of Durban. The section with lowest shear strengths was selected for detailed investigation using mechanical cone penetration testing with the standard Dutch cone with friction sleeve. This section was at Umhlangane, also called Sea Cow Lake, where the proposed embankment about 7 m high traversed a valley having up to 20 m depth of soft clays of undrained shear strengths of about 15 kPa. From the laboratory and field testing, which included the construction of a trial embankment predictions were made of the settlement of the main embankment. These were that settlements of about 2 m were to be expected.

History has shown that settlements close to 4 m have occurred and that the consolidation time has been about twice as long as anticipated. This embankment had an additional distinction. It failed during construction a few weeks before the 6th African Regional Conference on Soil Mechanics and Foundation Engineering, which was held in Durban and thus provided the opportunity for a most valuable site visit. Appendix I also contains the discussion from the 1975 conference on the causes of the failure.

This embankment formed part of a subsequent research project and is discussed in more detail in section D4.

D2.2 Mtwalumi (Jones, 1975)

This 16 m high embankment for the national road south of Durban was constructed over about 25 m of silty sand. Mechanical cone penetration tests were carried out and settlement predictions based on these gave settlements of 0,7 ± 0,2 m which compared well with the measured settlement of 0,8 m. The predictions are discussed in detail in the CSIR, NITRR publication RS/6/74 and mentioned by Jones (1975) together with the settlements at Uvusi.

D2.3 Uvusi (Jones, 1975)

This 5 m high embankment was for an access road to the national road close to the previous, Mtwalumi embankment. The subsoil was similar but shallower. The predicted settlements were 0,2 ± 0,05 m and the measured settlement was 0,18 m.

D2.4 Umgababa (Jones and Rust, 1981)

This embankment later formed part of the research project conducted in 1989 - 1990 and is discussed in the summary of the project given in Section D4. The 400 m long, 5 m high national road embankment was constructed over 23 m of alluvial deposits. These consisted of 15 m of very soft dark grey clay (undrained shear strength about

20 kPa) sandwiched between two silty sand layers. Detailed investigations were undertaken including inter alia cone penetration testing with the standard mechanical cone and friction sleeve using the author's improved strain gauge load measuring system with chart recording of the results. Based on the CPT results and laboratory consolidometer testing, including large diameter (150 mm) Rowe cells, the predicted embankment settlement was 0,9 m to 1,2 m. The actual embankment settlement more than ten years later was about 2,8 m and is expected to increase to about 3,8 m over the next 10 years since significant excess pore pressures are still present in the clay. The discrepancy between the settlement prediction and the measured settlement was unsatisfactory and this was one of the main reasons for the two subsequent research projects.

D2.5 Umzimbazi (Jones et al, 1980)

This embankment also forms part of the research project referenced in Section D2.4. It is about 4 km north on the same section of road as the Umgababa embankment. The embankment height is also about 5 m and the depth of soft subsoil is about 13 m. It comprises about 7 m of soft clay (undrained shear strength of 10 - 15 kPa) sandwiched between two 3 m layers of sand. Settlement predictions based on the CPT results and laboratory consolidometer tests gave estimated values of about 0,7 m. The measured settlement some 10 years later is 1,7 m and it appears that this has now stopped.

The original estimate of the settlement led to the engineering decision at the time to proceed without any subsoil improvement technique such as a vertical band or sand drain system. The result has been, as at the Umgababa embankment, that adjacent to the river bridge the ongoing embankment settlements have given rise to a poor riding quality and considerable maintenance has been necessary. With hindsight it is likely that a better settlement prediction would have resulted in the application of subsoil drainage and hence reduced maintenance problems.

D2.6 Sabi River, Zimbabwe (Rea and Jones, 1984)

The paper describes part of a multistage investigation of the site for a dam. The requirement was to define the extent and nature of the transported deposits in the river channel both for the design of a coffer dam and for the main dam. Very detailed geological sections were produced using the Jones and Rust Soils Identification Chart (1982) and this correlated well with a very detailed seismic survey.

D2.7 Waste Ash Dam - Kilbarchan (Jones and Rust, 1982)

The essential results of this investigation were two fold. Firstly, that across the waste ash dam between the hydraulic discharge point and the walls a distinct change of material type could be deduced from the CPTU soils identification which showed that the material was coarsest at the discharge point and became finer towards the wall as the effluent velocity decreased. Secondly, that by allowing time for dissipation at each rod change (1 m depth intervals) the ambient pore pressures were measured and these indicated very clearly that the piezometric conditions were far from hydrostatic and that a significant flow was taking place downwards through the base of the dam, which had implications for the pollution control of the system.

D2.8 Tailings Dam - Bafokeng (Jones and Rust, 1982)

In the same paper as for the previous Section D2.7 an investigation at a tailings dam is described. An unusual feature of this was that it was possible to carry out CPTU's in the clay underlying the dam at a six month interval. The results showed that drainage was taking place in the clay, both upwards and downwards, and that a significant decrease in pore pressure had occurred in the period despite only a small drop in water level. The coefficient of consolidation could be estimated from this, hence the allowable rate of rise of the dam calculated so that the excess pore pressures generated in the clay subsoil would not be sufficient to cause stability problems.

D3 DIVERSE APPLICATIONS OF CPTU

At sites on alluvial deposits the potential geotechnical problems are generally related to shear strength and consolidation settlement both of which may be assessed by piezometer cone testing. Additionally the CPTU, because of the high resolution of different soil layers and because of the ability to measure ambient pore water pressures may be used in many different situations. Although these are not the primary subject of this research they serve to illustrate the usefulness of the technique. The following examples are taken from work carried out at various sites which have been reported only in project reports and not in publications.

D3.1 Tailings Dams

The dominant use of the CPTU in South Africa other than at embankments on alluvial deposits, has been at mine tailings dams and investigations have been carried out at the operating sites at Bafokeng, Westonaria, ERPM, Phalaborwa, and at many operating and disused dams in the Johannesburg area.

At these sites the purpose of the investigations has been the determination of existing phreatic surfaces so that assessments of safety could be made in feasibility studies for the raising of the dams or for closure and rehabilitation. The CPTU is ideally suited for this purpose since not only are the piezometric conditions determined, but at the same time the layering system and a measure of shear strength are also determined.

In the past five years a number of investigations have taken place during the planning and design for major roads in the Johannesburg area where either cutting through old tailings dams or filling over them has been considered. The CPTU has become almost standard practice in such instances.

A case history of particular interest is described in the paper by Rust, van Zyl and Follin (1984). The author's CPTU equipment was taken to USA in 1983 to demonstrate the usefulness of the CPTU for the investigation of tailings dams to the US Bureau of Mines. It was used to determine the conditions within an uranium tailings dam which was due to be closed and hence subjected to extensive testing to ensure conformation with strict environmental controls. The consolidation characteristics of the materials were assessed by CPTU to predict the settlement of a proposed earth blanket over the dam. It is understood that piezometer cone penetration testing has subsequently become accepted practice for USA tailings dams.

D3.2 Gypsum Waste Dam

The possibility of reclaiming an extensive gypsum waste dam at Richards Bay led to an intensive investigation in 1980 of the deposited gypsum tailings by Wrench - Figure C6.4 (b). This inter alia led to the successful presentation of a doctoral thesis by Wrench (1987) in which he described the use of the CPTU for the investigation of this unusual material using the author's equipment. Both the Dutch mechanical cone with friction sleeve and the piezocone were used. Wrench devoted some thirty pages of his thesis to the cone testing and concluded that the CPTU was very useful for the testing and was far superior to the mechanical cone system. He noted large variations in pore pressure response over very short depths and concluded that this was largely due to the great variation in the material brought about by the erratic addition of lime in the process. Nevertheless using the author's soil identification chart Wrench determined that the tailings were equivalent to a very loose to loose sand and silty sand. He considered this to be an appropriate classification because when used in various semi empirical relationships to obtain soil parameters such as friction angle and moduli, the values of these parameters compared favourably with values obtained from other forms of testing viz both laboratory and pressuremeter testing.

D3.3 Scour Depth - Umfolozi

During the floods in Natal resulting from the Demoina Hurricane (1984) many bridges were washed away or damaged and river channels altered. Two of the consequent investigation projects involved piezocone testing to define scour depth. The first of these was at a bridge site where the depth for piling was required, together with the depth to which the channel had been scoured and refilled during the flood. The latter was readily detected by the change in material type and cone resistance at the scour level. A similar exercise was undertaken at a number of positions across the flood plain as part of an investigation by the Department of Water Affairs aimed at more fully understanding the morphology of the river system under extreme flood conditions. Again the depth of the very recent deposits could be readily determined.

D3.4 Natural Ground Level Identification - Richards Bay

A site was hydraulically filled to a depth of about 5 m. Some contractual queries arose regarding the original ground level and hence quantity of fill which had been imported. Although the fill and the in situ material were very similar it was straightforward to carry out CPTU's from the surface of the fill and detect the original ground level thus providing the required information. The original ground level showed as a small change in material type over a thin layer, there being a slightly more silty zone which was clearly and consistently detectable across the site.

D3.5 Irrigation Scheme Feasibility - Makatini Flats

A study of the potential for rice growing was conducted over a very large area and one of the important factors was the permeability of the subsoils, since this strongly influenced the irrigation requirements. The CPTU was used to extrapolate information from a small number of boreholes with sampling, laboratory testing and in situ permeability testing, to the large area. Both the soil identification and dissipation aspects of the CPTU were utilized.

D4 CPTU RESEARCH PROJECT, 1989 - 1990

D4.1 Introduction

This research project was funded by the Research and Development Advisory Committee of the South African Roads Board following a proposal by the author for support to continue the work on cone penetration testing for the assessment of the consolidation characteristics of alluvial deposits, (Rust and Jones, 1990).

The three embankments involved in this project viz Umgababa, Umzimbazi and Umhlangane (Sea Cow Lake) have been the subject of papers by the author and colleagues not necessarily dealing only with cone penetration testing but also with embankment design and performance. Descriptions of the sites are given in these papers, copies of which are in Appendix I. The locations of the first two sites are shown on Figures D4.1 and the Sea Cow Lake site on Geology of Durban, Figure A3.2.

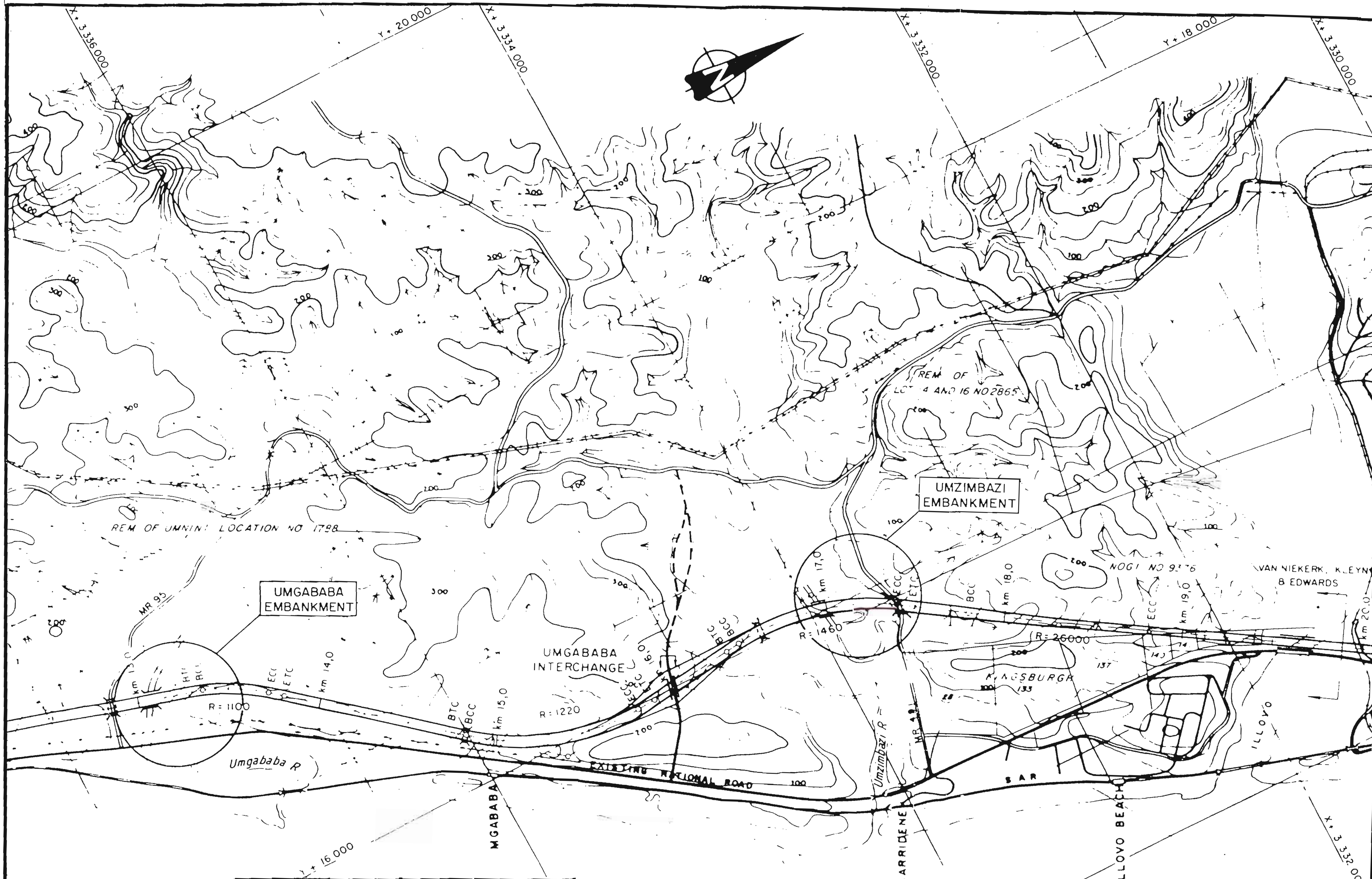
The approach adopted for the project is described below as four steps :

- The first step was to determine the field coefficients of compressibility (m_v) and consolidation (c_v) from back analysis of the measured settlements of one of the embankments, Umgababa. Since a large amount of laboratory data was available from the original design investigation some check on the validity of these back analysed parameters could be conducted.

- The second step in the process was to correlate the results of the new CPTU's with the back analysed m_v and c_v and hence obtain values for the constrained modulus coefficient, α_m, and the cone dissipation factor.

- The third step was to apply these values to the recent CPTU data for the other two embankments viz Umzimbazi and Umhlangane, in order to derive predicted settlements and times or c_v's.

- The fourth step was to compare these predictions with the measured settlement and time data.



NATIONAL ROUTE / NASIONALE ROETE	2	SECTION / SEKSIE	23,24

AMAHLONGWA RIVER – ILLOVO RIVER

KEY PLAN

PIEZOMETER CONE PENETRATION TEST (CPTU) RESEARCH

SCALE: 1:20 000

FIGURE No. D·4·1

There are a number of different ways in which the steps could be defined and combined, but they necessarily use the same basic data, ie the field settlement records and the recent CPTU results.

In addition to the evaluation of the compressibility and consolidation characteristics it was also possible to estimate shear strengths both undrained and effective from the CPTU results. Further comparisons were also possible of subsoil descriptions from the borehole records and from the CPTU using the Jones and Rust soils identification chart.

D4.2 Back Analysis for Umgababa

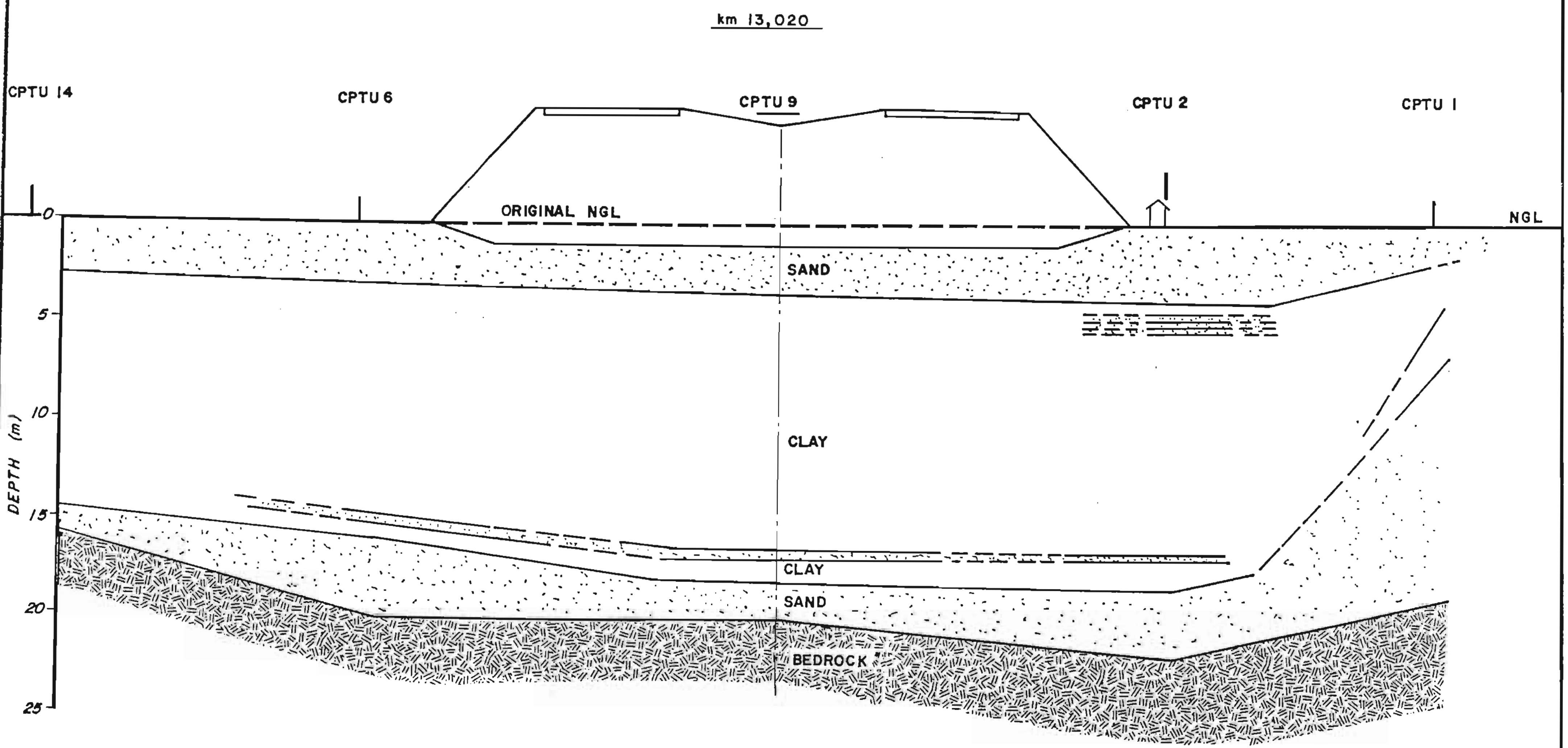
As stated earlier the first step in the process was to determine the appropriate field m_v and c_v by analysis of the settlement records. This analysis is described in the following sub sections D4.2.1 to D4.2.6.

D4.2.1 Drainage path length

For the purpose of this analysis the section at km 13020 was selected since the 1979 design investigation provided most information at this section. The geological cross section that was drawn up from the CPTU results is shown in Figure D4.2. From this it could be seen that the subsoil consisted of the following profile measured from the top of fill :

0	-	7,0 m	Predrilled
7,0	-	8,5	Silty sand
8,5	-	21,6	Clay
21,6	-	22,0	Sandy layer
22,0	-	23,1	Clay
23,1	-	24,0	Sand

The 0,4 m sand layer toward the bottom of the profile (21,6 to 22,0 below top of fill) acted as drainage boundary. This was apparent from the profile between CPTU 2 and 6, which showed the layer to be continuous with a different type of clay below the sand layer. Further evidence of the fact that this sand layer was a drainage boundary is the dissipation test carried out at 17,0 m in CPTU 2. This test showed that there was no



NATIONAL ROUTE / NASIONALE ROETE	2	SECTION / SEKSIE	23,24
GEOLOGICAL SECTION Umgababa km. 13,020			

PIEZOMETER CONE PENETRATION TEST
(CPTU) RESEARCH

SCALE:
HOR. 1:400
VERT. 1:200

FIGURE No.
D.4.2

excess pore pressure in this sand layer. Therefore it could be concluded that double drainage existed, that the drainage path length was 6,55 m ie half of the distance between the top sand and the lower sand layer, 8,5 m to 21,6 m, and that the total thickness of compressible clay was 14,2 m.

D4.2.2 Loading

The final design height of the embankment was 5,6 m above N.G.L. The settlement vs time data indicated that 1100 mm of the settlement took place before the final grading of the embankment. Therefore the total amount of fill at 13020 was at least 5,6 m plus 1,1 m ie 6,7 m.

The response of the pore pressure in the clay due to loading was governed by the following equation :

$$\Delta u = B\{\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)\} \qquad \text{D4.1}$$

If it is assumed that $\Delta\sigma_1 - \Delta\sigma_3 = 0$, ie no shear stresses are generated, then

$$\Delta u = B.\Delta\sigma_3 \qquad \text{D4.2}$$

From the Rowe Cell tests the B parameter was established to be unity. Therefore :

$$\Delta u = \Delta\sigma_3 \qquad \text{D4.3}$$

This meant that the initial pore pressure at instantaneous loading was equal to the stress placed on the clay by the embankment. This stress was calculated as follows :

$$\begin{aligned} \Delta\sigma &= \gamma_{nat} \times h \qquad \text{D4.4} \\ &= 22 \times 6{,}7 \\ &= 147 \text{ kPa} \end{aligned}$$

where γ_{nat} = the natural (unit) density of the fill material estimated to be 2200 kg/m^3

h = height of the placed fill (6,7 m)

Because of the width of the embankment no significant attenuation of stress with depth occurred, therefore at t_0 (30 June 1980) :

$$\Delta u = 147 \text{ kPa}$$

D4.2.3 **Settlement record**

Settlement records for the Umgababa embankment were taken during and after construction. Figure D4.3 shows the settlement up to February 1990 when it was 2,690 m.

D4.2.4 **Rate of settlement**

A series of settlement beacons were placed on the final road surface at the inner and outer edge of each lane. The level of these beacons were regularly measured by relating the levels to existing fixed benchmarks. Typical results of these surveys done in 1988 and 1989 were plotted to show the total settlement - Figure D4.4 as well as the rate of settlement in mm per month during this period, Figure D4.5. The latter showed the maximum rate of settlement to be 7 to 8 mm per month in 1988 - 1989.

D4.2.5 **Degree of consolidation**

Typical ambient pore pressures obtained from the dissipation tests are shown in Figure D4.6. The ambient pore pressure at 15,5 m was measured as 205,8 kPa with the water table at 3,3 m. Therefore the excess pore pressure, u_e, is given by the following :

$$\begin{aligned} u_e &= 205{,}8 - (15{,}5 - 3{,}3)\ 10 \\ &= 83{,}8 \text{ kPa} \end{aligned}$$

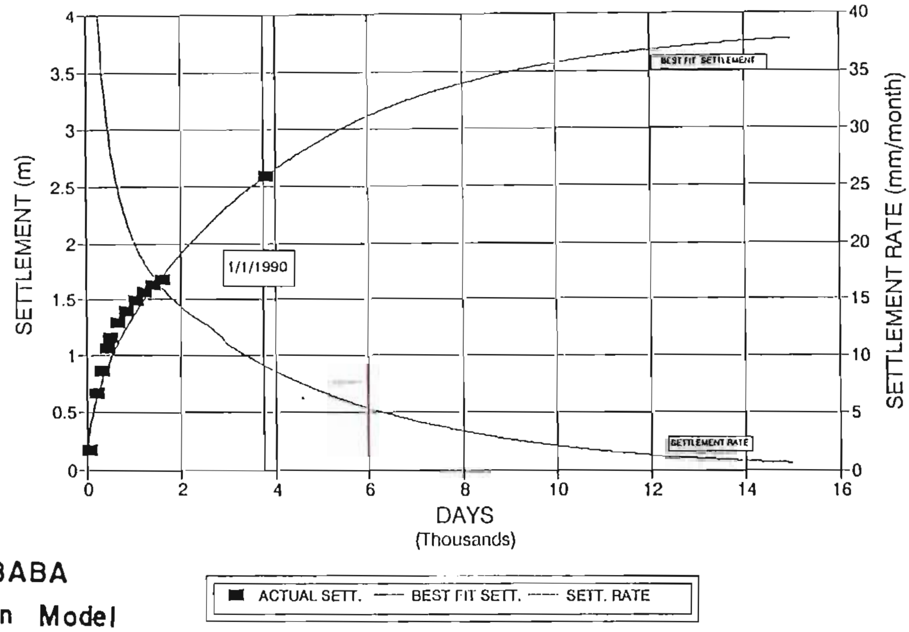
This test was carried out at 15,5 m which is 7,0 m into the clay layer. The normalized depth factor Z = (z/H)

Therefore :

$$\begin{aligned} Z &= (7{,}0/6{,}55) \\ &= 1{,}07 \end{aligned}$$

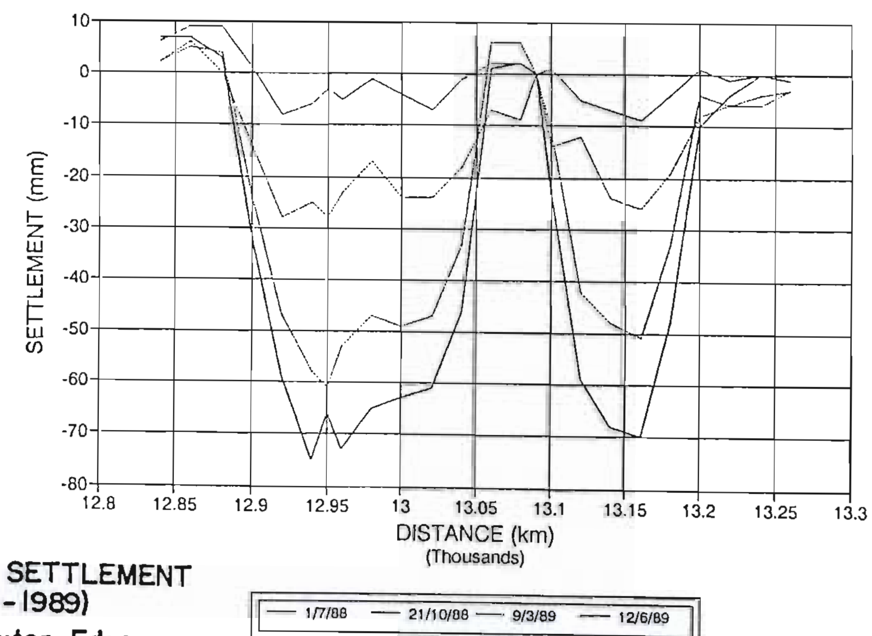
where z = depth into the clay layer

and H = drainage path length



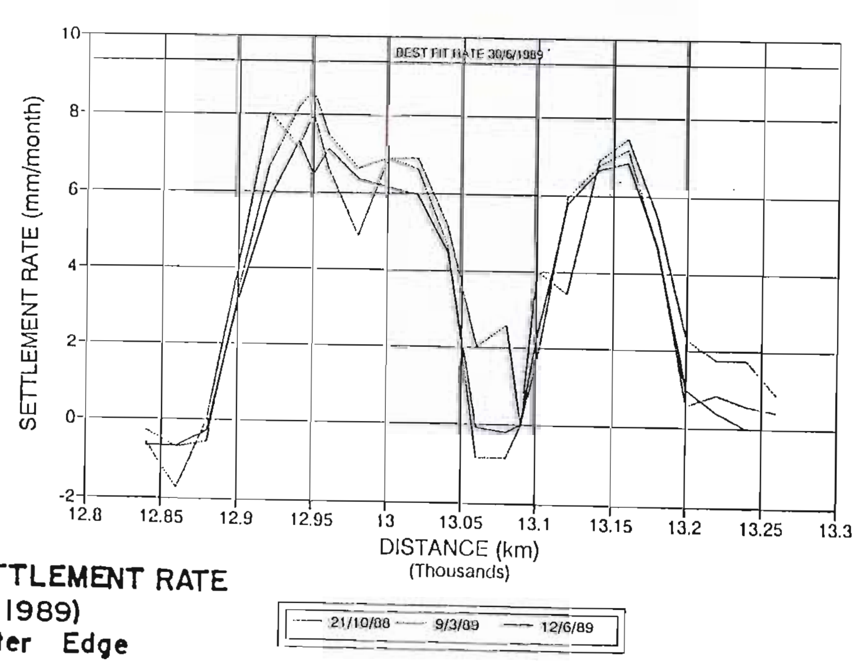
UMGABABA
Consolidation Model

Fig. D·4·3



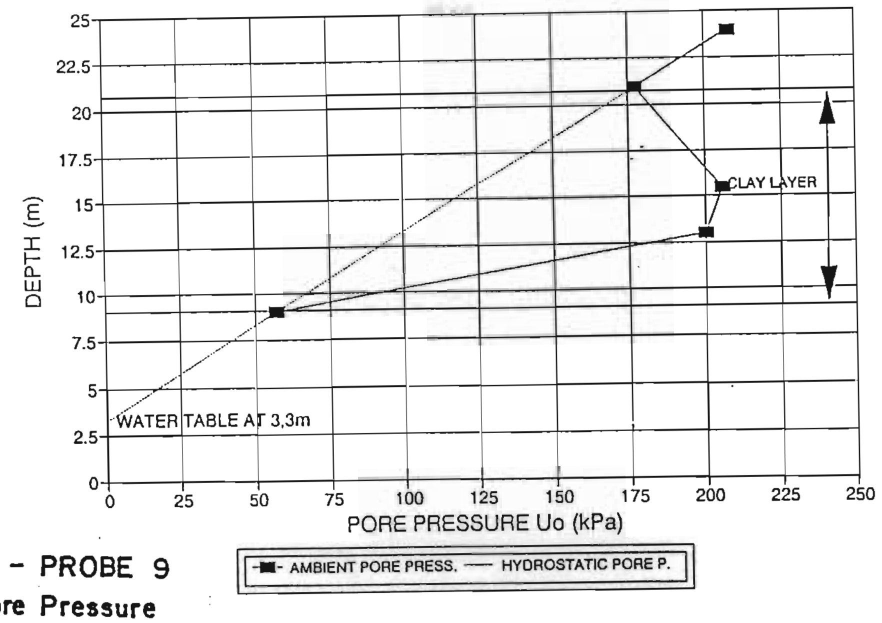
UMGABABA SETTLEMENT
(1988-1989)
NBC - Outer Edge

Fig. D·4·4



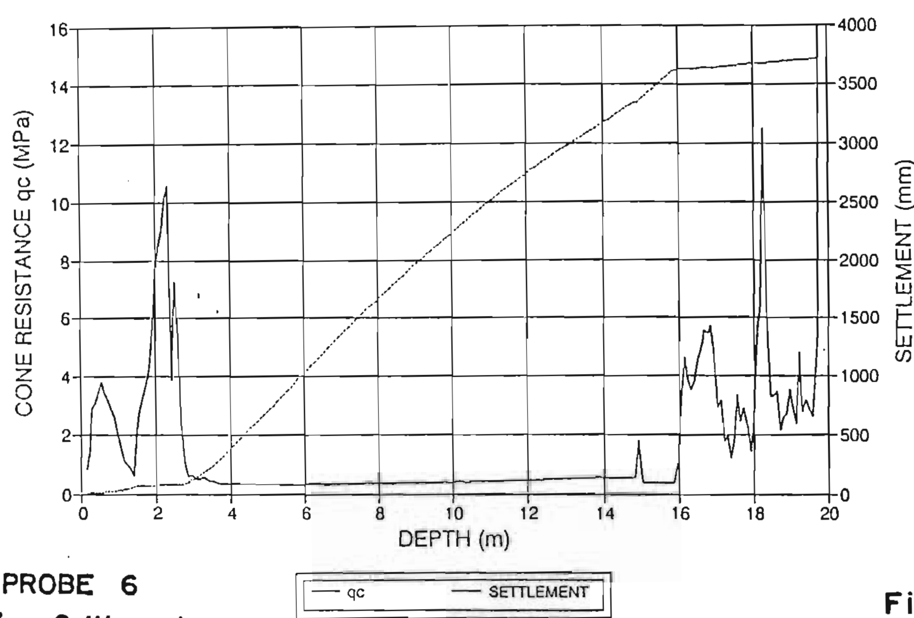
UMGABABA SETTLEMENT RATE
(1988-1989)
NBC - Outer Edge

Fig. D·4·5



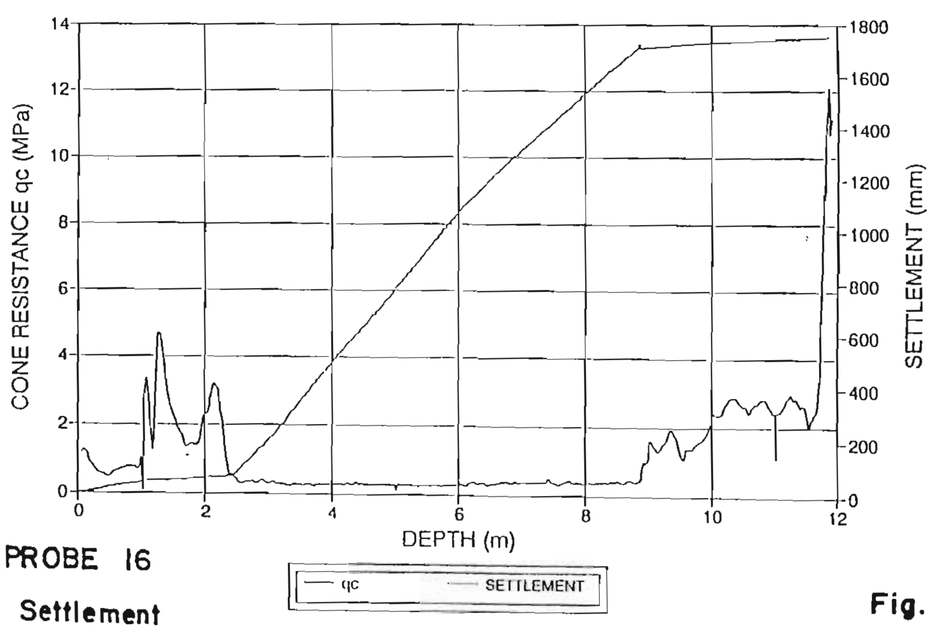
UMGABABA - PROBE 9
Ambient Pore Pressure

Fig. D·4·6



UMGABABA - PROBE 6
CPTU Cumulative Settlement

Fig. D·4·9



UMZIMBAZI - PROBE 16
CPTU Cumulative Settlement

Fig. D·4·10

The consolidation ratio at 15,5 m, or at z = 7,0 m, ie Z = 1,07 can be calculated as follows :

$$U_{1,07} = (147 - 83,8)/147$$
$$= 0,43$$

From the conventional Terzaghi consolidation chart U_z vs T_v and Z, Figure D4.7 it can be seen that :

where $Z = 1,07$ and
$U_z = 0,43$
then $T_v = 0,33$

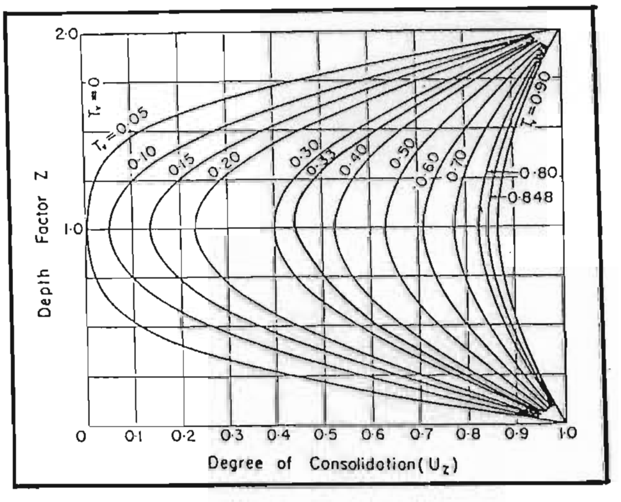


Figure D4.7 : Degree of consolidation v depth factor

From the usual equation relating U% and T_v or from Figure D4.8 it followed that the average degree of consolidation $\bar{U}$% was only 64,2% at the time of measurement, June 1989, some ten years after construction of the embankment.

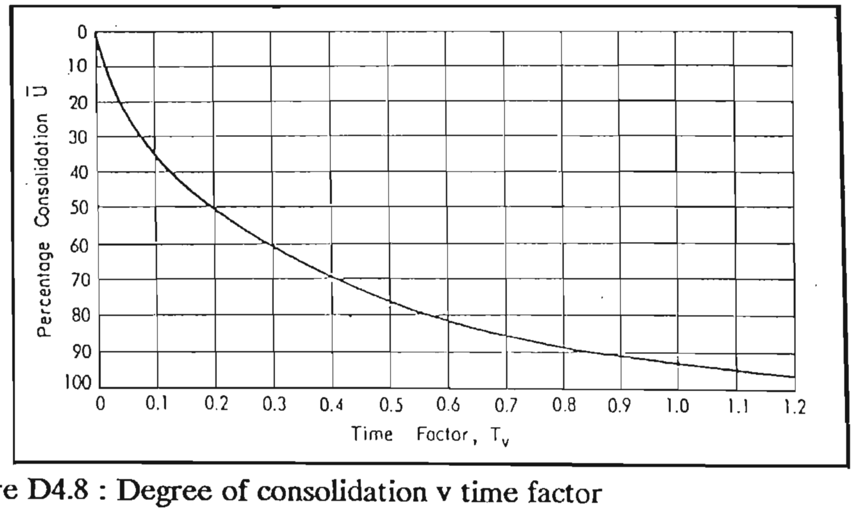


Figure D4.8 : Degree of consolidation v time factor

D4.2.6 Consolidation model

A consolidation model was fitted to the actual records of settlement, rate of settlement and degree of consolidation. Classical consolidation theory was used to establish the governing consolidation parameters. The physical characteristics of the Umgababa embankment were as follows :

Drainage path length	=	6,55 m
Compressible layer	=	14,6 m
Stress increase	=	147 kPa

It was estimated, on the basis of the cone penetration testing, that 96,5% of the measured settlement took place in the clay and the remainder in the sand. It was assumed, in accordance with Terzaghi's theory, that the consolidation parameters c_v and m_v, as well as the drainage path length, remained constant with increasing stress and strain.

A spreadsheet approach was adopted to enable multiple iterations of combinations of c_v and m_v to be carried out. The solution is given in Figure D4.3 showing the best fit for the settlement and rate of settlement.

The best fit solution gave the following consolidation parameters :

$$c_v = 1{,}5 \text{ m}^2/\text{year}$$

$$m_v = 1{,}8 \text{ m}^2/\text{MN}$$

The fit was clearly satisfactory despite the limitations imposed by the conventional simplifying assumptions of constant m_v and c_v. The model predicted that it would take 24 years for 90% consolidation to take place and that the total settlement would be 3,86 m. It followed that a further 1 m of settlement would take place over the next 14 years. These predictions for the total settlement and the time for 90% consolidation were very close to the equivalent predictions made in the comprehensive back analysis conducted by van Niekerk Kleyn and Edwards (1985).

D4.3 Settlement and Time Settlement Predictions from CPTU at Umgababa

The second step in the back analysis was to correlate the results of the CPTU with the back analysed m_v and c_v given in the previous section. CPTU data was required for the subsoil both under and outside the embankment. The former, as described in the previous section, was in order to estimate the excess pore pressures and hence degree of consolidation, and the latter to assess by interpolation the virgin conditions which were originally under the embankment.

D4.3.1 Compressibility correlation

For this purpose a total of eleven CPTU's were carried out at the Umgababa site, four of which were considered to be outside the influence of the embankment. The results of these as average q_c in the clay layer are :

Probe 2 , q_c = 0,447 MPa
Probe 6 , q_c = 0,431
Probe 7 , q_c = 0,465
Probe 14 , q_c = 0,453

Average q_c = 0,449 MPa

A typical CPTU result (Probe 6) is given as Figure D4.9. The q_c values increase with depth, but provided the layer thickness and depths and the rate of increase in q_c with depth are similar for all the probes, then it is valid to use average values for the full depth, because the relatively large width of loaded area compared with the depth imposes stresses in the clay layer which are practically constant with depth.

The back analysed m_v for Umgababa was 1,8 m^2/MN then since :

$$\frac{1}{m_v} = \alpha_m q_c \qquad \text{D4.5}$$

and m_v = 1,8 m^2/MN

and q_c = 0,449

therefore α_m = 1,24

As a check, this value of α_m was used to re-estimate the settlement for Probe 6 using the actual cone pressure data for each digitally stored reading ie at 20 mm depth intervals. The calculated settlement was 3,55 m compared with the measured (plus future estimated) of 3,86 m which was satisfactory noting that the Probe 6 average q_c (0,431 MPa) was lower than the average q_c (0,449 MPa).

D4.3.2 Consolidation correlation

The average CPTU measured t_{50} was 37 minutes and the back analysed c_v was 1,5 m^2/year. Therefore using the form of equation given below :

$$c_v = \frac{\text{cone time factor}}{t_{50}} \qquad \text{D4.6}$$

Cone time factor = 1,5 x 37

= 55,5 min ^{2}/year

This compared very closely with the Jones and van Zyl, (1981) value of 50 which was to be expected since the initial value (50) was derived from data from the original site investigation at this site, and from similar sites, and from cone dissipation tests subsequently carried out in 1980 - 1981.

D4.4 Application of Umgababa Derived Parameters to Umzimbazi and Umhlangane

The third step (see Introduction D4.1) was the application of the constrained modulus coefficient, α_m, (1,24), and the cone time factor (55) obtained from the Umgababa analysis to the other two embankments viz Umzimbazi and Umhlangane.

The fourth step was the comparison of the resulting predicted settlements and times with those measured at the two embankments.

D4.4.1 **Umzimbazi**

(a) Settlement

The CPTU showed that the average q_c value for the probes outside the embankment was 0,316 MPa.

$$M = 1/m_v = 1{,}24 \times 0{,}316$$
$$m_v = 2{,}55 \ m^2/MN$$

The original thickness of the clay layer, H, measured by the CPTU (and compared with the original investigation) was 6,95 m.
Therefore the estimated settlement was :

$$\Delta H = \Delta\sigma \times m_v \times H \qquad \text{D4.7}$$

where $\Delta\sigma$ is the stress due to the embankment at the centre of the clay layer : this is approximately 100 kPa since the fill density is 2200 kg/m^2 and the fill height was 4,6 m. Therefore :

$$\Delta H = 100 \times 2{,}55 \times 6{,}95$$
$$= 1{,}77 \text{ m (Figure D4.10)}$$

The measured settlement at this section in 1985 was 1,75 m. This was obtained primarily from the settlement records, but also from the CPTU carried out through the centre of the embankment which showed that the present clay thickness is 5,20 m ie 1,75 m less than the clay layer thickness outside the embankment.

(b) Consolidation

The CPTU gave an average t_{50} of 22,8 minutes, hence the coefficient of consolidation derived from this, using the Umgababa derived cone time factor of 55 is :

$$c_v = \frac{55}{22,8} \ m^2/year$$

$$c_v = 2,44 \ m^2/year$$

Back analysis of the settlement records gave an average or field c_v of about 4 m^2/year. Figure D4.11 shows the actual settlement data and a best fit single value c_v and m_v plot.

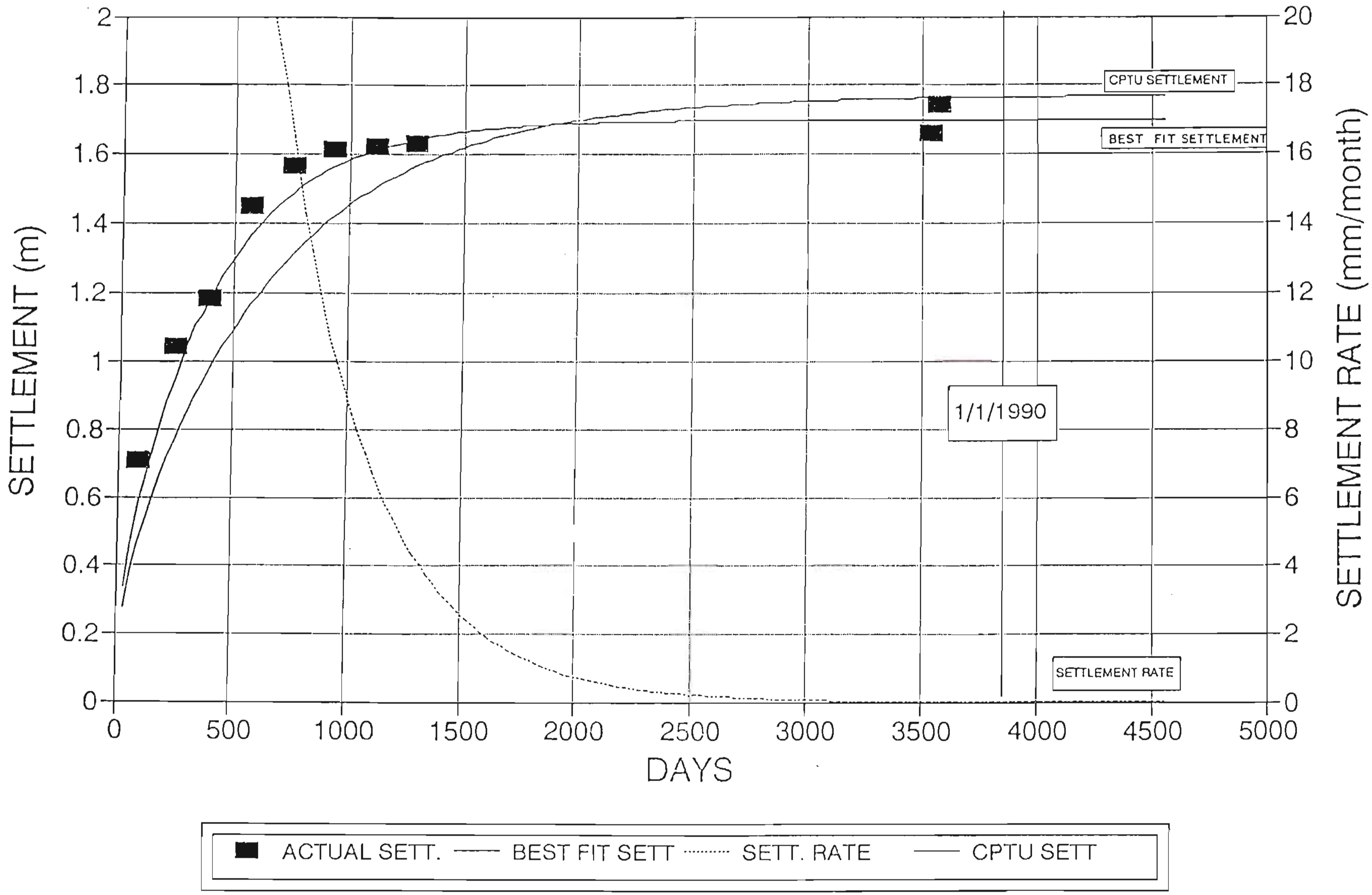
D4.4.2 **Umhlangane**

(a) Settlement

The CPTU showed an average q_c of 0,563 MPa hence :

$$M = \frac{1}{m_v} = 1,24 \times 0,563$$

$$m_v = 1,43 \ m^2/MN$$



SETTLEMENT AND RATE OF SETTLEMENT, UMZIMBAZI

Fig. D·4·11

The thickness of the clay layer was 12,0 m, the embankment was 7,8 m high with a density of 2200 kg/m^3, therefore the settlement estimated from the CPTU was :

$$\Delta H = \Delta\sigma \times m_v \times H$$

$$= 171 \times 1{,}43 \times 12 \text{ mm}$$

$$= 2{,}94 \text{ m}$$

The measured settlement was (1990) 2,79 m and the rate of settlement was about 1,0 to 1,5 mm per month. The CPTU showed there was a small excess pore pressure, and therefore it was reasonable to assume that part of the current settlement was consolidation and part secondary.

(b) Consolidation

The CPTU gave an average t_{50} of 30 minutes, hence the CPTU derived,

$$c_v = \frac{55}{30} \text{ m}^2\text{/year}$$

$$= 1{,}85 \text{ m}^2\text{/year}$$

Back analysis of the settlement records showed that reasonable data fits could be obtained with c_v in the range of about 2 - 6 m^2/year depending on assumption made regarding the present small excess pore pressure and the contribution of secondary settlement to the total.

D4.5 Summary of Results

The settlement and consolidation prediction data is summarized in Table D4.1 together with typical laboratory test data from the original site investigations.

Table D4.1 : Measured and predicted settlement data

Test Method	Umgababa	Umzimbazi	Umhlangane
Coefficient of compressibility, m_v, m^2/MN			
Lab 50 mm	1,67	0,82	0,53
Lab Rowe 112-224 kPa	0,93	1,06	0,47
Lab Rowe 224-392 kPa	1,53	0,92	0,39
CPTU predicted	1,80	2,55	1,43
Performance measured	1,80	2,36	1,38
Settlement, m			
CPTU predicted	3,86	1,79	2,94
Performance measured	3,86	1,66	2,85
Coefficient of consolidation, c_v, $m^2/year$			
Lab 50 mm	0,71	2,1	0,68
Lab Rowe 112-224 kPa	2,60	3,8	0,47
Lab Rowe 224-392 kPa	1,40	3,7	0,52
CPTU predicted	1,50	2,44	1,85
Performance measured	1,50	4	4 - 6

Table D4.1 shows that prediction of settlements at Umzimbazi and Umhlangane using the α_m value (1,24) derived from Umgababa gave a remarkably close estimate of settlements.

The laboratory tests average values of m_v gave very poor settlement predictions. The higher laboratory m_v values would, at Umhlangane, have given a reasonable prediction, but at Umzimbazi a poor prediction.

Table D4.1 also shows that the comparisons of CPTU predicted and measured coefficients of consolidation were better than the comparisons of laboratory and measured coefficients of consolidation.

The conclusions from the 1989 - 1990 research project were unequivocal and are stated below :

(a) **Settlement magnitude**

The data shows that the constrained modulus coefficient, α_m, (1,24) backanalysed from the Umgababa data gives excellent settlement predictions for the two similar embankments at Umhlangane and Umzimbazi.

(b) **Consolidation time**

The data shows that the cone time factor of 55 backanalysed from the Umgababa information gives satisfactory time predictions for the Umhlangane and Umzimbazi embankments.

(c) **Undrained shear strength**

The research also confirmed that undrained shear strengths, s_u, can be derived from the cone pressures in the usual way and that the conventional values of N_k viz approximately 15, gave satisfactory values.

(d) **Effective stress shear strength parameters**

From the piezocone data it was possible, in the apparently homogeneous clay layers of significant depth, to plot cone pressures against effective vertical stress and hence in a manner similar to that suggested by Janbu and Senneset (1974) to derive ϕ'. The data gave a value of 19° with a c′ of zero. Whilst this is not very close to the laboratory triaxial test value of about 25° - which would not in any case be expected since the type of test is very different, as are the initial stress conditions - it does provide a means of deriving equivalent triaxial ϕ' values from the CPT for the typical estuarine clays at these sites :

$$\tan\phi'_{cp} = 0{,}75 \tan \phi'_{tc} \qquad \text{D4.8}$$

where ϕ'_{cp} = CPT derived

ϕ'_{tc} = triaxial compression measured

More confidently, this approach can be adopted at any site to examine the variation of ϕ' with depth or position rather than to assess its equivalent triaxial compression value.

The most significant conclusion is that given in (a) which is that for the three embankments the constrained modulus coefficient for the soft clays is :

$$\alpha_m = 1{,}24$$

and that this value should be used to derive drained modulus E′ (or M) values for the clays from CPT cone pressure values, q_c, using :

$$E' = \frac{1}{\alpha_m q_c} \qquad \text{D4.9}$$

It must be emphasized, however, that this value of α_m is considerably lower than that given in the international literature which, for normally consolidated medium to low plasticity clays, and for q_c less than 700 kPa is in the range 3,7 to 10 and for highly plastic clays 2,5 - 6. In other words using the literature values would have only predicted about one third to a half of the settlements which have actually occurred. Examination of the laboratory measured m_v values given in Table D4.1 shows that they predicted only about half the total settlements which occurred at the Umzimbazi and Umhlangane sites. Therefore although there is excellent agreement on the appropriate value of α_m for the three embankments within this project, it remained to be shown that the use of this value is valid for other embankments.

The value is certainly appropriate for the prediction of total settlement for the three embankments (since it is obtained from back analysis of their performance), but the total measured settlements may have been due not only to primary consolidation, but also to other factors. Since at both Umzimbazi and Umhlangane partial stability failures took place during construction, then because of the high stress ratios the total settlements should be expected to include relatively large non consolidation components. Thus in cases of similar high stress ratios, (or whatever factors distinguish these embankments), the low value of α_m is appropriate for settlement predictions.

It should be emphasized, however, that this value of α_m is for the prediction of the total settlement and includes components due to local yield, immediate, primary and secondary settlement. In other words it allows prediction of the medium to long term worst case scenario.

Correspondingly, however, for situations where consolidation settlement is dominant, presumably where lower stress ratios are applied, then the α_m value of 1,24 may be over conservative.

The project concluded that an α_m value of 1,24 was appropriate for total settlement estimates for highly stressed soft clays which typically occur in the alluvial deposits along the Natal coast and that a cone time factor of 50 gives a satisfactory conservative estimate of consolidation time.

Further research was however, recommended at other embankments for which settlement records were available and a larger variety of subsoils were present, so that the applicability of the α_m value could be substantiated, or a range of appropriate α_m values obtained for different subsoils and stress conditions. This proposal was accepted by the Research and Development Advisory Committee of the South African Roads Board and a description of the resulting project is given in section D5.

D5 CPTU RESEARCH PROJECT, 1991 - 1992

D5.1 Introduction

The objective of the project was to determine constrained modulus coefficients and a cone time factor with sufficient confidence so that they could be used in practice. The approach adopted was to increase the data base from the three embankments of the 1989 - 1990 project described in D4 so that a variety of subsoils and embankment situations would be included.

The criterion for selection of the embankments was that adequate settlement records should be available. The methodology of the research was to carry out CPTU's at these selected embankments at positions which were not expected to be influenced by the settlement, but would yet be representative of the before construction conditions. This presupposed that original geotechnical investigation information would be available, and for those cases where the situation necessitated embankment monitoring this was almost inevitably so.
Using the settlement records a back analysed coefficient of compressibility, m_v was calculated and this was correlated with the CPTU results to obtain an α_m for each embankment.
In a similar manner a coefficient of consolidation, c_v, was back analysed for each embankment from the time settlement records and this c_v correlated with the dissipation times from the CPTU's.

Where sufficient information existed from the design geotechnical investigation, in particular laboratory consolidation test data, then further analysis was possible ie "predictions" based on this data and on the CPTU could be compared.
The expectation was that a range of α_m would be obtained and that the values would reflect the nature of the subsoil. In addition it was considered likely that the unusually low values of α_m deduced from the 1989 - 1990 research project could to some extent be due to the high stress ratios for those three embankments and the inclusion of secondary compressions in the measured settlements. It was also realized, however, that the selection process for the 1991 - 1992 embankments necessarily included only those embankments with adequate settlement records, and that there was a probability that these too would be potential problem sites and high stress ratios could apply.

A total of fifteen sites were selected from discussions with road authorities and consultants. In view of the time and budget constraints not all these could be utilized. A further selection process reduced the number of sites to eight at which it was believed the best records were available and which covered as wide a range of materials as possible. At some of these sites there was more than one embankment so that a total of sixteen embankment positions were included.

At each of these CPTU's were carried out, so that a total of 35 CPTU's were performed. During previous investigations at these sites, and at the 1989-1990 research project sites, 60 CPTU's and 60 mechanical CPT's had already been carried out, with the result that data from total of 155 cone penetration tests was available. The project began in June 1991 and was scheduled for one year. Obtaining and sifting though the records to select the embankments for testing took much longer than anticipated as did the final assembly and evaluation of the data from the selected sites with the result that the project will not be completed until the end of 1992. The information and conclusions given in the sections D5.3 and D5.4 are therefore taken from an evaluation of as yet unpublished data. Brief descriptions of the sites are given in the following section D5.2.

D5.2 Site Descriptions

D5.2.1 Introduction

Eight sites were selected and since some of these had either more than one embankment, or a long embankment over which conditions were significantly different, sixteen evaluation positions were available.

The project sites are listed below :

- Manzamyana - Richards Bay - one embankment
- Umlalazi - Natal N coast - two separate embankments
- Umhlatuze - Natal N coast - one embankment
- Prospecton - Durban area - one embankment
- Mzimkulu - Natal S coast - three positions at one embankment
- Goukamma - Cape E coast - one embankment
- Hartenbos - Cape E coast - three positions at one embankment
- Bot River - Cape S coast - four positions at one embankment

Descriptions of the sites are given in sub-sections D5.2.2 to D5.2.10.
Typical piezometer cone penetration test logs are given in Figures D5.1 and D5.2.

In addition, because construction at some of the embankments was divided into distinct stages resulting in significantly different stress conditions for each stage, it is considered valid to analyse these stages as separate cases. The total number of cases, including the three embankments for the 1989 - 1990 research project, is twenty five.

D5.2.2 Manzamyana

The project comprised an 850 m long viaduct to carry a 10 m wide harbour access road over railway lines and the Manzamyana canal. The approach embankment at the north end is only about 3 m high and has been monitored over a length of about 60 m adjacent to the piled viaduct. The recorded settlement over a five year period has been about 1,3 m which has necessitated remedial measures.
The subsoils consist of about 2 m of silty sands deposited recently by hydraulic filling; over 10 - 15 m of very soft silty clay (Lagoonal clay); over estuarine sands and silts; over firm to stiff silty clay (Estuarine clay); over silty sands; over stiff silty clay (Lower Lagoonal clay) to a depth of about 45 m - CPTU log Figure D5.1 (a).
Laboratory results from indicator, consolidometer and triaxial tests were available from the original, 1983, investigation.

D5.2.3 Umlalazi

The road project included two embankments relevant for the research project. One of these is the national road north approach embankment to the Umlalazi river bridge and the second, about half a kilometre to the north, is across part of the flood plain of the Umlalazi where a minor tributary enters.

The embankments are each about 200 m long and up to 8 m high. The original extensive investigations (1979) resulted in the prediction of large settlements together with stability problems during construction. Vertical sand drains and sand blankets were incorporated to reduce these problems.
Excellent monitoring results were available over a five year period from the start of construction until 1992 and these show that settlements of 1,3 m and 1,8 m occurred at

the first and second embankments. The subsoils at the first embankment comprise recent alluvium of sands, silty and clayey sands and clayey silts from 3 m to 16 m deep, the deepest being furthest from the present river course indicating that this had changed - CPTU log Figure D5.1 (b). At the second embankment the subsoils are shallower, being up to 7 m deep, and comprise finer sediments of clayey silt and silty clay. Comprehensive laboratory test results were available from the original 1979 - 1980 geotechnical investigation.

D5.2.4 Umhlatuze

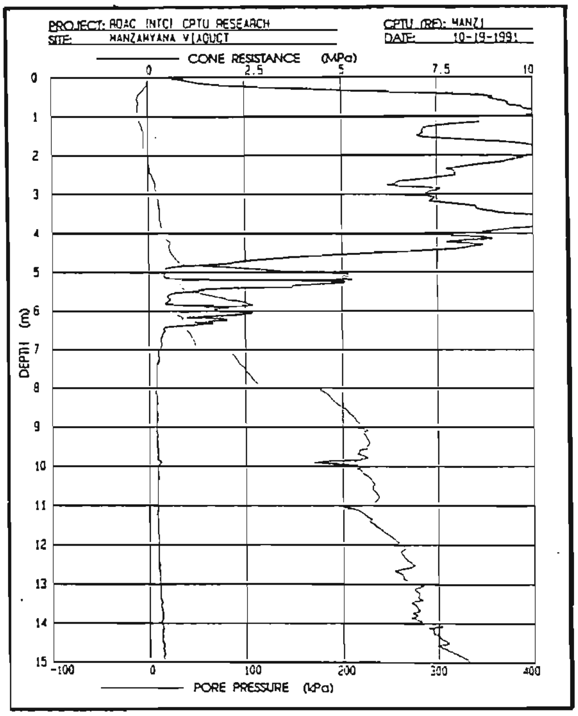
This site is unusual in that one section of the embankments for the national road across the wide Umhlatuze flood plain lay over a deep peat deposit, whereas most of the embankments are over the more common silty sands and clayey silts. The extent of the peat had not been fully determined at the early investigations, (1979), but subsequent settlement measurements during the preloading stage of construction highlighted the problem along the section of embankment included in the research project.

The relevant section of embankment is about 500 m long and up to 11 m high (including 3 m surcharge). Monitoring results were available, although many of the monitoring systems were destroyed when the embankment was partially washed away during a flood in 1987. The embankment has settled over 7 m (May 1992) and is continuing to settle at about 8 mm/month.

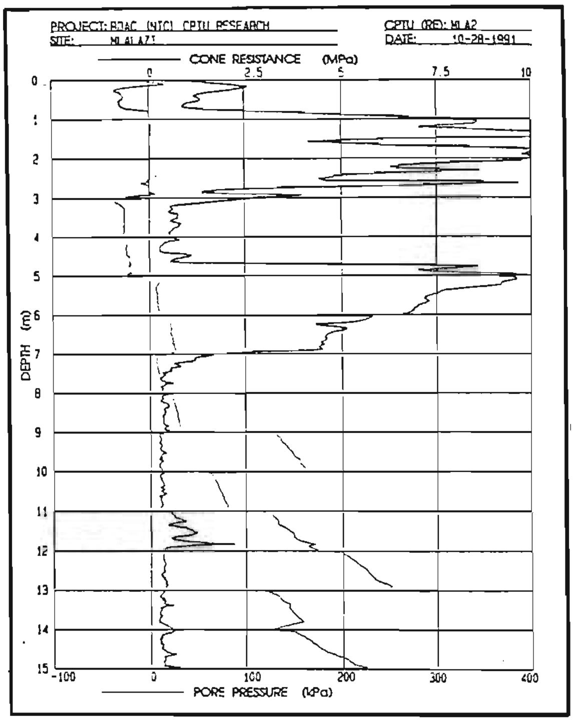
The subsoil consists of up to 13 m of peat over silty sand and silty clay layers to a depth below ground level of about 40 m - CPTU log Figure D5.1 (c). Comprehensive laboratory test results were available from a supplementary investigation carried out in 1987.

D5.2.5 Prospecton

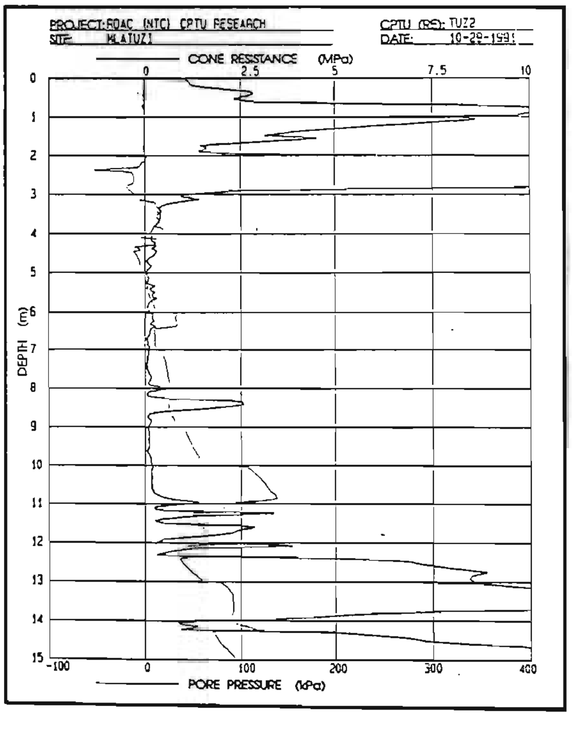
An interchange was required over the existing freeway south of the airport at Durban in the area of the Isipingo and Mbokodweni rivers flood plain. The national road is at existing ground level and the interchange structure approach embankments are about 8 m high. The area was developed from the flood plain over the past twenty five years to be an extensive industrial township by filling with about 3 m of Berea Red sand obtained from nearby hills. A history of foundation problems existed in the area due to settlements of the deep alluvial deposits.



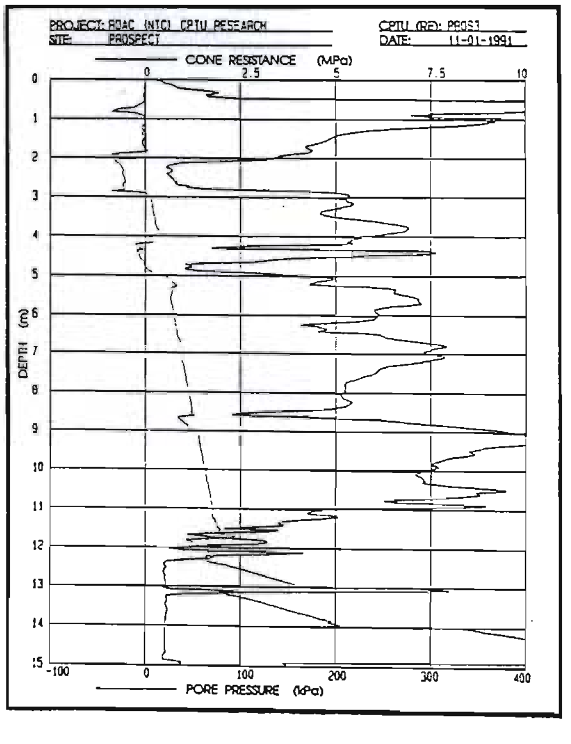
a) MANZAMYANA



b) UMLALAZI



c) MHLATUZE



d) PROSPECTON

TYPICAL PIEZOMETER CONE LOGS

Fig. D·5·1

The interchange approach embankments investigation was conducted in 1985 and consisted of 13 mechanical CPT's and six boreholes. The subsoils in the area are extremely variable even along the short lengths of the ramps to the interchange structure. They comprise erratic sands, silty sands, clayey sands and silty clay layers to depths of up to 35 m. At the settlement measurement position the soft clay layer which is about 12 m thick, is overlain by about 12 m of silty sand - CPTU log Figure D5.1 (d). From the results of the investigation, including laboratory tests, and from local geotechnical history it was decided that band drains, a sand blanket and surcharging were necessary to minimize post construction settlements. Construction took place in 1988 and settlements were monitored. These showed a maximum of 0,65 m, which was close to that predicted. Settlement stopped after about one year, which had been anticipated by the drain spacing design. The embankment was constructed in two distinct stages which allows separate analyses for these.

D5.2.6 **Mzimkulu**

The Mzimkulu embankment on the Natal south coast national road will carry the freeway over an existing railway line, which is on a fill, and across the Mzimkulu river. The embankment follows a subsidiary north-south old tributary valley to the Mzimkulu. The embankment is up to 15 m high and the relevant section is about 600 m long. The geotechnical investigation was carried out in 1981 and extensive laboratory test data was available. As a result of the analyses the embankment was constructed during 1987 - 1988 as a preloaded fill ahead of the final construction which has not yet been completed (1992).

Settlement records were available from three sections of the embankment and showed settlements of 0,36 m, 1,20 m and 1,65 m and these can be considered as separate cases. The subsoils comprise up to 17 m of layers of silty sands, clayey silts and silts which were extremely variable both laterally and longitudinally at the embankment position due to the sloping topography in the relatively narrow valley - CPTU log Figure D5.2 (a).

D5.2.7 Goukamma

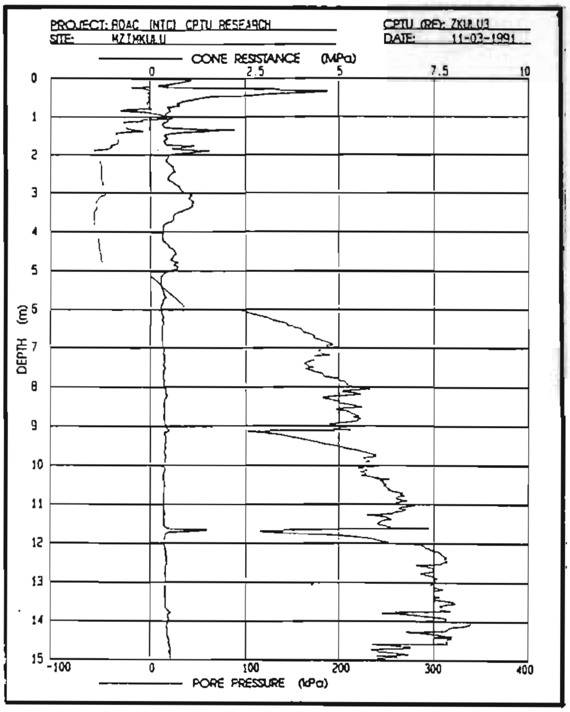
The national road through the Goukamma river valley in the Eastern Cape was improved by extensive realignment and the construction of a new river bridge. Local experience had shown that severe settlement problems could be expected across the flood plain and hence a comprehensive investigation was undertaken in 1985. Installation of vertical band drains, with a sand blanket, and staged construction were recommended to minimize the stability and settlement problems. Despite these precautions post construction settlements have occurred leading to severe cracking along one embankment at the southern end of the flood plain and significant settlements requiring repairs at the north approach embankment to the Goukamma river bridge. It is only the latter which is relevant for the research project.

The embankment is about 6,5 m high and 70 m long. Construction began early in 1986 and was completed about one year later, including a 2 m surcharge. The rate of construction was controlled by the monitoring of piezometers and settlements. The monitoring was discontinued one year after completion; however after a break of a further year surface settlement records were re-instituted because settlement had not stopped. These surface settlement records have continued until 1992 and the total measured settlement has been about 1,5 m. Because construction was divided into three distinct stages it was possible to analyse these separately.

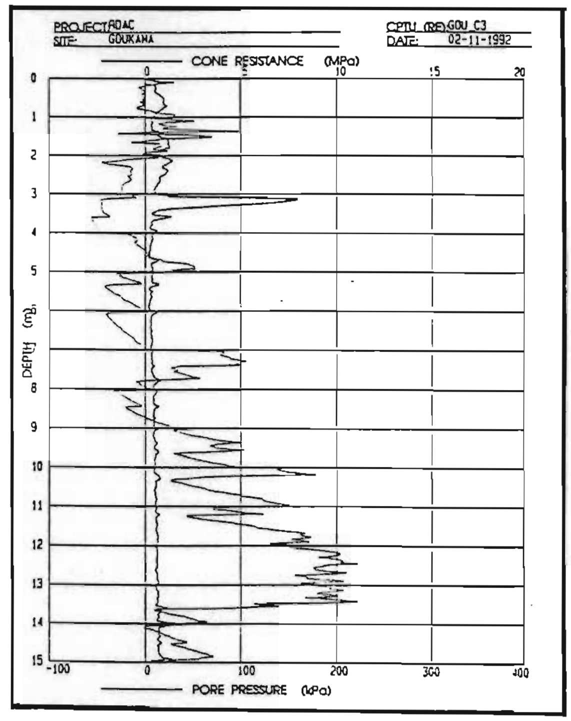
The subsoils consist of an upper 3 m thick layer of silty sands underlain by up to 20 m of very soft clayey silt and silty clay - CPTU log Figure D5.2 (b).

D5.2.8 Hartenbos

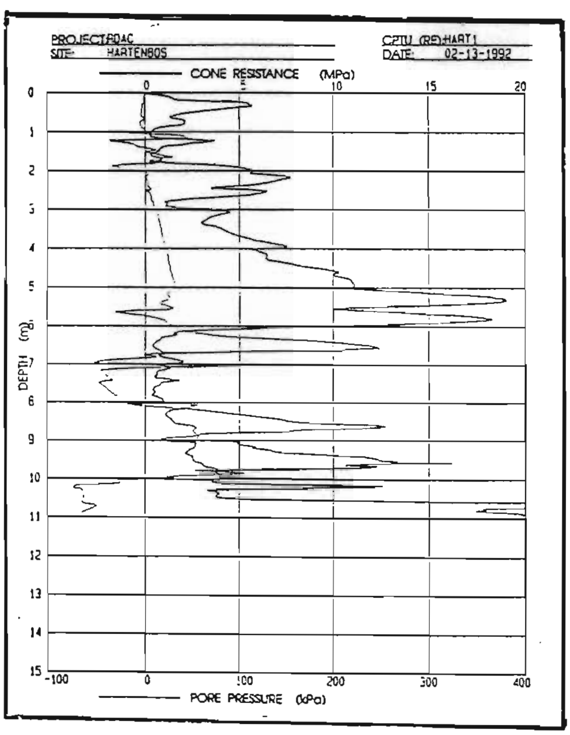
The Hartenbos embankment is on a Cape Provincial main road and is the approach embankment to the newly constructed bridge over the Hartenbosch river. The flood plain is about 1 km wide in this area and although the recent alluvium is generally sandy, soft clay layers about 1 m to 3 m thick were known to be present within the relatively shallow depth of about 10 m of alluvium - CPTU log Figure D5.2 (c). The investigation consisted only of cone penetration testing and from this, settlement predictions of up to 0,3 m were made for the embankment which is up to 4 m high.



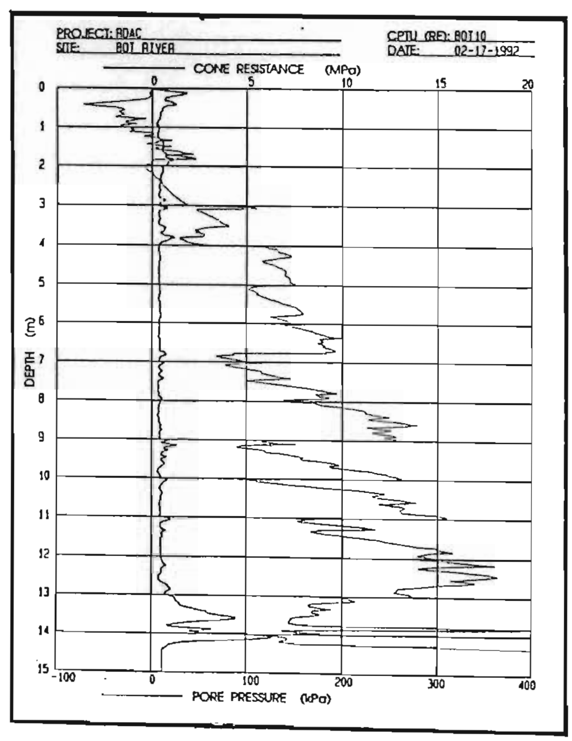
a) MZIMKULU



b) GOUKAMMA



c) HARTENBOS



d) BOT RIVER

TYPICAL PIEZOMETER CONE LOGS

Fig. D·5·2

The bridge was piled and shielded from lateral loading by stone columns through the alluvium. Settlements were monitored during construction and were only up to 0,15 m; nevertheless the embankment is considered to be relevant since it represents an example in the low stress range on relatively shallow alluvium with sandy clays of soft to firm consistency. Three of the positions monitored are considered to give a sufficient information to allow separate analyses of the settlement. However, because of the short time for settlement, the records are considered to be inadequate for any meaningful consolidation time deductions to be made.

D5.2.9 Bot River

The Bot River embankment is on a Cape Provincial Trunk Road and is the west approach embankment to the Bot River bridge. The embankment is about 0,5 km long with a constant height of about 6 m over the relevant section which is about 0,3 km long. The author, whilst at NITRR, assisted with the original site investigation in 1975, using the then new 100 kN imported standard Dutch CPT rig, with the standard mechanical friction sleeve cone and load measuring equipment. The investigation, which included boreholes, sampling and laboratory testing, the results of which are available, showed that severe stability and settlement problems were to be expected over the recent soft clay alluvium which is up to 18 m deep.

The embankment was therefore constructed in two stages, viz an initial 4 m followed by a further 2 m, and in addition a 1 m high surcharge was placed close to the position of the proposed bridge abutment to minimize subsequent lateral loads and negative skin friction on the bridge piles.

Extensive monitoring was conducted at four positions along the embankment and full records were available for the construction period from 1976 to 1981. Since there was a long delay between the first and second stages the information was sufficient to consider the embankment separately for these two stages.
The settlements along the embankment have varied from about 0,4 m at the western end to about 1,9 m close to the bridge abutment. Only small and even settlements have taken place since construction and no remedial measures have been necessary.

D5.3 Research Project 1991 - 1992 Site Investigation

During 1991 - 1992 each of the embankments described in the previous sub-sections was investigated using the latest piezometer cone penetration test equipment. This, as described in section C6, included a lap top computer in the field which enabled immediate production of cone and pore pressure logs and the creation of data files which has facilitated subsequent data processing.

A total of 35 CPTU's were carried out at the eight project embankment sites and typical logs for each site are given in Figures D5.1 and D5.2. A total of 75 pore pressure dissipation tests was carried out.

In addition to the information from this field investigation, both field and laboratory test results have generally been available from the geotechnical investigations carried out during the original road design projects some of which were more than fifteen years ago. It was a prerequisite for the selection of embankments for this project that settlement records should be available. In some cases excellent records were kept and two examples are given in Figures D5.3 and D5.4. In most cases the records have not been so comprehensive although they are adequate for the present purpose.

D5.4 Methods of Analysis of Results

The primary purpose of the project was to determine appropriate constrained modulus coefficient, α_m, values for the South African recent alluvial deposits by comparing measured settlements with CPT data.

The secondary purpose was to obtain data on the consolidation time characteristics by carrying out pore pressure dissipation tests. From these t_{50} times coefficients of consolidation could be estimated and compared with coefficients of consolidation calculated from back analysis of the measured rates of settlement.

Derivation of α_m

In addition to comparing the piezometer cone penetration test predictions with the measured embankment performance it was also possible to compare these CPTU predictions with predictions of settlement and times for settlement made on the basis of the original site investigation laboratory test data.

As discussed in D5.1, the method of analysis of the data was essentially a comparison of the new CPTU results with the back analysed coefficients of compressibility, m_v, and coefficients of consolidation, c_v, for each embankment in order to obtain α_m and cone time factors. The data set consisted of twenty two cases from the 1991 -1992 research project, together with the three sites from the 1989 - 1990 research project.

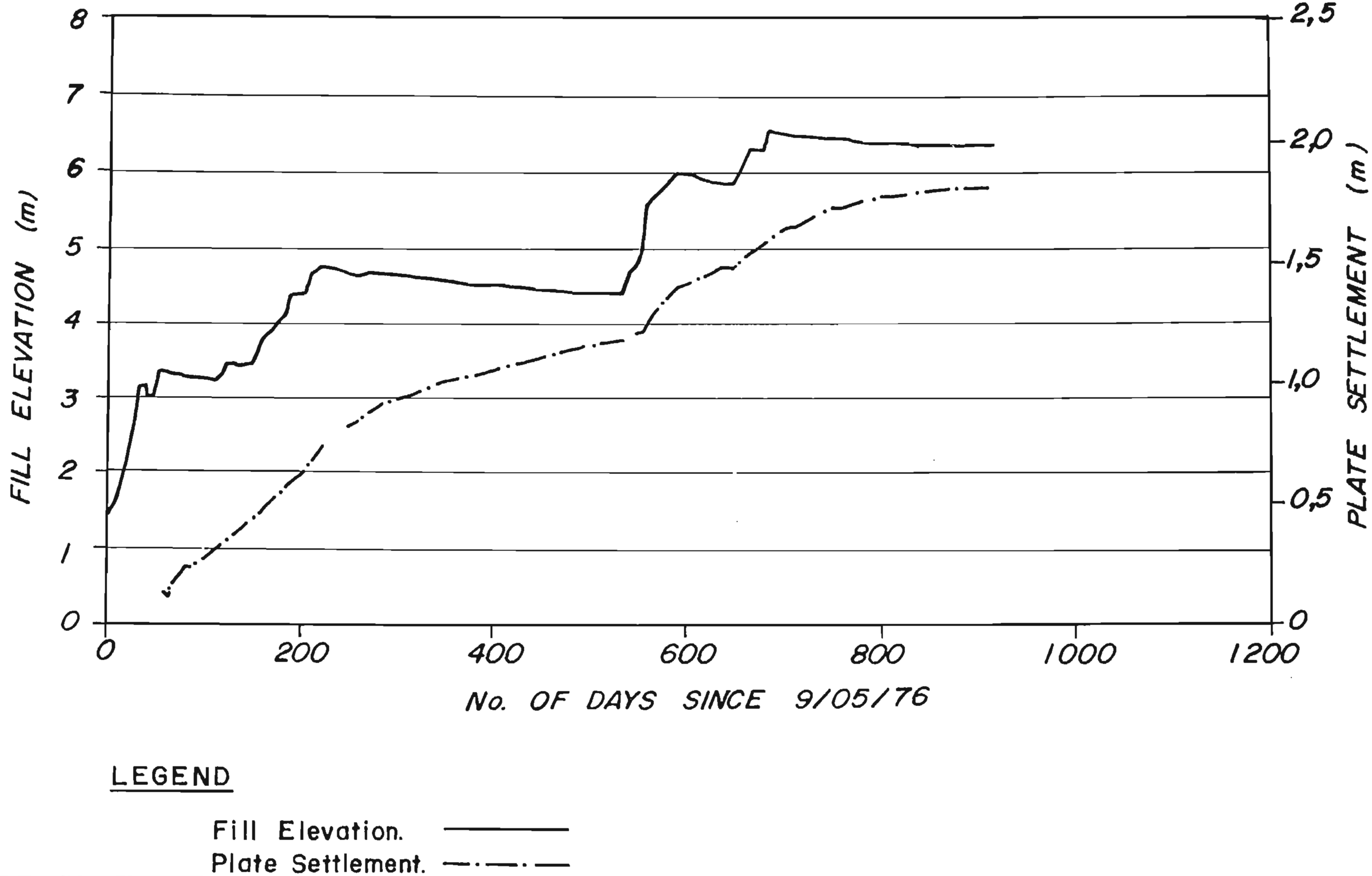
The subsoils are all recent alluvial deposits, varying from silty sands through sandy silts, clayey silts and silty clays to peats. It was anticipated that because of the wide material variations, and the cone pressure variations within any material type, then the α_m values could also have a wide range. If this was so, then it was hoped that the variations within this range could be rationalised using the materials information from the original site investigations viz natural moisture contents, void ratios, plasticity data, gradings, compressibility and undrained shear strength, together with initial and final stress ratios and the measured cone pressures.

Section B5 discussed settlement analysis for embankments and it was noted that the total settlement comprises components due to local yield, immediate settlement, primary consolidation and secondary compression. Although the international literature is not explicit on precisely which settlement parameter may be derived from CPT cone pressures, the consensus is that it should be the coefficient of compressibility, m_v. The implication of this is that it is only the immediate and primary consolidation components of settlement which are represented by the cone pressure derivation and that the remaining two components must be assessed by other means. It follows that back analysis of the measured embankment settlements must separate out those portions of the settlement due to secondary compression and local yield. In most cases it has been possible to separate any secondary compression because adequate time settlement data is available to allow modelling using Asaoka's method. This defines the end of primary consolidation, hence settlement in excess of this may be assumed to be due to secondary compression. From the settlement data available it is not possible to separate the non secondary compression into primary, immediate and local yield (where this is relevant). These can only be subdivided as described in section B5 by making various assumptions regarding the stress conditions, the relative values of the drained and undrained moduli and Poisson's ratio.

BOT RIVER

STATION 8 + 830

Plate no. 4 R



CONSTRUCTION AND SETTLEMENT RECORDS

CONSTRUCTION AND MONITORING RECORDS

UMLALAZI 2

FIGURE No. D·5·4

It must be emphasized, however, that for most of the cases examined the primary consolidation settlement is the dominant component and any approximations involved in ascribing different proportions to yield or immediate settlement are therefore relatively minor. Similarly, the secondary compression proportion for most of the embankments in the ten years or so since construction is also relatively small and can be adequately estimated from the time-settlement records and checked by laboratory consolidation test data where this is available.

The 1989-1990 research project (section D4), adopted the approach of deriving m_v from the total measured settlements (ie including any yield and secondary compressions). The α_m so derived of 1,24 was lower than those now derived in the 1991 - 1992 project which uses only the immediate and primary components of settlement.

A further modification was considered appropriate to the 1989 - 1990 project analysis and this concerns the embankment heights. All the embankments settled significantly during construction. This loss of height was made up during construction and therefore the effective height, ie that height which gives rise to the change in effective stresses in the subsoil, is the actual final height at the end of construction plus the settlement. The settlement takes place almost entirely below the water table, hence the increase in stress is due to the submerged density of the fill. It should also be noted that if the settlement records ended before the completion of consolidation then it is necessary to estimate the projected amount of settlement which would have occurred in this subsequent time. These modifications can be significant and it is necessary to take them into account. Each embankment has its own history of settlement and in a number of cases some ambiguity exists and some judgement has to be exercised. Fortunately where this has been necessary the potential errors are estimated to be less than say 10% of the total settlement.

This division of the total settlement into the components of yield, immediate, primary and secondary for the clays and immediate for sands can be expressed as follows to illustrate the steps in the analysis :

$$\delta_{total} = \delta_{measured} + \delta_{projectd}$$

$$\delta_{total} = \delta_{sand} + \delta_{clay}$$

(i) Used fixed value α_m for sand to determine δ_{sand}

$$\delta_{clay} = \delta_{yield} + \delta_{immediate} + \delta_{primary} + \delta_{secondary}$$

(ii) Assume that the yield is a function of immediate settlement depending on the initial shear stress ratio and the applied stress ratio, (see Figure B5.6, p93).

$$\delta_{yield} = a \times \delta_{immediate}$$

(iii) Assume the immediate settlement is a function of the primary consolidation, (see Figure B5.4, p91) :

$$\delta_{immediate} = b \times \delta_{primary}$$

(iv) Use the settlement records (Asaoka) to determine $\delta_{secondary}$:

$$\delta_{secondary} = c \times \delta_{primary}$$

The settlement predicted by the CPT derived m_v is the immediate plus primary components, so :

$$\delta_{CPT} = \delta_{immediate} + \delta_{primary} = \delta_{clay} - \delta_{yield} - \delta_{secondary}$$

$$= \delta_{clay} - a\,\delta_{immediate} - c\,\delta_{primary}$$

For highly stressed subsoils "a" may be say 0,7, (see p.93), and "b" may be say 0,2 for a relatively wide embankment (see p.91) and for organic soils say five years after completion of construction assume "c" may be say 0,1:

Therefore :

$$\delta_{CPT} = \delta_{immediate} + \delta_{primary} = \delta_{clay} - 0{,}2 \times 0{,}7\ \delta_{primary} - 0{,}1\ \delta_{primary}$$

or $$\delta_{CPT} = 0{,}83\ \delta_{clay}$$

It may be seen therefore, that even where local yield is fairly high and immediate settlement is a high proportion of primary consolidation and where secondary compression is a significant portion of the measured settlement, the overall error in ignoring these would not be very high (17%). It would result in a conservative value of α_m as in the 1989 - 1990 research project.

A further consideration could also be applied which is illustrated in Figure B5.3, p91. This is that depending on the situation geometry and the Poisson's ratio, the error in assuming that settlement is one dimensional as opposed to three dimensional could be significant, as shown in the Figure. In practice, however, if the embankments are wide relative to the subsoil thickness, and if Poisson's ratio is less than 0,4, then the error will be less than 5%, which may be considered as negligible.

The measured settlements, ie including yield and secondary, and the modified settlements are used to calculate the initial α_m and the final α_m. The method of calculation is based on the conventional equation for each layer :

$$\delta = \frac{\sigma \times H}{\alpha_m\, q_c} \qquad \text{D5.1}$$

Since the cone pressures are recorded during penetration testing in a digital data file it is convenient to use this file as the basis for the calculations by computing the change in stress due to the embankment load at each cone data depth. A program was written for this and for summing the increments of settlement in each layer for an assumed α_m value. By successive iterations of α_m the calculated settlement was matched to the measured settlement so that the appropriate value of initial α_m was obtained.

The final α_m was calculated by modifying the initial α_m in direct proportion to the ratio of measured to modified settlement.

As well as calculating α_m as described, values can also be derived by directly comparing cone pressures with coefficients of compressibility, m_v, obtained from the laboratory testing carried out during the initial site investigations.

Although ideally these α_m values would be identical, implying that the initial m_v's would have predicted the precisely correct settlement, the reality is that the two sets of α_m would be expected to have different values. This is because the laboratory m_v's represent specific samples of the subsoil, whereas the cone penetration test derived moduli represent an average of the subsoils at each CPT position. It should be emphasized that the q_c values are not averaged throughout the depth, nor are the settlements, but the distribution of settlements within the total subsoil depth is assumed to be proportional to the imposed stresses and cone pressures.

This issue of the potential difference in the two sets of α_m values, ie measured settlement derived and laboratory test derived, could be of considerable significance since they are both designated α_m yet are derived from entirely different data. It would be preferable to give them separate designations viz α_{mv} for the laboratory m_v and CPT correlation and α_{mba} for the back analysed version.

Practically all α_m values given in the literature have been derived by comparison of cone pressures with laboratory data, although corroboration is occasionally given with settlement records. The latter of course suffer the difficulty mentioned above of averaging in a multilayered system, unless measurements of settlements are available at different depths.

The approach adopted for this research has been that evaluation of the subsoil compressibility can be made directly from cone penetration testing. If this is shown to give reliable settlement predictions then it is not necessary to invoke correlations with laboratory data, although of course this does not suggest that settlement predictions should not be made from laboratory data. The two processes are independent but complementary.

Clearly the implication of this is that any correlations derived between cone pressures and compressibility are limited to the range of materials and stress conditions at the sites investigated. Within the South African conditions this is not a significant limitation, but it would be unjustified to extrapolate these results to different soils under different stresses. For this reason considerable basic soils data from the sites is given in section D5.5 so that the possibility of justifiable extrapolation is enhanced.

Despite this emphasis on the correlation of cone pressures and measured settlements to obtain α_m, similar direct correlations were also made with laboratory coefficients of compressibility. This has the potential benefit of providing a bridge between the two approaches and gives a basis for suggesting that the results could be more reliably extrapolated to other materials and stress conditions. In making the comparison between the laboratory m_v's and cone pressures it is first necessary to show that the m_v's are reliable. To do this, settlement estimates have to be made on the basis of the laboratory data and compared with the modified settlements, ie after making allowance for any local yield and secondary compressions. If the comparison is satisfactory an appropriate cone pressure has to be selected for correlation with the m_v. Ideally this would be from a CPT at the position of the original consolidometer sample. This was not possible because the original boreholes were generally on the line of the proposed embankment which has been subsequently constructed. Not only is the position effectively inaccessible, but the embankment has materially changed the subsoil stress conditions and hence the cone pressures. Compromise is necessary and that adopted is that the cone pressure used is that at the same depth as the laboratory sample.

The results of the α_m analyses and discussion of these are in section D5.5.

The derivation of coefficient of consolidation is described in D5.5 Discussion of Results, in subsection D5.5.2, p222.

D5.5 Discussion of Results

Summaries of the results of the 1991 - 1992 research project are given in tabular form in the following three subsections dealing with settlement (α_m), consolidation time (c_v), and soil parameters respectively.

D5.5.1 Constrained modulus coefficient, α_m

The back analysis of the embankment settlement records together with the cone penetration test results allows two sets of α_m values to be calculated as described in D5.4. The initial set comprises α_m values based on the total measured settlements; the final set is modified by deducting the estimated local yield, secondary compression and sand settlements where these are applicable. The results are given in Table D5.1 The means and standard deviations of both sets of α_m values are given at the bottom of the Table.

In order to give an idea of the overall situation at each embankment the height, clay thickness and cone pressures are listed in the Table.

The height is the height above original ground level, ie it excludes the amount subsequently allowed for in the calculations for the settlement.

The clay thickness is the total soft clay thickness before compression and is not necessarily in a single layer. The cone pressure is a typical value in the depth zone through which a slope stability failure would probably pass if failure were to take place. This typical value is not used in the back analysis and only serves to illustrate the shear strength of the clay.

In addition to the α_m values from the settlement back analyses, values may also be obtained by direct comparison of laboratory coefficients of compressibility, m_v, and cone pressures using :

$$\alpha_m = \frac{1}{m_v \, q_c}$$

These values are shown in Table D5.2 in the Lab/CPTU column.

For comparative purposes the α_m values from settlement back analyses from Table D5.1 are given in the Measured α_m column (Table D5.2) and expressed as a ratio to the Lab/CPTU α_m in the last column. The means and standard deviations of the α_m's and their ratios are given at the bottom of the Table. Figure D5.6 (a) shows the two sets of α_m values and the equality line which is the best fit if the Umgababa data is excluded, ie the α_m calculated from back analyses of the settlement is approximately equal to α_m calculated from the laboratory m_v and cone pressures.

Examination of Table D5.1 shows that the mean initial α_m value is 2,31 compared with the final α_m value of 2,63. The difference of the means is less than 10%, but is misleading because the individual site differences are up to 30% and the mean of the ratios is 1,15. These results demonstrate that at some of the sites the local yield and secondary components of settlement are significant, but that on average, on these alluvial deposits, these components add only about 15% to the settlements.

This observation, although generalized, has value in that it allows the deduction to be made that if no other basis exists for the prediction of secondary compression, then an allowance for say 10% for a 10 year period would be reasonable, but that up to say 25% is possible.

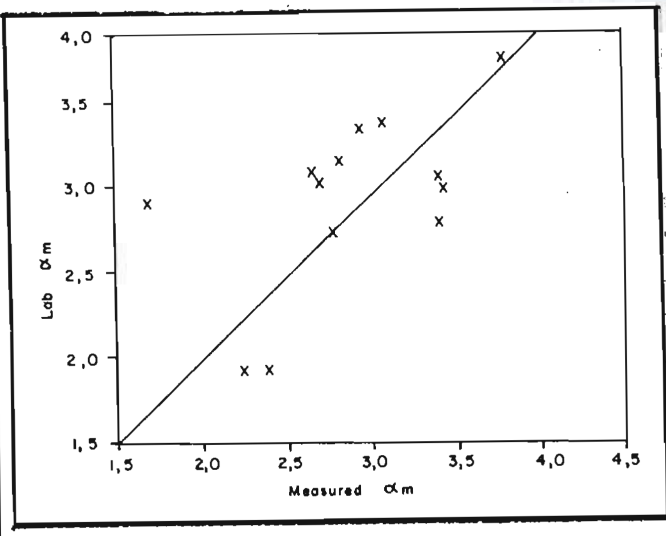
The estimates of local yield, which are based primarily on assumptions regarding Poisson's ratio and the initial stress ratios, show that only a few cases is the local yield significant. These cases are where the cone pressures, or undrained shear strengths, are relatively low and may be predicted on this basis. Alternatively an average of 5% could be included in predictions in addition to the average 10% for secondary compression. For embankments the amount of local yield is not generally practically significant because it occurs during construction and is compensated for. It does not therefore cause long term problems, rather only the contractual problem of the cost of the level compensation.

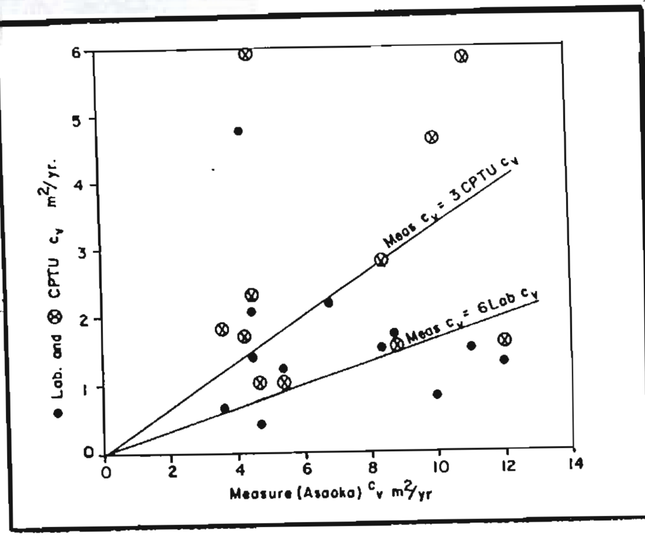
Table D5.1 : Constrained modulus coefficients, α_m, from settlement analyses

Embankment		CPTU	Height m	Clay Thickness m	Typical Cone Pressure kPa	Measured Settlement	Initial α_m	Yield and Secondary m	Final α_m
Manzamyama			2,50	11	250	1,265	1,96	0,450	3,04
Umlalazi 1		Mla 1 Mla 2	8,4	14	350 500	1,30	2,95 2,05	0,165	3,38 2,35
Umlalazi 2		Mla 4	6,0	7	210	1,80	2,59	0,400	3,33
Umhlatuze		Tuz 1 Tuz 2 Tuz 4	11,2	15 12,5 7,5	300 250 175	7,37	2,87 2,88 2,32	0,500	3,08 3,09 2,49
Prospecton	Full	Pros 3	8,0	12	650	0,65	2,67	0,070	2,99
	Part	Pros 3	5,0	12	650	0,40	2,69	0,020	2,83
Mzimkulu	220	Kul 1	15,0	14	800	1,650	2,52	0,120	2,72
	360	Kul 4	13,0	4	1200	0,360	3,76	0	3,76
	520	Kul 3	10,0	13	800	1,200	2,08	0,075	2,22
Goukamma	Full		6,5	20	850	1,510	2,32	0,345	3,01
	Part		4,0	20	850	0,670	3,15	0,020	3,25
	Part		1,8	20	850	0,270	3,32	0,015	3,51
Hartenbos	290		2,9	2,5	780	0,130	2,18	-	2,18
	240		3,4	3,5	875	0,120	2,97	-	2,97
	170		4,0	5,0	875	0,150	3,60	-	3,60
Bot River Full Ht	830 L	Bot 9 Bot 10	5,0	14,0 12,0	375 420	1,81	2,58 2,31	0,265	3,15 2,83
	830 R	Bot 6 Bot 7 Bot 8	5,0	11,0 14,0 14,0	500 540 500	1,85	1,77 1,88 1,98	0,270	2,07 2,14 2,27
	730 R	Bot 5	5,4	11,0	420	1,55	2,32	0,230	2,79
	630 R		5,0	9,0	900	0,49	2,20		2,30
Bot River Part Ht	830 L	Bot 9 Bot 10	3,0	14,0 12,0	375 420	1,25	2,29 2,05	0,20	2,41 2,16
	830 R	Bot 6 Bot 7	3,0	110 14,0	500 540	1,35	1,51 1,59	0,060	1,64 1,73
	730 R	Bot 5	2,9	11,0	420	1,20	1,63	0,020	1,73
Umgababa		Gab 2 Gab 6 Gab 7	5,6	13,0	450 430 460	2,6	2,17 1,88 2,32	0,600	2,97 2,57 3,17
Umzimbazi		Baz 12 Baz 15 Baz 16	4,0	6,9	320	1,75	1,72 1,51 1,43	0,250	2,12 1,86 1,76
Umhlangane			7,0	12,0	560	2,94	1,40	0,640	1,91
						Mean	2,31		2,63
						Std Dev	0,60		0,59
						$\frac{\text{Final } \alpha_m}{\text{Initial } \alpha_m}$	Mean = 1,15 Std Dev = 0,13		

Table D5.2 : Constrained modulus coefficients α_m from laboratory and CPTU data

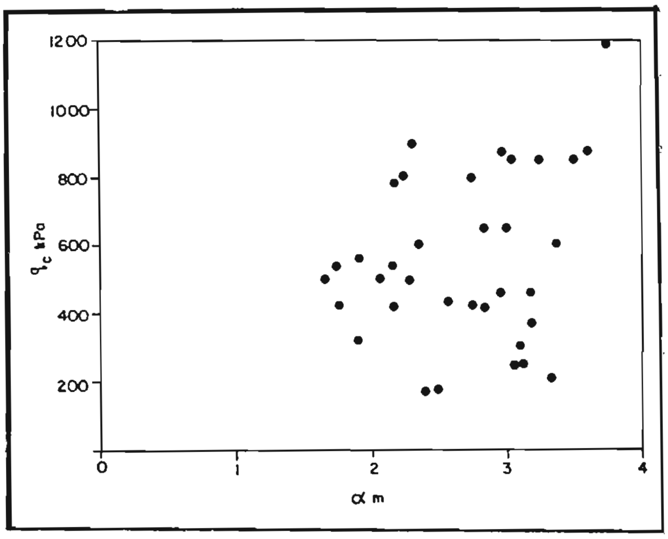
Embankment	m_v m^2/MN	q_c MPa	Lab/CPTU α_m	Measured α_m	$\frac{\text{Meas } \alpha_m}{\text{Lab CPT } \alpha_m}$
Manzamyama	1,2	0,2	3,40	3,04	0,89
Umlalazi 1	0,9	0,350	3,08	3,38	1,10
Umlalazi 2	1,2	0,500	2,94	3,33	1,08
Umhlatuze	1,5	0,250	2,67	3,09	1,16
Prospecton	0,4	0,650	3,42	2,99	0,87
Mzimkulu	0,4	0,800	3,79 2,78	3,76 2,72	0,99 0,98
Goukamma	0,6	0,850	2,70	3,01	1,11
Bot River	0,95	0,375	2,81	3,15	1,12
Bot River	0,7	0,420	3,40	2,79	0,82
Umgababa	1,26	0,450	1,59	2,90	1,82
Umzimbazi	1,4	0,320	2,23	1,91	0,86
Umhlangane	0,75	0,560	2,38	1,91	0,80
	Overall	Mean	2,86	2,92	1,05
		Std Dev	0,59	0,52	0,26
	Excluding Umgababa	Mean	2,97	2,92	0,98
		Std Dev	0,46	0,55	0,13

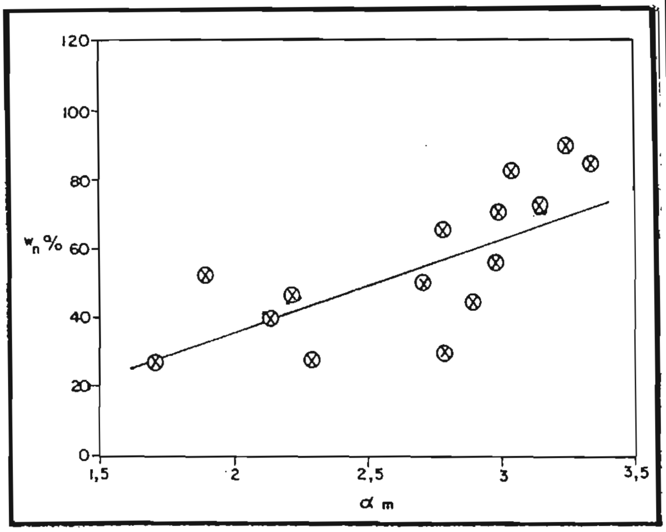




a) Settlement measured α_m v Laboratory derived α_m

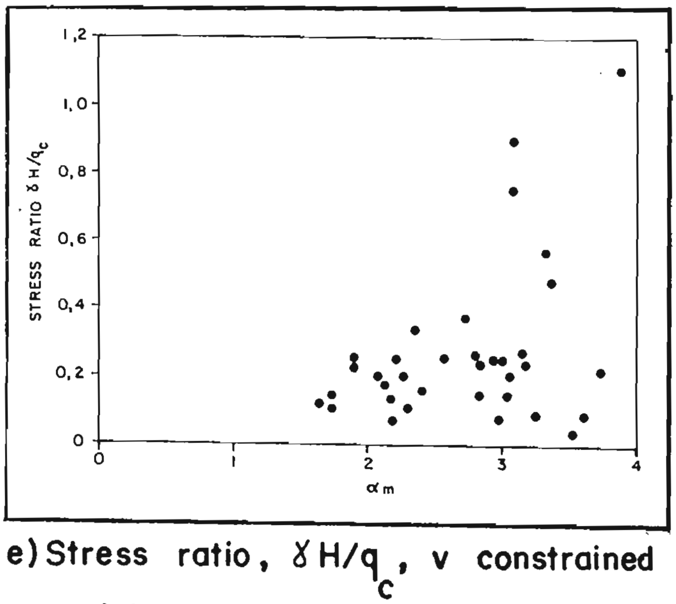
b) Laboratory, CPTU and settlement measured C_v.





c) Cone pressure, q_c v constrained modulus coefficient, α_m

d) Moisture content, w_n v constrained modulus coefficient, α_m



e) Stress ratio, $\gamma H/q_c$, v constrained modulus coefficient, α_m

Fig. D·5·6

D5.5.2 **Coefficients of Consolidation, c_v**

As indicated at the beginning of this section the secondary purpose of the 1991 - 1992 research project was the derivation of consolidation characteristics from piezometer cone penetration testing.

The field technique adopted was that which is now accepted internationally and first advocated by the author in the literature (Jone and van Zyl, 1981). Cone penetration is stopped at selected depths and the time for half dissipation, t_{50}, measured. Half dissipation t_{50} is defined as the time at which the initial excess pore pressure, u_o,(at time t_o) has dissipated to u_{50} half way towards the final pore pressure, u_{100}. At all the embankments in this project the water level was close to ground level hence u_{100} is hydroststatic and is simply calculated from the depth at which the dissipation test is conducted. The value of u_{50} can therefore be readily determined as soon as u_0 is registered on ceasing penetration and the subsequent time taken to reach pressure u_{50} is measured.

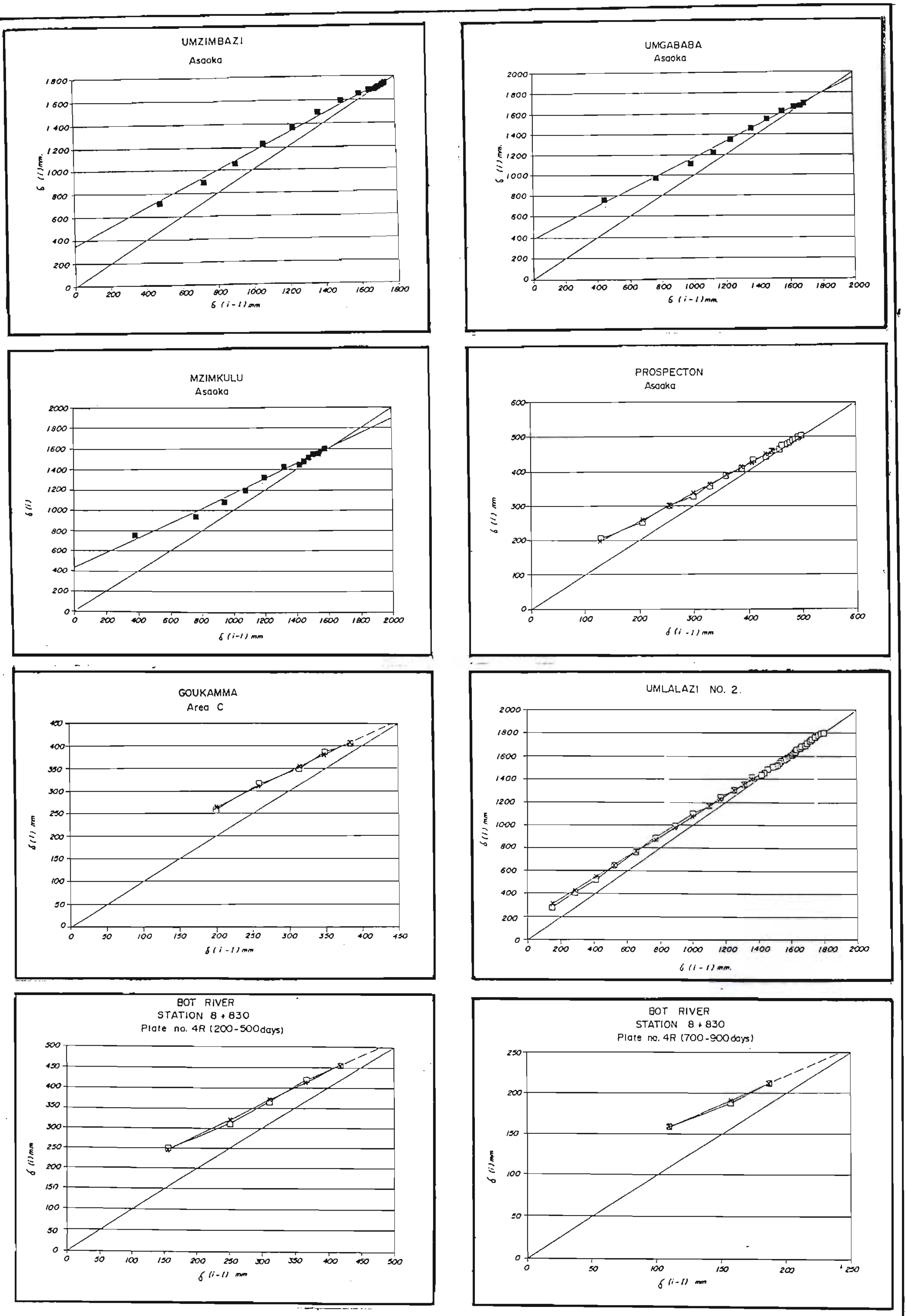
The author's method of estimating the coefficient of consolidation, c_v, was based on the empirical equation for the South African alluvial deposits viz :

$$c_v = \frac{50}{t_{50}} \qquad \text{D5.2}$$

where c_v is m^2/yr and t_{50} is in minutes

Since the alluvial deposits have coefficients of consolidation in the range of say 1 to 100 m^2/yr the half dissipation times vary from about an hour to one minute.

The c_v values obtained from the CPTU's are then compared to those derived from back analyses of the embankment performance. The latter are calculated from the Asaoka plots of settlement. This is an extremely useful technique and has the major advantage that c_v's can be estimated before the end of consolidation. An explanation of the application of Asaoka's method is given in Appendix III and examples of the use of this method are shown in Figure D5.5. In these the end of primary and beginning of the subsequent secondary consolidations can be readily distinguished. The number of points and the fit of a straight line through them give a good indication of the reliability of the definition of c_v. This method has the further advantage that the difficulty of analysing settlement records during construction by artificial curve fitting is largely eliminated.



TYPICAL ASAOKA PLOTS

Fig D.5.5

The Asaoka c_v for Umgababa, given in Table D5.3 viz 8,8 m^2/yr, is significantly different from that given in the analysis in section D4.2.6 p188, viz 1,5 m^2/yr; in addition the estimated final settlement is also different viz 3,86 m in the 1989 - 1990 research project and 2,6 m in the 1991 - 1992 project and explanations of these anomalies are given below.

In the earlier project, as shown in Figure D4.3, the time settlement line modelled from the conjunctive use of c_v, m_v and the rate of settlement gave a poor fit with the measured data points for the initial 2000 days. The measured excess pore pressures in the subsoil and the subsequent calculation from these however led to the assessment that much settlement was yet to occur; this determined a final settlement hence the value of c_v was chosen to be compatible with this.

In the 1991 - 1992 re-analysis of the Umgababa embankment the Asaoka method was used extensively. This shows very clearly - Figure D5.5 - that the end of primary consolidation is in the range of about 1,7 m to 2 m depending which data points are selected. The line shown is the linear regression through all the points, whereas it is believed that emphasis should be given to the later points, which then estimates the primary consolidation as 2,0 m. Both the c_v and m_v for Umgababa are significantly modified by this re-analysis.

As a result of this re-analysis the data point shown as 2,6 m settlement in Figure D4.3 is anomalous; it does not fit the primary consolidation curve and it appears to be too high to be secondary compression. The data is being rechecked and the embankment continues to be subject to ongoing monitoring to check that the rate of settlement continues to decrease, which is the current situation.

In common with any other method of back analysis of settlement, unless records are available at different levels in the subsoil, the coefficient of consolidation is a global one for the amalgam of consolidation characteristics that may exist under the measuring point. Whilst this estimated c_v is of great value caution must be exercised in direct correlations with c_v derived from laboratory tests and cone dissipation tests since both these refer only to small samples.

It is also noted that calculation of c_v from Asaoka plots is directly dependent on the square of the assumed drainage path length. The estimation of the effective drainage path length in variable alluvial deposits is considerably assisted by piezometer cone penetration test records. These very clearly display the different layers and in particular show boundaries defined by positions at which the pore pressures are hydrostatic during penetration. Boundaries can also be designated, if sufficient dissipation tests are carried out, at those positions at which the t_{50} times differ by an order of magnitude. The drainage path length estimated in this way represents only the situation at that particular CPTU position. It is necessary to either assume that it is representative of the overall situation at the measuring position, or to carry out more CPTU's. This is of course no different from the reverse position of estimating settlement times from laboratory c_v data where critical assumptions have to be made regarding drainage path lengths. The potential of CPTU's to improve this aspect alone in alluvial deposits, when compared to conventional boreholes and sampling, is believed to more than justify the technique.

A further set of c_v's is available in addition to the CPTU and Asaoka derived values. This is from the laboratory testing carried out during the original road design investigations. The data now available is less comprehensive than would have been desired and is often in the form of a few c_v values in reports without the original laboratory records. Nevertheless, these c_v provide a useful set of representative values. The results of the comparisons of the three data sets are given in Table D5.3. They are also shown in Figure D5.6 (b). Various averages of c_v are given in the Table but these do no more than convey a general idea of the values of c_v which occur in the South African alluvial deposits. For example for the soft clays the laboratory c_v is about 2 m^2/yr, but may vary from 0,5 to 5 m^2/yr. Similarly the CPTU derived c_v averages about 4 m^2/yr and varies from 1 to 10^2/yr. The c_v, however, from back analysis of the measured settlements averages about 8 m^2/yr and varies from about 4 to 12 m^2/yr.

Four of the back analysed coefficients of consolidation are for embankments where vertical drains have been installed before embankment construction. These coefficients of consolidation are therefore predominantly c_h not c_v. It could be argued that they should be transformed into equivalent c_v values, but to do this it would be necessary to make an assumption regarding c_h/c_v. If no data exists it would generally be accepted that in these alluvial deposits an assumption of $c_h/c_v = 2$ is reasonable and therefore the appropriate values in the Table should be halved. The overall average c_v is then changed from 7,4 to (6,7). The Umlalazi 2 laboratory coefficients of consolidation, viz 2,2/4,4 are from vertical and horizontal drainage tests and support the $c_h/c_v = 2$ assumption, as do those at Umgababa and Umzimbazi, although those at Umhlangane do not.

Table D5.3 : Coefficients of consolidation from laboratory, CPTU and settlement analyses

Embankment	c_v (or c_h) m^2/year			Remarks
	Laboratory	CPTU	Asaoka	
Manzamyama	1,2	1,0	5,4	
Umlalazi 1	2,2/4,4	9,6	6,8	drains
Umlalazi 2	4,8	1,7	4,3	drains
Umhlatuze	0,5 - 2,5	2,8	8,2	
Prospecton	0,4	1,0	4,7	drains
Mzimkulu	1,8	7,4	12	
Goukamma	1,4	6,9	4,5	drains
Hartenbos	No data			
Bot River	1,5 1,3 0,8	5,8 1,6 4,6	11 12 10	
Umgababa	1,6/2,5	1,5	8,8	
Umzimbazi	2,1/7	2,5	4,5	
Umhlangane	0,7/0,5	1,8	3,6	
Averages	2,0	3,8	7,4 (6,7)	overall
	2,2	4,8	5,1	drains
	1,9	3,4	8,4	no drains
	Ratio	CPTU/Lab = 2,5	Asa/CPTU = 3,1 (2,7)	

Of more value than the simple averages of the data is the ratios of the different c_v's at each site viz CPTU/Lab and Asaoka/CPTU. These are approximately 2,5 and 3,1 (2,7 if c_h transformed to c_v).

Lines showing approximate relationships between these c_v are shown on Figure D5.6 (b) as :

$$\text{Measured } c_v = 3 \text{ CPTU } c_v = 6 \text{ Lab } c_v \qquad \text{D5.3}$$

The Figure shows a considerable scatter of the data, but it is believed that the relationships shown are extremely useful as guidelines for what may be expected in alluvial deposits.

The laboratory c_v's are within the range usually expected for the alluvial clays (Jones and Davies, 1985).

The CPTU c_v's are higher than the laboratory c_v's as is expected, because the author's equation D5.2 was intended to give higher c_v's, since these were believed to be more representative of full scale experience. It is also generally accepted that piezometer cones having the filter element at the base of the cone, or above it in the shaft, measure largely horizontal dissipations; hence the coefficients of consolidation should be interpreted as c_h and therefore CPTU values double those from laboratory measurements could be expected.

The Asaoka analysed c_v are significantly higher than the CPTU c_v. This is supported by experience and generally justified on the basis that the laboratory samples are small and often partially disturbed and that the samples selected for testing usually have the poorest properties in order to be conservative. It is also often stated that the field drainage path lengths in layered alluvial deposits are less than assumed in design and that depending on the geometry, ie embankment width and subsoil depth, three dimensional drainage may play a significant role in determining the rate of consolidation.

The results represented by equation D5.3 suggest that the author's equation D5.2 should be modified to :

$$c_v = \frac{150}{t_{50}} \qquad \text{D5.4}$$

However, in view of the scatter of the data and because of the belief that a reasonable factor of safety should be maintained, it is recommended that the original equation D5.2 should continue to be used, with the understanding that a factor of safety is implicit.

A further deduction from the results is that the field coefficients of consolidation are about 6 times larger than the laboratory measured values. As with the CPTU c_v, however, it is not recommended that the design c_v should be 6 times larger than the laboratory measurements, but that a factor of safety should be included. If this is taken as 3 then it follows that the design c_v's may be twice the laboratory measured values, viz :

$$\text{Design } c_v = 2 \text{ lab } c_v = \text{CPTU } c_v \qquad \text{D5.5}$$

D5.5.3 Soil parameters

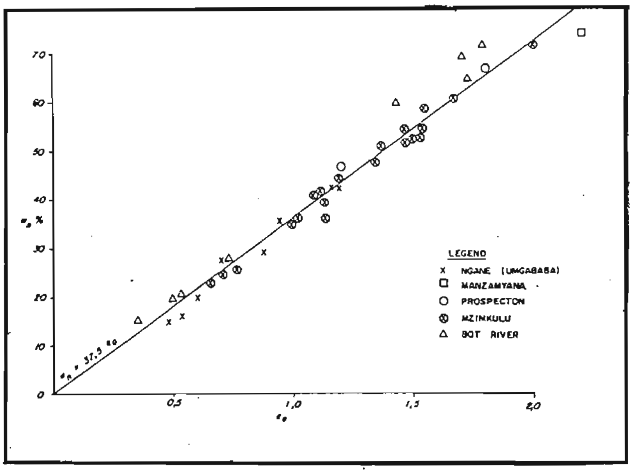
In order to compare the results from the different sites examined and to extrapolate from these to other sites, it is necessary to characterize the subsoils using the conventional basic soil parameters. Much of the data that could be extracted from the design investigation reports is reproduced in graphical form in Figure D5.7. It is considered that the reliability of the data does not justify detailed statistical analysis, and envelopes or simple linear relationships have been shown on these Figures solely to indicate trends.

The individual data sets in Figure D5.7 are discussed below.

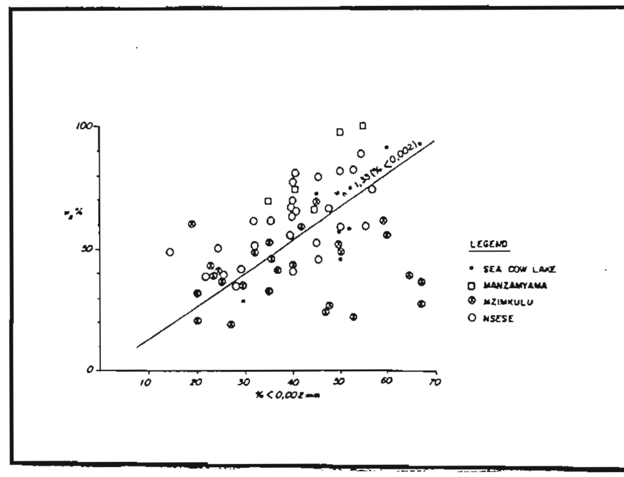
Figure D5.7 (a) Moisture content versus void ratio.

The best fit line through the data is given by :

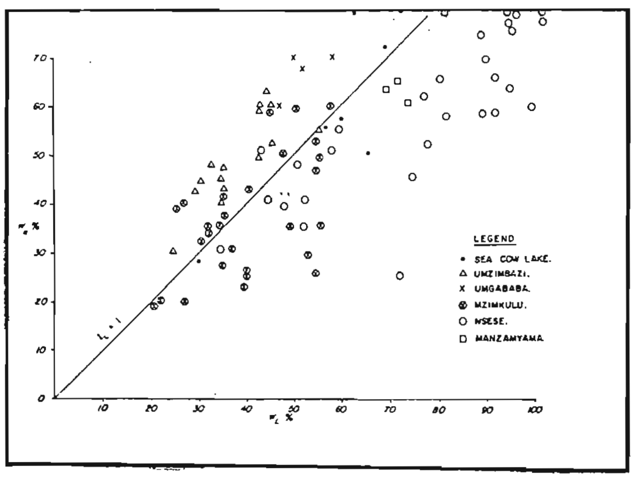
$$w_n = 37{,}5 \ e_o \qquad \text{D5.6}$$



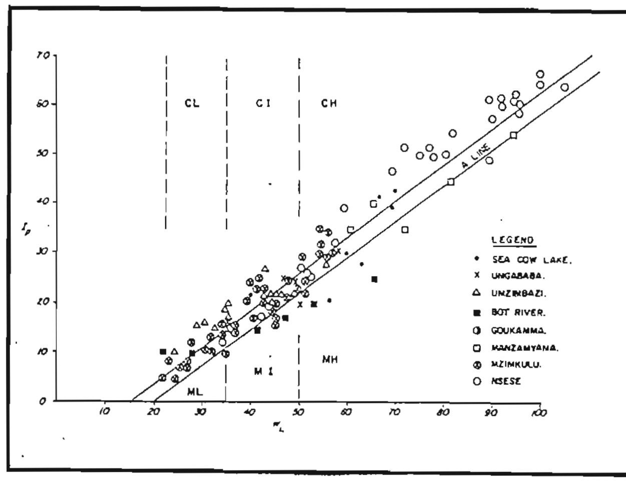
a) Water content, w_n, v Void ratio, e_o.



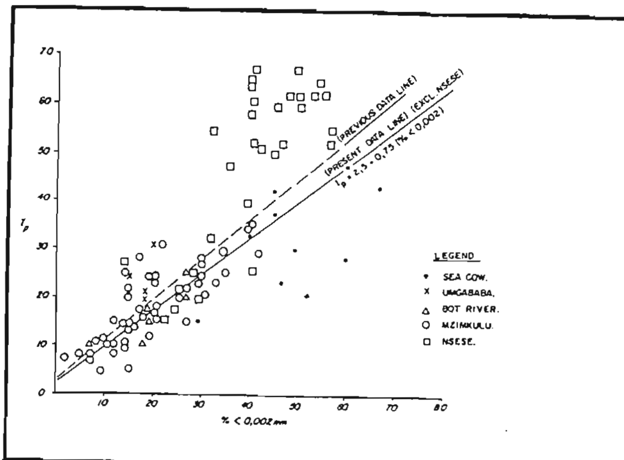
b) Water content, w_n, v %<0,002 mm.



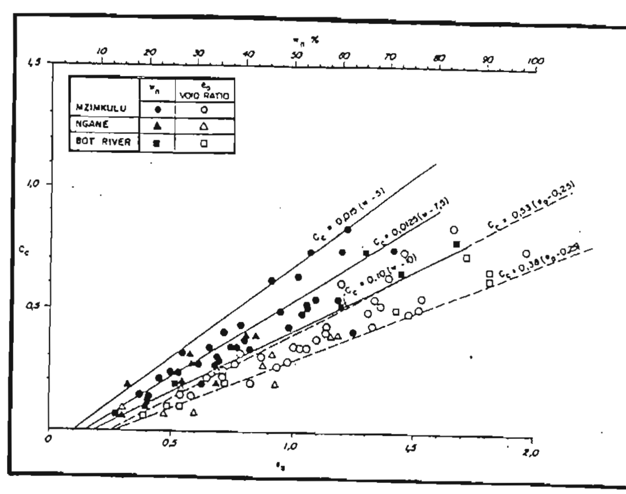
c) Water content, w_n, v liquid limit, w_L.



d) Plasticity index, I_p, v liquid limit, w_L.



e) Plasticity Index, I_p, v % < 0,002 mm.



f) Compression index C_c, v water content, w_n, and void ratio, e_o.

Fig. D·5·7

This of course should be expected if the subsoil is saturated and the specific gravity is 2,67. Nevertheless the results and the relationship are useful because they show that the alluvial deposits are saturated and that if the moisture content is known then a close estimate of the void ratio can be made.

Figure D5.7 (b) Moisture content versus percentage clay.

The data confirms the trend which would be expected, that the moisture content is to a large extent a direct function of the clay content of the alluvial deposits. The scatter is fairly large but the relationship can be expressed as :

$$w_h = \frac{4}{3} (\% < 0{,}002 \text{ mm}) \qquad \text{D5.7}$$

with most of the data falling between
$w = (0{,}002)$ and $w = \frac{5}{3}(0{,}002)$

The potential usefulness of equation D5.7 is that if either no moisture content samples are available or moisture content results are suspect, then an estimate can be obtained from clay content.

Figure D5.7 (c) Moisture content versus liquid limit.

This data would be expected to have a significant scatter since it is simply a way of expressing the liquidity index; only one liquidity index line is shown viz $I_L = 1$.
The data shows that generally the alluvial deposits have liquidity indices close to unity but there is a fairly even distribution each side of this line. Those soils above the line would be expected to be underconsolidated, whereas those below the line would be expected to be lightly overconsolidated. The liquidity index for normally consolidated clays decreases with depth, however, so that the preceding indication of

degree of consolidation from the data is no more than a general observation.

Figure D5.7 (d) Plasticity index versus liquid limit.

The data is plotted in the standard way showing the Casagrande "A" line. It can be seen that the soils plot predominantly above the "A" line and closely fit the line shown, which can be represented by the equation :

$$I_p = 0{,}73 \, (w_L - 15) \qquad \text{D5.8}$$

The soils vary from silts to clays of low plasticity to clays of high to very high plasticity, but practically all the soils at the embankments investigated fall into the low to high plasticity range ie liquid limits of between 20 and 60 and plasticity indices between 5 to 35.

Figure D5.7 (e) Plasticity index versus clay content.

This data follows a very clear trend, other than that shown for Nsese. This embankment was not included in the analyses since it was fairly recently constructed and is presently undergoing primary consolidation. The material has a high organic content and is not considered typical of the more common alluvial deposits. A linear regression line through the data points (excluding Nsese) is given by :

$$I_p = 2{,}5 + 7{,}5 \, (\% < 0{,}002) \qquad \text{D5.9}$$

This is close to the previous similar equation given by the author (1975) and which was used to assess soil type from cone penetration test friction ratios.

Figure D5.7 (f) Compression index versus moisture content and void ratio.

Both sets of data are represented by lines showing the linear regression equations and envelopes which enclose 90% of the data points.
The regression lines are :

$$C_c = 0{,}0125\,(w_n - 7{,}5) \qquad \text{D5.10}$$

and $$C_c = 0{,}45\,(e_o - 0{,}25) \qquad \text{D5.11}$$

These are very similar to published data on soft clays and therefore can be confidently used for the South African alluvial clays. It can be seen from subsection (a) ($w_n = 37{,}5\ e_o$) that the above two equations are in very close agreement although the confidence limits for the second appear to be somewhat closer. Equations such as these are extremely useful both for checking that C_c values obtained from consolidation tests are satisfactory and for estimating C_c values solely from moisture content data.

D5.5.4 Constrained modulus coefficient, α_m, compared with cone pressures, stress ratios and soil parameters

As well as describing the soils the definition of the basic soil parameters in the previous section was also intended to provide information so that if necessary and possible any range of values of α_m could be rationalised - for example, the dependency of α_m on moisture content.
Figure D5.6 (c), (d) and (e) show such comparisons.

It is apparent, as discussed in D5.5.1, that the α_m values determined from the back analyses and directly from the laboratory m_v's, cover only a relatively small range. Table D5.1 showed that the mean value was 2,63 with a standard deviation of 0,59. The different data set in Table D5.2, which only included one or two α_m determinations for each site, gave a mean value of 2,92 with a standard deviation of 0,52 ie somewhat higher because the multiple and relatively low values from Bot River were represented by only two values. It is arguable which of these is the more correct selection. Possibly the preferable view would be to consider Bot River as having only the four different positions, viz those at full height, but excluding the partial height positions. In this case the final α_m would be :

$$\alpha_m = 2,75 \pm 0,55 \qquad \text{D5.12}$$

This range may be considered as due to a general reflection on the accuracy of all parts of the total system, eg from settlement measurement to cone pressure measurement.

Alternatively the range may be due to the differences in subsoils between sites.

More probably however it is a combination of these which gives rise to a range in values because the international literature certainly suggests that α_m is to some extent both material dependent and cone pressure dependent.

In order to examine this comparisons were made of α_m with cone pressures, with representative material parameters and with stress ratios. These are shown in Figure D5.6 (c), (d) and (e). The first indicates no correlation, although there is insufficient spread of data of both q_c and α_m for any conclusion to be expected.

The second, Figure D5.6 (d) shows moisture content against α_m. As already shown the moisture content is closely related to the void ratio, liquid limit, plasticity and to the compression index, so that all these would be expected to show the same trend. Since moisture content in this case forms the primary data set, it is this that is plotted rather than any derivatives.

The Figure shows an indication of a positive correlation which is shown by a line, the equation of which is :

$$\alpha_m = 0,0375\ w_n + 0,5 \qquad \text{D5.13}$$

or since $w_n = 37,5\ e_o$ - Figure D5.7 (a) then :

$$\alpha_m = 1,41\ e_o + 0,6 \qquad \text{D5.14}$$

This is a surprising finding and is contrary to what might have been expected, which is that since the moisture content or void ratio increases so the compressibility increases, then α_m would decrease. But α_m is a function of both the compressibility and shear strength so it is the relative change in these which determines the change in α_m.

However, because of the fairly poor correlation shown and because there is considerable doubt as to whether the moisture contents recorded are representative for all the subsoils at any settlement measurement position, then the correlations given by equations D5.13 and D5.14 cannot be considered as valid. Much more field work would be necessary to validate or invalidate this apparent correlation.

This however is not a significant problem since in anycase the range of α_m is not large viz 2,75 ± 20%. Settlement estimates to this level of accuracy would for most practical purposes be considered to be readily acceptable. If the estimated settlements are high and the consolidation time unfavourable it would be necessary to carry out further investigations from which settlements could be independently estimated from laboratory tests. The two different sets of estimates should then be compared and judgement exercised in their interpretation.

The further correlation attempted is of stress ratio against α_m and is shown in Figure D5.6 (e).

Stress ratio is defined as the embankment unit weight times height, γH, divided by the subsoil shear strength which is represented by the cone pressure, q_c.

It might be expected that at high stress ratios the moduli would be lower, hence α_m too would be lower. The data does not support this and no correlation exists. In the Figure the three highest stress ratios are for Mlatuze where the subsoil is predominantly peat and this data should probably be excluded. The remaining points show that the range of stress ratios is not high which is as expected since the range of embankment heights is not large nor is the range of cone pressures.

D5.6 **Summary and Conclusions of 1991 - 1992 Research Project**

The cone penetration test derived data is summarized in Tables D5.1, D5.2 and D5.3 and shown in Figures D5.6 (a) and D5.6 (b).

The soil parameter data is shown in Figures D5.7 (a) to (f).

The conclusions are summarized as follows :

(i) The constrained modulus coefficients, α_m, determined from back analysis of the embankment settlements, after making allowance for local yield and secondary compression, are given by :

$$\alpha_m \quad = \quad 2{,}75 \pm 0{,}55 \qquad \text{D5.12}$$

(ii) The constrained modulus coefficients, α_m, determined directly from

$$\alpha_m \quad = \quad \frac{1}{m_v\, q_c}$$

in which the m_v values are obtained from laboratory tests are equal to those given by the equation above - see Figure D5.6 (a).

(iii) The constrained modulus coefficients show no correlation with cone pressures and stress ratios.

(iv) The constrained modulus coefficients show some correlation with moisture content - Figure D5.6 (d). This is a weak correlation and because it is based on what may be unrepresentative moisture content data it is recommended that at this stage no correlation of α_m with moisture content should be assumed for the alluvial deposits of South Africa.

(v) Coefficients of consolidation estimated from piezometer cone penetration tests show a good correlation both with laboratory values and embankment back analyses values. The correlations are given in Table D5.3 and shown in Figure D5.6 (b) and may be summarized by :

$$\text{Measured } c_v \quad = \quad 3 \text{ CPTU } c_v = 6 \text{ Lab } c_v \qquad \text{D5.3}$$

(vi) The relationship given below to determine c_v from CPTU dissipation tests is supported by the evidence, provided that it is understood to include a factor of safety, ie this c_v is suitable for design purposes :

$$\text{Design } c_v \quad = \quad \frac{50}{t_{50}} \qquad \text{D5.2}$$

where c_v is m^2/yr and t_{50} in minutes.

(vii) Correlations of a range of materials parameters viz moisture content, void ratio, clay content, plasticity and compression index give very useful relationships not only for characterisation of the alluvial deposits but also for the estimation of dependent material parameters from basic information. In particular the most useful of these are :

$$C_c = 0{,}0125\ (w_n - 7{,}5) \qquad \text{D5.10}$$

$$\text{and} \quad C_c = 0{,}45\ (e_o - 0{,}25) \qquad \text{D5.11}$$

The overall conclusion from the 1991 - 1992 research project is that it has confirmed that piezometer cone testing is an extremely useful technique for the investigation of alluvial deposits. Not only can it provide information, practically unobtainable by any other means, but it does so rapidly and economically. Because of this the technique is unsurpassed for preliminary investigations of alluvial deposits. Where potential problems are revealed it indicates the most effective positioning of boreholes and of sampling and testing, as well as providing compressibility and consolidation characteristics and strata delineation. Where these potential problems are indicated, however, it is considered essential that the piezometer cone information should be corroborated by conventional boreholes with sampling and laboratory testing.

E SUMMARY OF CONCLUSIONS AND RECOMMENDATIONS

E1 INTRODUCTION

This thesis comprises five main parts viz :

A Introduction, Problem Definition and Geology

B Historical Review of Cone Penetration Testing

C South African Developments in Cone Penetration Testing

D South African Application of Cone Penetration Testing

and this part E Summary of Conclusions and Recommendations

In drawing overall conclusions, and hence recommendations from the work described, it is necessary to appreciate that because of the time scale for the development of cone penetration testing, many of the conclusions have already been accepted and now form part of common practice. It is also necessary to consider the purpose of the work expressed by the title "The development of sounding equipment for the assessment of the time-settlement characteristics of recent alluvial deposits when subjected to embankment loads". The implication of this is that a process was intended of first developing cone penetration testing so that it was adequate for the purpose and then showing that the data acquired could be used for embankment performance prediction.

It is therefore appropriate to summarize the conclusions of the process in chronological order rather than in order of significance, thus leading to the essential conclusion that sounding, or cone penetration testing, is particularly suitable for the prediction of embankment performance.

E2 SUMMARY OF CONCLUSIONS

E2.1 General Conclusions

The prediction of the performance of embankments on soft variable recent alluvial deposits has long been recognized to be problematic. The nature of many of these materials is such that undisturbed sampling is difficult and their multilayering requires a large number of samples if adequate representation of the variability is to be achieved. These problems lead to the desirability of having an alternative method of investigation. Cone penetration testing has, since the 1930's, been used for this purpose, eg Standard Penetration Tests (SPT), but primarily for cohesionless materials. The use of Dutch probing or cone penetration testing (CPT) began in South Africa in the 1950's and by the mid 1960's was in fairly common use for assessing pile founding depths. At this time it also began to be used for the estimation of settlement of embankments and large oil tanks; as a means for assessing the necessity for vibroflotation under footings or floors and for checking the effectiveness of vibroflotation. This experience in the Durban area led to equations for assessing the moduli of sands and clayey sands for which the author was jointly responsible - equations C2.2, C2.4 and C2.5 pp105,106.

From 1969 to 1973 the author was directly involved in site investigation for the major freeways along the north and south coasts of Natal and for the Durban Outer Ring Road. Many rivers and flood plains with extensive soft alluvial deposits had to be crossed and the investigation of these was a major task. In a number of cases the route location was yet to be determined and the geotechnical considerations played a significant part in this. It was clear that cone penetration testing could be a valuable adjunct to the conventional boreholes with sampling and Standard Penetration Testing. Since the cone penetration equipment then in current use was crude, the author introduced improved load measuring systems and standard Dutch 1 000 mm^2 cones and made their use mandatory for the major road investigations by introducing appropriate specifications for investigation contracts.

Toward the end of this period the international literature - notably Sanglerat (1972) - reflected a much broader usage of cone penetration testing and various semi empirical methods were available for the assessment of the compressibility of clays as well as for sands.

Nevertheless the equipment itself was not sufficiently sensitive for use in very soft materials; in addition, as with all semi empirical correlations, there was considerable doubt regarding the use of these correlations in materials other than those from which they were derived.

Although the potential of cone penetration testing for the estimation of settlements was becoming recognized, the prediction of consolidation times could only be attempted from laboratory testing. Furthermore both local experience and the international literature appeared to accept that the reliability of predictions of consolidation times for variable alluvial deposits was generally very much poorer than the prediction of settlements. The challenge was to develop a method of predicting consolidation times, with acceptable accuracy, using cone penetration testing.

The author began a period of research, 1973 - 1977, at the National Institute for Road Research, primarily aimed at the development of cone penetration testing. In 1975 the work seemed to have progressed sufficiently to justify registering for a MSc degree at the University of Natal with the same title as this thesis. The findings of this work were twofold. The first concerned the improvement to the equipment and the second concerned the ideas and tests for measuring consolidation times.
The improvement to the equipment was intended to allow cone pressures to be adequately measured even in the softest clays. A method of achieving this was designed and built using a range of interchangeable strain gauge load cells in conjunction with standard cones and rods. The load cells were calibrated in the laboratory and the output was fed to a chart recorder so that continuous records of cone pressures were made, which eliminated the problem of reading and recording changing gauge pressures.

The resulting equipment was able to measure undrained shear strengths of soft clays to an accuracy of ± 2,5 kPa when operating in the range of 5 kPa to 100 kPa.
This represented an improvement of an order of magnitude compared with the previously available equipment and meant that for the first time in South Africa meaningful cone penetration test results could be achieved in soft clays.

The author then introduced the use of the Begemann friction sleeve to South Africa. The loads required for operating the sleeve were measured and recorded through the strain gauge/chart recorder system. It was found that the friction ratio interpretation data in the international literature was not applicable for the low pressure ranges found in the South African soft clays and silts. The author, through the correlation of friction ratios with material descriptions obtained from immediately adjacent borehole samples, drew up a friction ratio - material description chart. This work is summarized in the first paper given in Appendix I - Jones, G A (1975) and in Sections C3.3 and C3.4.

The primary aim of the research project was to develop a method for the prediction of the consolidation times from cone penetration testing. Although a reliable indication of the material type became possible using the improved load sensing equipment and friction sleeve, the correlation of material type with consolidation characteristics (Table C3.2 p122) could not be considered as anything but a crude estimate, the main use of which was to determine what subsequent investigation was necessary.

The author therefore conceived and developed a consolidometer-cone for the estimation of consolidation characteristics, which is described in detail in Section C4 (pp124-149). It was shown by laboratory testing that satisfactory results were obtained relating settlement times measured by the consolidometer-cone to consolidation characteristics from conventional consolidation tests. The results of these tests are summarized in the second paper given in Appendix I - Jones, G A (1977) and illustrated in Figures C4.13 and C4.14 p137 and Table C4.2 p140.

Field tests of the consolidometer-cone system (Figure C5.4, p155) showed two significant problems.

The first was that it was difficult to judge the load required to produce only apparent consolidation type behaviour and not failure, although some skill eventually developed to permit this.

The second was that to complete a test to the end of settlement could take an hour or two. It was envisaged that with sufficient experience an approximate theoretical model could be developed so that matching field results with model curves would enable tests to be discontinued after the time for half settlement, which would take possibly 20% or so of the final settlement time. Without this, however, the field testing would be very time consuming, and hence a potential benefit of a relatively inexpensive testing system would be lost.

In order to develop this model it was considered necessary to understand the process during cone settlement. The idea was conceived by the author of measuring changes of pore pressure around the cone.

Thus the first piezometer cone was developed.

The original laboratory 1975 model, (Figure C5.1) was a combined conventional mechanical cone and a piezometer in the cone having filter elements in the face of the cone. The system operated remarkably well and very clearly showed when used in the consolidometer-cone mode that settlements and dissipations of pore pressures were strongly correlated.
Additionally, and of much greater significance, was the realisation that the potential of a field version was extremely exciting.

The laboratory version had shown that cone penetration generated excess pore pressures which dissipated on ceasing penetration. Rates of dissipation could be measured and it was shown that these could be correlated with coefficients of consolidation derived from consolidometer tests. Hence the concept of a consolidometer-cone was soon overtaken by the piezometer cone. It was also realized that by measuring pore pressures during penetration much more information would be available on the shear strength characteristics.

The shear strength aspect had been discussed theoretically in the international literature (ESOPT I, 1974) but no cones were available for the simultaneous measurement of cone and pore pressures. In 1975 Torstensson and Wissa et al independently reported on cones having only pore pressure systems which were used to assess consolidation characteristics. In the following five years these cones were used by a number of researchers, notably Baligh and colleagues, in conjunction with conventional cone tests at adjacent positions so that comparative cone and pore pressure readings could be obtained.

In this period the author developed the field piezometer cone from the laboratory version and in 1976 the first field model was operational.

In early 1977 the research environment was exchanged for consulting engineering. Fortunately the consulting practice was heavily involved in national road design along the Natal coast and also operated a geotechnical site investigation company. They were very supportive of the development of cone penetration testing and the field piezometer cone continued to be improved.

Numerous projects were undertaken in the 1977 - 1981 period in which piezometer cone results were obtained for alluvial deposits and mine tailings dams. By the latter half of that period it was concluded that the piezometer cone was no longer experimental, but produced very useful information of a number of subsoil aspects. A series of papers was published by the author and colleagues in 1981, 1982 and 1983 (see Appendix I) which describe the development of the piezometer cone system and the interpretation of the results.

At the conferences at which these papers were published a number of other authors also described piezometer cones and it was obvious that similar ideas had been developing elsewhere. These culminated in papers by the author and others at the 10th International Conference, Stockholm, 1981; ASCE Symposium on Cone Penetration Testing and Experience, St Louis, 1981; ESOPT II, Amsterdam, 1982 and International Symposium on In Situ Testing, Paris, 1983.

In addition to the first descriptions of piezometer cone equipment and its subsequent development, the author also contributed in these papers to the interpretation of the results on two aspects in particular.

The first of these was on consolidation characteristics, which is described in B4.4, pp59-62, and the second on soils identification, B4.5, pp74-79. These are briefly summarized below.

Consolidation characteristics were assessed by correlating piezometer cone dissipation t_{50} times with coefficients of consolidation from laboratory tests. This data, together with some judgement based on experience, gave rise to the equation :

$$c_v = \frac{50}{t_{50}} \qquad \text{B4.27}$$

where c_v is in m^2/yr and t_{50} is in minutes.

The approach adopted is pragmatic and contrasts with the theoretical modelling approaches of some authors. There is continuing discussion regarding the appropriate theory and it has been found in South Africa after ten years of experience that the pragmatic approach yields more satisfactory results than the alternative theories at this stage. However, the method was developed from the local alluvial deposits data and it should only be applied universally with caution.

The second specific aspect of cone penetration testing interpretation to which the author has contributed is that of soils identification. This is illustrated by the soils identification chart - Figure B4.20 p78, which has been referenced and reproduced in the international literature.

The chart was derived from the observation during piezometer cone testing that the pattern of cone pressures was mirrored by an opposite pattern of pore pressures in multilayered deposits, viz low cone pressures matched high pore pressures and high cone pressures matched low excess pore pressures. The former were in clays and the latter in sands and simply reflected the permeabilities. Field results were observed at many sites at which samples were taken for particle size and Atterberg limit tests. From this data the soils identification chart was drawn up primarily for normally consolidated soils. Overconsolidated clays and silts caused negative excess pore pressures to be generated and the chart also accommodates these. It should be noted that the pore pressures measured depend on the position of the measuring element and therefore the chart is applicable only for cones with this element in a similar position to the author's cone viz immediately above the shoulder, which is the most common position.

Charts have subsequently been produced by other authors on a similar basis (Senneset and Janbu, 1985, and Robertson et al, 1986). The author has also produced a further version of the chart, using the same data as the original, Figure B4.22, p79, primarily for use for low cone pressures.

The immediate benefit of this soils identification method was not only that the material type within any stratum could be described, but also that the strata boundaries could be accurately determined, which is essential for the analysis of multilayered alluvial deposits.

In the period from the initial 1981 papers on piezometer cone penetration testing - called CPTU after suggestion by the author - through to ISOPT I at Orlando in 1988, international interest developed considerably. By 1988, the emphasis was not so much on new ideas or uses, but on the spread of knowledge from the early few practitioners to the position where piezometer cone testing became almost standard practice in site investigation. This too was the situation in South Africa. The equipment was significantly improved, and data loggers and computers were introduced. The early 1980's equipment produced chart outputs of cone and pore pressure readings for every few millimetres of penetration; translating this into usable logs was a formidable task. Fortunately this need coincided with developments in the instrumentation industry and in computers. The present South African piezometer cone is now used in the field with a lap top computer being both the data logger and production unit for logs which can be viewed during penetration.

During the mid 1980's many of the embankments built years earlier were being monitored and it was observed that the settlements of a number of these exceeded the predictions. The author applied for funding to examine this problem and this application eventually resulted in the first CPTU Research Project, 1989 - 1990 and the second CPTU Research Project, 1991 - 1992, which are described in D4 and D5. The results of these projects are confirmation of the value of cone penetration testing for the prediction of the performance of embankments on alluvial deposits.

In the 1989 - 1990 project three embankments were selected for further study on the basis of the availability of monitoring data for between ten and fifteen years. The purpose of the study was to carry out piezometer cone testing at the embankments and correlate settlement and time settlement predictions with the measured performances and hence calibrate the prediction methods. Settlement predictions use the constrained modulus coefficient, α_m, to correlate m_v with cone pressure, and time predictions derive c_v from the $50/t_{50}$ relationship described by the author.
The results of the project are summarized in section D4 and in a paper in Appendix I, (Jones, G A and Rust, E, 1991). These are that a value for α_m of 1,24 was appropriate for these three embankments and that a cone time factor of 50 allowed satisfactory settlement time predictions at least comparable in accuracy with those from laboratory consolidation test data.

Despite the consistent results the large difference between the derived α_m, value of 1,24 and those quoted in the literature, (Table B4.1, p48), raised doubts about the application of these findings on a general basis. Two of the embankments had settled much more than had originally been predicted, but both had undergone partial failures during construction. It was considered likely that these were not typical embankment situations, and hence the very low α_m values should not be generally applied. It was nevertheless considered to be prudent to publish the data, noting that it may only be applicable to cases of highly stressed subsoils. Because of these limitations it was clear that more comprehensive information was required on embankment performance.

This gave rise to the second CPTU Research Project, 1991 - 1992. This project was similar in concept to the first, but included a further eight embankment sites along the Cape and Natal coasts. Since at some of these, settlement records were available at more than one position a total of twenty five cases were available.

The approach to the method of analysis of the data included two significant differences from the 1989 - 1990 project. The definitions of α_m and of embankment height were revised. In the 1989 - 1990 project the view was taken that the engineering practitioner would require an estimate of total settlement from cone penetration testing and this would include any components due to yield and secondary compression; on a similar basis it was assumed that the height used for settlement predictions would be the design height of the embankment. This approach was changed for the 1991 - 1992 project. The cone penetration test method has been generally accepted to give a measure of the coefficient of compressibility, m_v, hence only settlements due to the immediate and consolidation components should be included in the analysis. Where embankment settlement are small, the definition of the effective embankment height as the design height causes only a small error; where settlements are large, however, the error is significant. Therefore for back analyses of settlements for this research project the effective height is defined as the constructed height, together with any settlement which has occurred and which may occur.

These two modifications significantly increase the values of α_m.

E2.2 Quantitative Conclusions

The 1991 - 1992 project has allowed a number of specific conclusions to be drawn. These are described in section D5 and summarized there in Tables D5.1, D5.2 and D5.3 and in Figures D5.6 and D5.7. The more important of these are listed below :

(i) The analyses shows that the value of the constrained modulus coefficient, α_m, derived from back analysis of embankment settlement is given by :

$$\alpha_m = 2{,}75 \pm 0{,}55 \qquad \text{D5.12}$$

It is concluded that this is representative of the alluvial deposits in South Africa.

(ii) Values of α_m derived from direct comparison with laboratory coefficients of compressibility are practically identical with those derived from back analysis of settlements. It is concluded that cone penetration testing allows good estimates of laboratory coefficients of compressibility to be made.

(iii) Attempted correlations of α_m with moisture content, cone pressure and stress ratio showed only a weak correlation with the first and no correlations with the other parameters. The correlation with moisture content (Figure D5.6 d) was unexpected and weak.
It is concluded that insufficient evidence was available from this project to support these correlations.

(iv) Back analysis of the time settlement characteristics showed that the use of the author's equation :

$$c_v = 50/t_{50} \qquad \text{D5.2}$$

gave a good correlation with both laboratory and back analysed c_v which can be represented by :

$$\text{Measured } c_v = 3 \text{ CPTU } c_v = 6 \text{ Lab } c_v \qquad \text{D5.3}$$

It is concluded that the previous equation D5.2 is supported by the new evidence, noting that a design c_v is derived, which includes a factor of safety. It is also concluded that equation D5.3 is a realistic representation of the relationship between the coefficients of consolidation derived from the three approaches of back analysis, CPTU and laboratory tests.

(v) The settlement analyses show that coefficients of consolidation from laboratory tests give a poor direct prediction of settlement times. The data set supporting this is not large and therefore no firm quantitative conclusion is justified, although cognizance should be taken of these findings for future design.
It is concluded that where prediction of the correct time for settlement is essential, then piezometer cone tests should be carried out to supplement laboratory data. If the relationship between these corresponds to that indicated by equation D5.3 then extrapolating them on the basis of the equation is justified, provided the CPTU's show no unusual subsoil stratification.

(vi) Considerable laboratory test data was available from the design investigation reports examined during the 1991 - 1992 project. This has been collated and shown in various forms in Figure D5.7 (a) to (f). Only data on the cohesive deposits is included. These show that the South African coastal alluvial deposits cover a fairly wide range from silts to clays of low, medium and high plasticity. No unusual characteristics are shown by the results. Useful working relationships between different parameters have been derived and are shown in Figure D5.7. The most useful of these are given by :

$$C_c = 0{,}0125\ (w_n - 7{,}5) \qquad \text{D5.10}$$

$$C_c = 0{,}45\ (e_o - 0{,}25) \qquad \text{D5.11}$$

These equations show close agreement with similar equations quoted in the international literature. It is concluded that the above equations represent the correlation of compression indices with moisture content and void ratio for South African alluvial deposits.

E2.3 Concluding Remarks

The author's work on cone penetration testing of South African alluvial soils over a period of twenty five years began with improvements to the equipment itself. These enabled meaningful results to be obtained even in very soft clays, hence the scope for application of cone penetration testing was greatly increased.
The use of the friction sleeve cone was introduced and a soils identification chart based on friction ratios was developed. Correlations of cone pressure with field vane tests and with laboratory derived undrained shear strengths confirmed that the cone factor, N_c, used elsewhere was applicable to South African soft clays.

Since the problem of adequate prediction of embankment settlement times is as important as that of settlement magnitude, a method of achieving this using cone penetration tests was sought. The laboratory consolidometer-cone concept was originated and this was shown to give encouraging results which led to a field version being tested. In order to test whether the settlements observed during laboratory consolidometer-cone tests were due to consolidation the piezometer cone was conceived. This proved to be so successful that it supplanted the consolidometer-cone. The piezometer cone developed by the author has been used, first experimentally and then on a routine basis, in South Africa for fifteen years, both on alluvial deposits and in tailings dams.

To a large extent correlation between piezometer cone test data and soil parameters had been based on those published in the international literature. It was necessary to confirm these by local experience.
In 1981 a soils identification chart was published which has been generally accepted.
The verification of settlement and settlement time predictions has necessitated waiting until a sufficient data base of embankment performance was available.

All of this has now been achieved, and conclusively shows that piezometer cone penetration testing is an extremely valuable technique for the geotechnical investigation of alluvial deposits.

E3 RECOMMENDATIONS

Two sets of recommendations emanate from the research and development on cone penetration testing by the author. The first concerns the practical implementation of the system and the second concerns those areas which require further research.

E3.1 Implementation

To a large extent piezometer cone penetration testing has already been accepted both in South Africa and internationally. Its application to alluvial deposits is particularly suitable and there are few geotechnical engineers in South Africa who would not readily use the method. Nevertheless, there is some concern that piezometer cone testing could be seen as a substitute for well conducted investigations incorporating boreholes, high class undisturbed sampling and appropriate laboratory testing. It is not the author's belief nor intention that this should happen and the complementary nature of the two approaches is strongly advocated.

The semi-empirical form of the interpretation of piezometer cone penetration test results means that such semi-empirical relationships that exist will continue to require amplification and corroboration of their applicability in conditions different from those where they were developed.

The research in South African alluvial soils has, however, now reached the stage indicated in the conclusions in the previous section, which is that the relationships derived for the constrained modulus coefficient and the cone time factor may be applied in practice with confidence.

It is therefore recommended that these relationships given in E2.2 (i) and (iv) should be adopted.

There is, in fact, no obstacle to such application other than informing the geotechnical practitioners of the findings and it is recommended that this should be done by appropriate publications.

These recommendations are twofold in that they deal with both the compressibility and the consolidation time characteristics of the alluvial clays.

The former are readily measured during penetration tests simply by recording cone pressures and transforming them to coefficients of compressibility, m_v, using the constrained modulus coefficient α_m. In other words, one of the major advantages of cone penetration testing, its rapidity and hence economy, is maintained.

Consolidation time assessment, on the other hand, requires dissipations tests which may take up to an hour to perform. In this time about 10 m of penetration testing could be carried out, including the time for adding rods and finally for extracting them. There is consequently an ongoing pressure to minimize dissipation testing to achieve an apparently economic result. This may lead to insufficient data being obtained on the consolidation characteristics and the potential benefit of the system is lost. It is absolutely essential to carry out sufficient dissipation tests, particularly in variable deposits.

There is only one realistic way of overcoming this and it is that geotechnical personnel who understand the implication of the results should closely supervise the field work so that the most satisfactory, hence economic, results are obtained.

E3.2 Future Research

The derivation of compressibility characteristics from what is essentially a test to failure implies that there is a definable relationship between shear strength and undrained moduli and in turn to drained moduli. This is a fundamental difficulty with the interpretation of cone penetration testing and although empirical data is assembled to define practical relationships for specific soils, it would be preferable for adequate theoretical models to be developed. Such international research efforts are currently being made and are referred to in Part B.

International literature also refers to a number of ways of interpretation of dissipation time data. At present there appears to be no consensus regarding a correct method. Much more research is required to develop a satisfactory theoretical approach for this, but until such time as this can be established, recourse to semi-empirical correlations such as that given by the author must suffice. More data is always required for such correlations both with laboratory tests and with embankment performance.

As in all research and development some problems, anomalies and unexpected, potentially interesting data occur. Three such aspects have been observed during this research and practice which should be pursued.

The first of these is not concerned with cone penetration testing per se, but with embankment performance. Figure D4.4 and D4.5, p185 illustrate the point. This is that the apparent influence of the piled Umgababa bridge on the settlement of the embankment extends about 100 m on each side of the bridge. This seems to be a remarkably long distance and is confirmed by similar results at Umzimbazi. The soil-structure interaction is therefore extensive and it would be valuable to model this analytically since superficially, at least, the bridge foundations may be subjected to larger loads than envisaged.

The second aspect is that in some soils and conditions the pore pressures increase on stopping penetration to add further rods: then, only after some metres of penetration, does the pore pressure reach a stable value relative to the cone pressure. This means that before this stability is reached the ratio of pressure to cone pressure is changing yet the material is not and no interpretation can be made of the data. The obvious immediate explanation would appear to be that the filter system has become desaturated and entrapped air has delayed the response time of the system. It is often observed, however, that when the cone passes into a lower different material no problem occurs indicating that the filter had become desaturated. It is necessary to understand the mechanism which causes this anomaly since it may have an important bearing on the interpretation of piezometer cone data.

The third point which has long intrigued the author is that it has been observed that when the cone passes from one material into another the pore pressure response does not occur simultaneously with the cone pressure response. This is of course expected because of the geometry of the system, viz the filter element is at the base of the cone. However this would lead to the expectation that the difference in response time is constant, but observation shows otherwise. The difference is small and could not be measured with earlier cone systems. With modern cones this limitation does not exist and this variable response time could be isolated from existing data and an attempt made to check whether it could be rationalized and compared with any aspect of the soil nature or state.

The author's work described here has been primarily concerned with the use of cone penetration testing to predict embankment performance. The dominant problem, after the development of the piezometer cone, has not been the equipment itself, or even the interpretation of data from it, but has been the adequacy of the embankment performance records and of the original design investigations for them. This is a recurring problem of geotechnical engineering, or of any subject which relies heavily on an observational approach and there appears to be no easy solution. Certainly comprehensive data recording at every embankment constructed simply to accumulate information is a luxury that could not be advocated. A more reasonable approach would be to preselect one or two suitable embankments a year and demarcate them for observation, not only during construction, but for some time thereafter until they are stable. The necessary condition for this approach to be effective is that long term research contracts must be arranged, which must be the dream of all researchers.

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APPENDIX I
AUTHOR'S REFERENCES AND PAPERS

APPENDIX I

AUTHOR'S REFERENCES AND PAPERS

A complete list of the author's references relevant for cone penetration testing and embankments on alluvial deposits is given on the following page.

Where these are in the form of papers, as listed below, copies are given.

Where the references are documents, viz National Institute for Transport and Road Research publications and South African Roads Board research reports, copies are not included.

(i) Deep sounding - its value as a general investigation technique with particular reference to friction ratios and their accurate determination.

(ii) Prediction of time for consolidation from sounding.

(iii) Embankments on soft alluvium - settlement and stability study in Durban.

(iv) Design and monitoring of an embankment on alluvium.

(v) The piezometric probe - a useful investigation tool.

(vi) Design and monitoring of an embankment on soft alluvium.

(vii) Mine tailings characterization by piezometer cone.

(viii) Piezometer penetration testing CUPT.

(ix) Piezometer probe (CPTU) for subsoil identification.

(x) Soft clays, problem soils in South Africa.

(xi) Piezocone testing to predict soft soil settlement.

(xii) A comparison between in-situ and seismic methods of site investigation in the alluvial beds of the Sabi River, south eastern Zimbabwe.

LIST OF AUTHOR'S REFERENCES

The author's references relevant for cone penetration testing and embankments on alluvial deposits are listed below.
Those papers marked * are included in this Appendix.
The remainder, which are longer documents have not been included for this reason, but are available, if required.

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(i) DEEP SOUNDING - ITS VALUE AS A GENERAL INVESTIGATION TECHNIQUE WITH PARTICULAR REFERENCE TO FRICTION RATIOS AND THEIR ACCURATE DETERMINATION

Sixth Regional Conference for Africa on SOIL MECHANICS & FOUNDATION ENGINEERING Durban, South Africa, September 1975

Deep sounding–its value as a general investigation technique with particular reference to friction ratios and their accurate determination

G.A. JONES, Chief Research Officer, National Institute for Road Research, CSIR, Pretoria

SYNOPSIS. Deep sounding is a well-established method for assessing the settlement of embankments over saturated alluvial deposits. Two examples are given in which predictions are made about the final settlement under embankments currently being constructed. A useful development in deep sounding is the friction sleeve which enables a description of the subsoil to be made solely from the sounding readings. However it is shown that a high degree of accuracy is required in the measuring system if meaningful friction ratios are to be deduced and that this accuracy is probably not possible when conventional equipment is used in very soft, clayey alluvium. An improved electrical resistance, strain gauge, load-sensing apparatus is desribed which is cheap, can be used with any sounding machine and has the considerable advantage of recording automatically on a chart. This equipment has made it possible to compare South African correlations of friction ratio against material type with correlations obtained elsewhere. These are shown to be in agreement. It is suggested that relating the friction ratio to the plasticity index may be more useful than the conventional method of relating it to a percentage smaller than a given particle size, and a chart is drawn up on this basis.

RÉSUMÉ. Le sondage à grande profondeur est une méthode bien établie pour l'estimation du tassement des remblais construits au-dessus d'un alluvion saturé. On cite deux exemples de la prédiction du tassement définitif sous des remblais actuellement en construction. Le manchon de frottement est une amplification utile du sondage à grande profondeur, qui permet la qualification du sous-sol uniquement à base des lectures du sondage. Une grande précision de mesure paraît pourtant nécessaire pour en déduire des taux du frottement significatifs; de plus, il est probablement impossible d'atteindre une telle précision si l'on emploie un appareillage classique dans un dépôt alluvionnaire très tendre et argilleux. On décrit ici un nouveau appareillage perfectionné pour la mesure des charges, qui est peu coûteux, muni des résistances électriques et d'un extensomètre, qui peut être employé avec n'importe quelle machine de sondage, et qui a aussi le grand avantage d'enregistrer un graphique automatiquement. A l'aide de cet appareillage, on a pu comparer les correlations entre le taux de frottement et le type du matériau rencontrées en Afrique du Sud, avec celles rencontrées ailleurs. On a mis ainsi en évidence un accord entre ces correlations. On suggère que le rapport entre le taux de frottement et l'indice de plasticité serait plus utile que la méthode classique de rapporter le taux de frottement à un pourcentage inférieur à une certaine valeur granulomètrique. A base de cette idée, on a exécuté un graphique.

INTRODUCTION

The continuing and increasing rate of development of the transportation infrastructure in South Africa has led to a need to review the technical approach to route location and investigation methods. The higher geometric standards now required and greater emphasis on social and environmental considerations, have already resulted in routes being located in areas which hitherto would have been rejected on geotechnical grounds. Typical of the resultant problems is that of having to design and construct embankments across soft alluvial deposits.

Frequently, rapid assessment of the feasibility of such constructions is required at an early stage and the alignment of a considerable part or even the whole of a proposed route may depend on this assessment.

Undisturbed sampling of recent alluvial deposits can be extremely difficult and, because of the typical inhomogeneity of the material, can be unreliable unless numerous samples are taken.

It is almost inevitable that there will be insufficient time available for the investigation, so it follows that a rapid in-situ testing system is highly desirable. Deep Sounding, also called Dutch Probe or Static Penetrometer Testing, provides such a technique.

The general use of this method, and in particular the value of the friction ratio as a means of identifying the material type, is described in this paper. The necessity for an accurate measuring system in order to determine the friction ratio reliably is demonstrated by analysing the results of investigations using conventional equipment, which is thereby shown to be inadequate for the purpose. New equipment, which is both versatile and inexpensive, has been developed at the National Institute for Road Research. Through the use of this equipment a convenient chart has been drawn up relating the measured friction ratios to plasticity indices as the soil descriptors, for some South African recent alluvial deposits. This relationship is shown to be similar to that developed in Europe.

DEEP SOUNDING DESCRIPTION OF EQUIPMENT

The system has evolved over the years from the basic concept of pushing a walking stick into the ground to assess the subsoil characteristics. Most penetrometers now consist of a cone which is pushed into the subsoil by a jacking system operating via a string of rods. A wide range of penetrometers is comprehensively described by Sanglerat (1972). The Dutch standard cone has a 60° point of 1 000 mm^2 cross-sectional area and the force required to advance the cone at a standard rate of penetration (20 mm/sec) is recorded. In the earlier versions the outer casings necessarily had the same diameter as the cone so that the force required to advance the casings would give a measure of adhesion or friction. It is unfortunate that in South Africa, because of the desirability of using locally available steel tubing for the casing, a non-standard test has evolved using a cone with a smaller diameter. This is described by Kantey (1951).

A marked disadvantage of the simple sounding methods is that no samples are obtained, hence some boreholes are necessary to identify the material type. However, with the incorporation of a friction sleeve or jacket, illustrated in Figure 1, (Begemann 1953) this disadvantage is to a large extent nullified since a reasonable identification of the material type is possible from the various measurements taken. The friction sleeve has to date had very limited use in South Africa, but the demonstrable value of the technique will undoubtedly lead to its increased application.

The conventional means of pushing the sounding cone into the ground is either by a special sounding rig or a normal diamond drill.

In either case a method is required of sensing the load necessary to achieve penetration of the subsoil by the cone. The conventional system is a closed circuit hydraulic load cell at the top of the sounding rods. Since a fairly wide range of loads occurs, i.e. 0-100 kN, it is normal to have twin hydraulic pressure gauges, measuring cone pressures of 1-10MPa and 0-100 MPa, with the low pressure one protected by an overload valve. With this type of load-sensing equipment the accuracy of the measurements is unlikely to be better than about ± 2% of the full-scale reading of the low-pressure gauge i.e. ± 200 kPa. At one site recently examined (Sea Cow Lake on the Durban Outer Ring Road) some of the soundings gave cone readings as low as 300 kPa and averaged about 750 kPa over a depth of 12 m. It will be seen therefore that the degree of accuracy of the conventional measuring equipment must be considered as somewhat doubtful for very soft deposits. The use of the friction sleeve makes accurate measurements even more important since it is the difference between two measured values which determines the ratio defined as the local friction divided by the cone resistance. The difference has to be evaluated because, as shown in Figure 1, the mechanism is such that the sleeve cannot be moved by itself but only in conjunction with the cone during the cone's second section of travel. Since the cone and sleeve have different areas it is necessary to relate the cone resistance and local friction in terms of loads per unit area and this results in the following standard expression for the calculation of the friction ratio (F.R.).

$$FR\ \% = 6{,}6\ \frac{(\text{Cone plus Sleeve - Cone reading})}{\text{Cone reading}} \qquad \text{..... (1)}$$

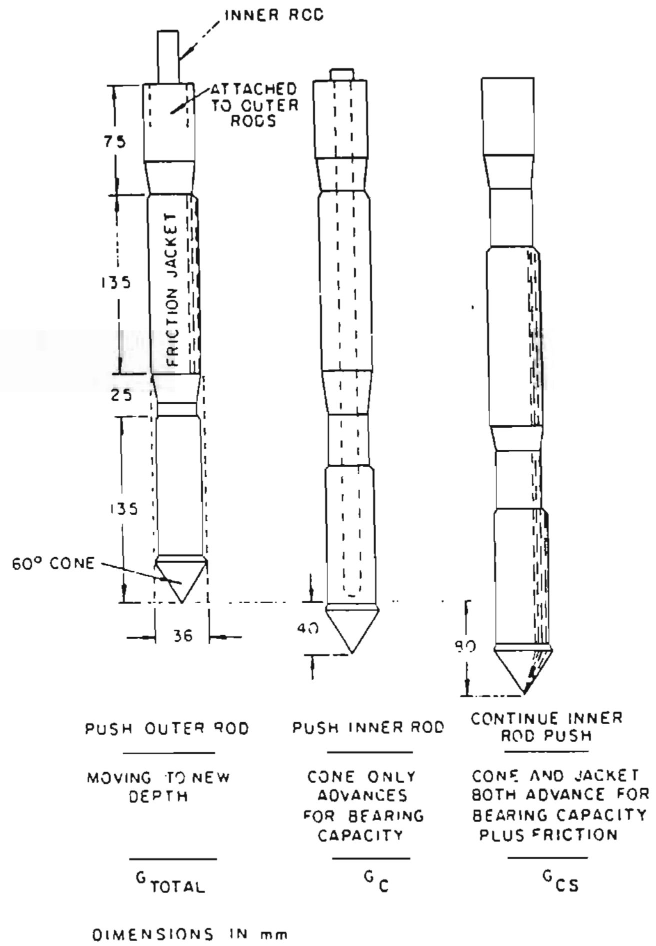

FIGURE 1

MECHANICAL FRICTION SLEEVE CONE

INFORMATION AVAILABLE FROM SOUNDINGS

Three distinct subsoil parameters may be deduced from soundings when these are performed with the friction sleeve:

(a) Shear strength
(b) Compression modulus
(c) Material identification

(a) Shear strength

Normal bearing capacity theory is applied to the failure of a cohesive soil due to penetration by a cone. In this way an equation (Meigh and Corbett, 1970) relating cone pressure, q_c, and shear strength C_u (assuming a $\phi = 0$ condition) is derived of the form:

$$q_c = N_k C_u + z \qquad \text{.................. (2)}$$

where Nk is a bearing capacity factor and z is the overburden pressure. Various other investigators have considered the overburden pressure term to be of little significance thus giving a simplified relationship. The simple relationship was checked at the Sea Cow Lake site by conducting both deep sounding and in-situ vane shear tests. The vane shear equipment was manufactured from non-standard deep sounding casings with the inner rods modified by the addition of square sockets to transmit the necessary rotation to the vane. The latter was made semi-retractable into a nose-cone for protection during advancing between readings. In this way vane tests were carried out without difficulty to a depth of 12 m. An N_k value of 18,4 was obtained by plotting Cu (vane) against q_c (cone) and then determining the best fit straight line through the origin by the least squares method. The correlation coefficient 'r'of this plot was 0,80. This was considered satisfactory to allow the results of deep sounding tests to be used in the preliminary stability analyses of embankments on the soft clay. The vane test results, multiplied by 18,4, are shown on the sounding log in Figure 2.

The value of 18,4 is in reasonable agreement with a frequently quoted theoretical value of 14, and with values of from 12 to 37 mentioned by Webb (1974) for deposits in the Durban and Richards Bay areas. (Webb also interestingly suggests that N_k increases with increasing values of q_c.)

For the purpose of assessing shear strength at the site a conservative value of N_k = 20 was used for simple calculations of factors of safety for circular arc failures from charts. Where the factors of safety were low, further sampling, testing and analysis was carried out.

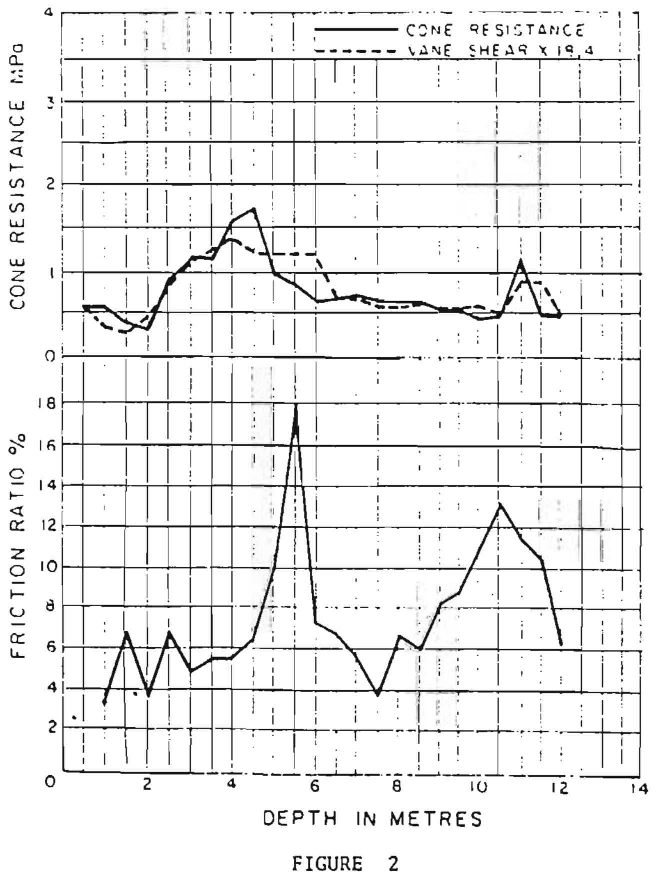

FIGURE 2

SOUNDING LOG SHOWING COMPARISON WITH IN-SITU VANE SHEAR TESTS

(b) Compression modulus

Over a period of about 30 years considerable experience has been gained in the prediction of settlement of structures founded on alluvial saturated deposits. The method of calculation is based on the Terzaghi equation:

$$\delta H = \frac{2,3}{C} \; H \log_{10} \frac{(Po + \sigma_z)}{Po} \quad \text{---------------} \quad (3)$$

where δH is the settlement of the layer being considered.

P_o is the overburden pressure at mid-layer depth
H is the thickness of the layer
σ_z is the increase in pressure at mid-layer depth due to the additional foundation load, and
C is a compression modulus.

A relationship between C and q_c (the sounding cone pressure) was given by de Beer and Martens (1957) as:

$$C = 1.5 \; q_c/P_o \quad \text{-----------------------------} \quad (4)$$

The pressure distribution used to calculate σ_z at depth from the known value at the surface was derived by Buisman, hence the calculation of settlement is usually known as the Buisman-de Beer method. The method is well known and has been used frequently in South Africa with acceptable results. There appears to be a generally held opinion (Meyerhof (1965) and Schmertmann (1970)) that it is over-conservative and that the de Beer relationship should be modified to the following:

$$C = 1,9 \; q_c/P_o \quad \text{----------------------------} \quad (5)$$

This modified Buisman-de Beer method has been used in the design of a number of road embankments currently under construction in South Africa.

In the 1973 Rankine Lecture, Professor Lambe, advocated what he called Type A predictions i.e. those made before the event and dependent on data available at that time. Two such predictions have been made using the method of settlement prediction described above. These are:

(i) Mtwalumi Approach Fill National Road 2, Natal South Coast.
(ii) Sea Cow Lake Embankments, Durban Outer Ring Road, National Road 2.

(i) Mtwalumi

The embankment at present under construction is the south approach fill to a major bridge over the Mtwalumi River. Boreholes for the bridge investigation indicated that at the south abutment the subsoil consisted of up to 25 m of silty sand overlying Dwyka Tillite bedrock. The embankment will be about 16 m high and 90 m wide. Nine soundings were carried out and a typical sounding log is shown in Figure 3. The detailed calculations have been reported elsewhere. (Jones, 1974).

The calculated settlement for the 16 m fill at the position of the sounding log shown is 1,11 m using the unmodified de Beer relationship, or 0,88 m using the modified version. Because of the variations between the different soundings, and for other reasons given later, the overall prediction for the embankment was 0,7 ± 0,2 m, the modified Buisman-de Beer relationship, Es = 1,9 q_c, being considered appropriate. At the time of writing the embankment is only partly complete, i.e. 8 m high, and if the calculation is accordingly adjusted, the predicted settlement will be about 0,45 ± 0,15 m.

The measured settlement at the time of writing averages 0,41 m but it is of course, too early to draw any significant conclusions from this. Further readings after the completion of the embankment should be available before the presentation of the paper so that the validity of the prediction can be checked. The settlements are being measured at two cross-sections through the fill by the system of pulling a water overflow device through a 100 mm - diameter plastic tube. Readings of the height of the overflow are taken by a manometer in a trench at one side of the embankment. Due to the necessity for using de-aired water, considerable practical difficulties have arisen and the system at this site is very time consuming.

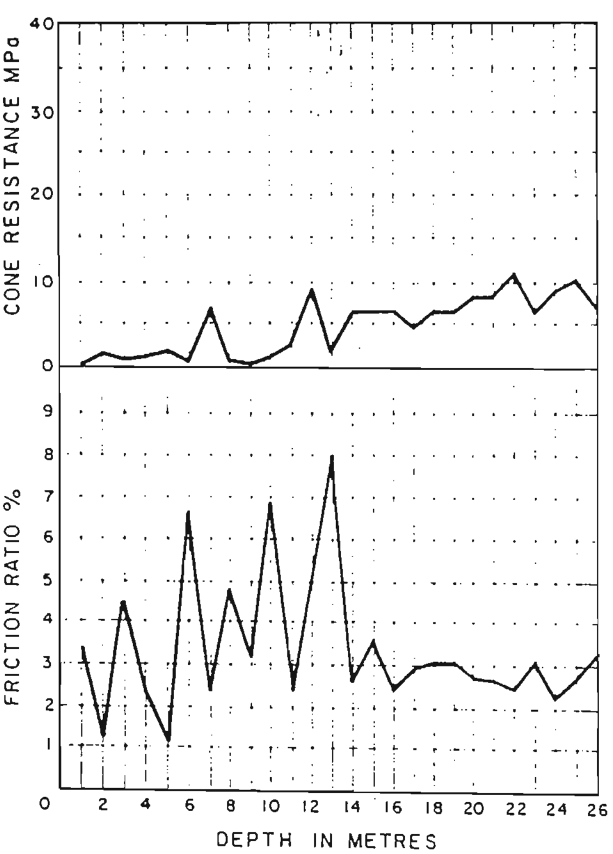

FIGURE 3

TYPICAL SOUNDING LOG FOR MTWALUMI SITE

(ii) Sea Cow Lake

Considerable investigation into the potential problems at this site has been carried out and this work is summarized in another paper presented at this conference. (Jones et al, 1975). The proposed embankments have a maximum height of 8 m. The subsoil is very soft sandy and silty clay to a depth of about 15 m, and overlies Ecca Shale bedrock. Boreholes sunk during a preliminary investigation showed Standard Penetration Test values of less than 5 and in one borehole a zero reading was recorded down to a depth of 7 m. It was considered desirable to construct a trial embankment to check the validity of both settlement estimates and the time settlement characteristics, since the latter could have a critical effect on the overall construction programme.

The trial embankment was 30 m square and 6 m high. Soundings were carried out at the four corners of the fill and from these a settlement of 1,6 ± 0,4 m was predicted, again using the modified Buisman-de Beer equation. The wide limits in the prediction represent an assessment of the reliability of the predictions and also the variation due to the difference in the four sounding values even at such close spacing. A typical sounding log is shown in Figure 2, which also shows the vane shear test results referred to previously.

Settlement measurements were taken at seven points across the centre line of the fill by commercially available mercury-head sensors installed at natural ground level. The readout for these sensors was done by pressure gauge in a hut 2 m outside the toe of the fill. The mean value of the measured settlement over 8 months was 1,2 m after which the measurements had to be discontinued because actual road construction was due to begin. It is estimated from the piezometer readings and from the shape of the time settlement curve, that this would have increased to a long-term value of about 1,4 m. It is interesting to note that the gauge house itself settled 0,13 m.

This Sea Cow Lake Type A prediction for the trial embankment now becomes a very valuable Type A prediction for the actual embankments to be constructed, with appropriate adjustments being made for the differing sounding value and fill heights along the route.

A major objection to the original Buisman-de Beer method was that it frequently produced over-conservative predictions. This appears to have been largely overcome by the simple modification which has been made to the equation. A further objection is that the method takes no account of the different material types encountered. Some variations in the method have been proposed whereby the relationship:

$$C = 1{,}9\ q_c/P_o$$

is modified to: $C = \alpha\ q_c/P_o$ ---------------- (6)

where: $\alpha = \dfrac{2{,}3}{\alpha_o}$ and α_o depends on the soil type.

Kerisel (1968), Bachelier and Parez (1968) and Gielly et al (1970) have given values for α_o. The values of α_o vary from about 0,2 to 1,7 depending on soil type, the cone reading and also moisture content in some cases. If α_o = 1,2 then $C = 1{,}9\ q_c/P_o$. This α_o value would be expected to hold for silty sands of average density which, broadly speaking, is the type of material in which most soundings and settlements predictions have been carried out.

Since the wide variation of the α_o value has a direct effect on the calculated settlement, it is obviously of great importance to identify the material type correctly. It is for this reason that a reliable correlation of friction ratio with material type would be of value.

(c) Material Identification

A considerable amount of evidence has been presented by Begemann (1965, 1969) and confirmed by Schmertmann (1967, 1970) that a relationship exists between the friction ratio and the material type. Begemann shows this relationship in the form shown in Figure 4. It will be noted that the friction ratio is not shown directly, the local or sleeve friction and the cone resistance being plotted separately. The material type is defined by

% < 16 μ size.

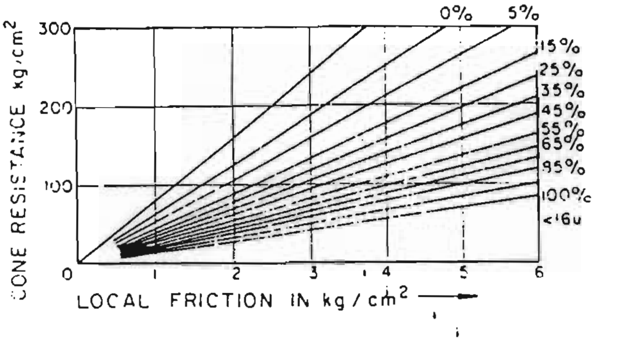

FIGURE 4.

RELATIONSHIP BETWEEN SOUNDING CONE RESISTANCE, LOCAL FRICTION AND PERCENTAGE SOIL PARTICLES < 16 μ. (AFTER BEGEMANN)

It was considered that correlation of this relationship would be valuable for South African soils. Therefore, at a number of sites where soundings were to be used to obtain settlement prediction data, boreholes were also put down immediately adjacent to the soundings and samples taken for laboratory testing.

Results of Initial Field Tests.

Twenty five soundings using the conventional load measuring system were carried out at three sites. A borehole was sunk adjacent to each sounding. Typical logs are shown in Figures 2 and 3. The cone pressure values are divided into ranges in the table below and the number of readings in each expressed as a percentage of the total.

TABLE 1

RANGES OF CONE PRESSURES

Cone Pressure q_c MPa	% of total readings
$q_c > 10$	10
$10 > q_c > 5$	20
$5 > q_c > 2{,}5$	20
$2{,}5 > q_c > 1$	15
$1 > q_c$	35

It will be seen that 50% of the readings at these sites were below 2,5 MPa and it can be seen from Figure 4 that the Begemann chart is difficult to use at such low cone pressure. This does not of course imply any deficiency in the basis for the chart, but merely serves to demonstrate that the cone pressures at the sites tested are generally extremely low.

A further disadvantage of the direct use of Begemann's results for the particular purpose described here - i.e. preliminary estimation of settlement under embankments - is that in soil testing for roads particle size distribution below the No.200 B.S. sieve size is not usually carried out. Hence the % < 16 μ size is not a very convenient parameter.

The present investigation therefore examined the possibility of correlating other parameters against friction ratio. The results are given in Figures 5,6 and 7 in which for 140 samples, taken from three different sites, the friction ratio is plotted against % < No. 200 B.S. sieve; % < 20 μ and Plasticity Index respectively.

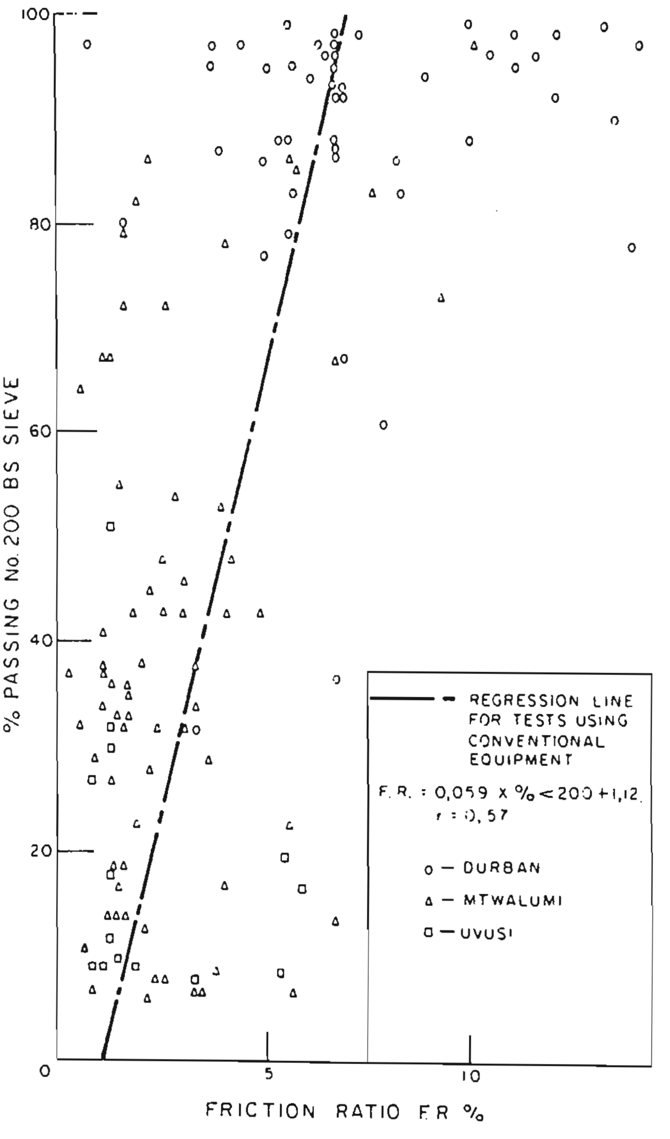

FIGURE 5.

RELATIONSHIP BETWEEN FRICTION RATIO AND PERCENTAGE SOIL < No.200 BS SIEVE

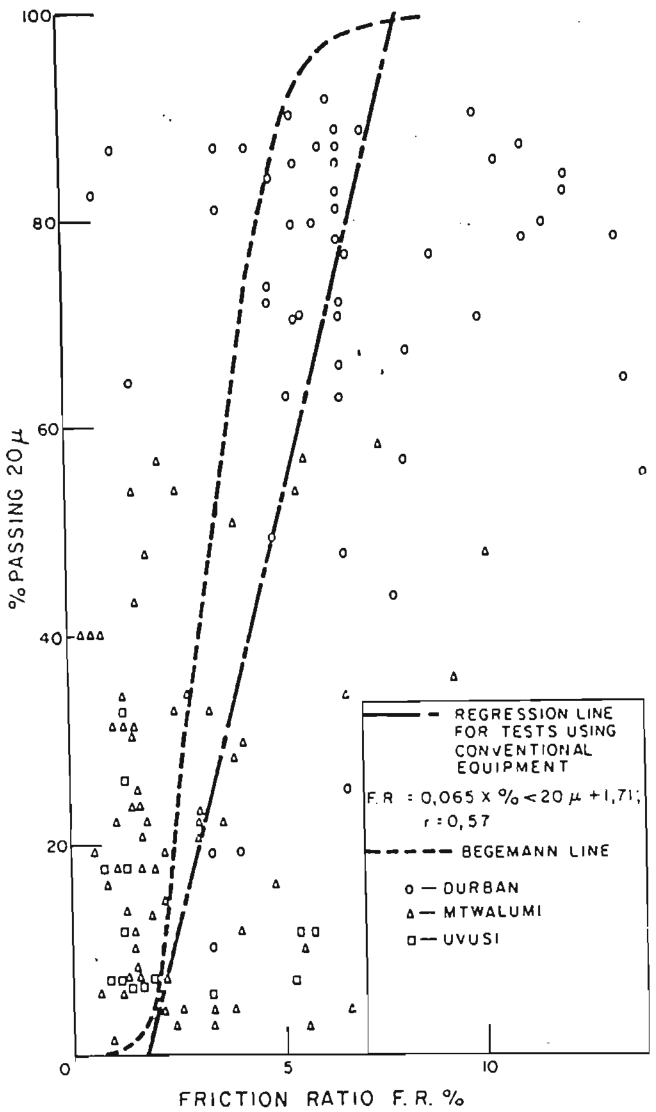

FIGURE 6

RELATIONSHIP BETWEEN FRICTION RATIO AND PERCENTAGE SOIL < 20 μ

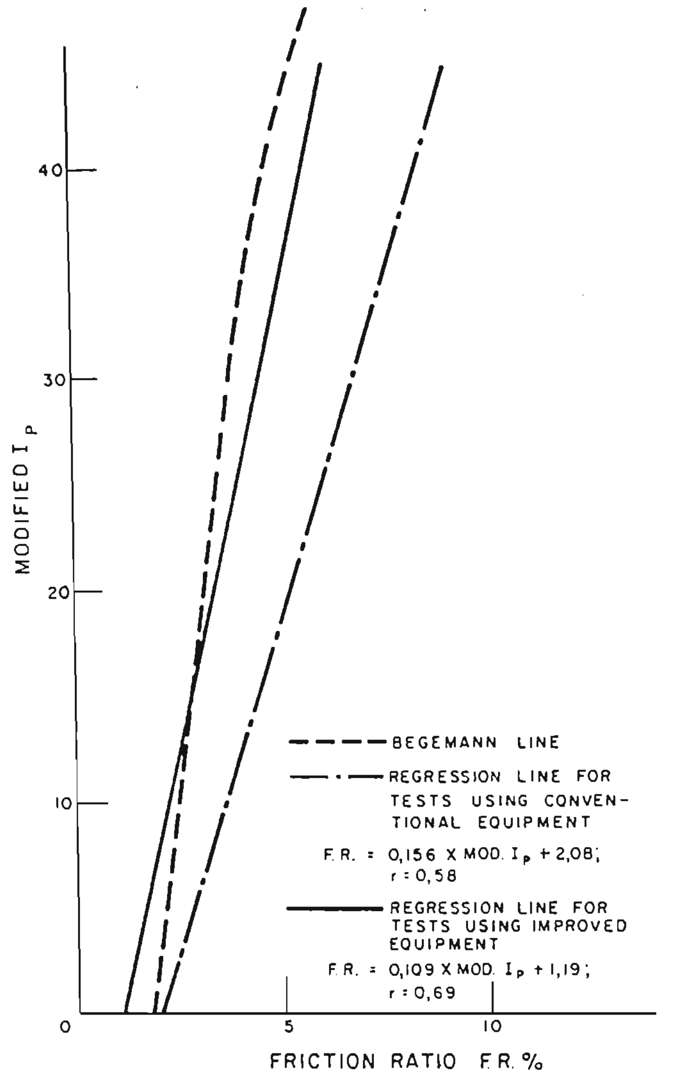

FIGURE 7

RELATIONSHIP BETWEEN FRICTION RATIO AND MODIFIED PLASTICITY INDEX

Statistical analyses were carried out and the resulting least squares linear regression lines are shown together with the correlation coefficients. The other lines shown in Figures 6 and 7 will be discussed later. From these figures the following comments can be made.

a) The correlation of friction ratio with any of the chosen parameters is fairly poor. The correlation coefficient in all cases is approximately 0,57.
With such low coefficients the most that can be said is that there is a trend towards an increase in the friction ratio with an increase in the value of any of the soil parameters representing the clayiness of the material.

b) The order of magnitude of the relationship between the friction ratio and % < 20 μ size is generally similar to that shown by Begemann.

c) Since the results from the three sites are shown differently on Figures 5 and 6 it is possible to see that the correlation coefficient for each site individually would be extremely poor.

d) The correlations for the three parameters show very similar trends.

It can be deduced from these observations that even if a valid simple relationship does exist, it has probably been obscured by inaccuracies in the method of determining the friction ratio using the conventional system since it is reasonable to assume that the probable error in determining the soil parameters is fairly small, say ± 10%. Improved load-sensing equipment is therefore required.

IMPROVED LOAD SENSING DEVICE

The various methods used elsewhere were briefly considered (de Ruiter 1971;Heijen 1973;Joustra and Fugro 1973). However, it was decided that most of these, whilst undoubtedly giving the desired accuracy, involved unacceptable costs and complications. The method developed concentrated on simple and cheap modification of the present sounding system. The hydraulic load cell was merely replaced by an electrical strain gauge system installed in the mechanism used to transfer the machine ram thrust alternately from the sounding casings to the sounding inner rods. When using a friction sleeve it is imperative that frequent readings should be taken during sounding, so that changes of material in a multi-layered subsoil can be detected and, if necessary, a corrected friction ratio calculated. With hydraulic gauges it is a very tedious task to record all the measurements. The improved device therefore records the output from the strain gauge load cell directly and automatically onto a chart. After some prototype testing, a simple and convenient system was made up which can be used on any conventional diamond drill or sounding machine. Figure 8 shows the complete equipment which consists of an upper and lower box.

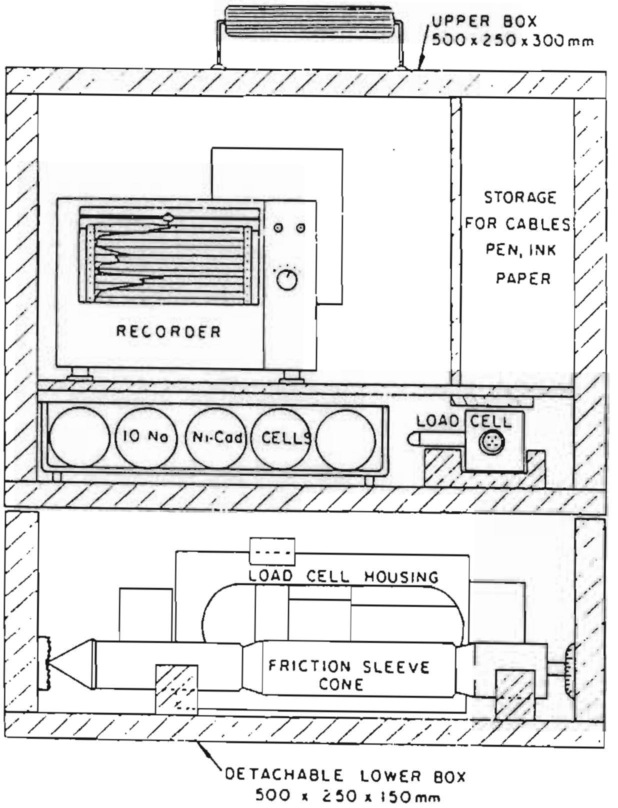

FIGURE 8

DIAGRAM OF SOUNDING LOAD MEASURING EQUIPMENT IN CARRYING CASE.

The upper box contains the chart recorder and nickel-cadmium, rechargeable battery, power supply together with space for the load cell, leads, spare chartpaper, pen and ink. The lower box, which can be detached, is simply a container for the load cell housing and two complete friction sleeve cones. The upper box can be used independently as a battery-operated recorder in the field or laboratory for purposes other than sounding. The chart on the recorder in Figure 8 shows three typical cone and cone-plus-sleeve readings.

The load cell was calibrated in the laboratory against a range of provings rings (0,9 kN to 27 kN) and checked for repeatibility of results on numerous separate occasions by different operators both before and after field use of the equipment. The limiting factor in the accuracy is the reading of the chart. With the range of sensitivity available it is possible to measure to say ± 10 N at the most sensitive, so that in terms of cone pressures the accuracy is within ± 10 kPa. This is about twenty times more accurate than a conventional hydraulic load cell and pressure gauge system.

Comparison of improved with conventional load sensing system

A comparison was made at one site by taking readings with the two systems simultaneously. The results are summarized by the following linear regression equations and correlation coefficients for the cone readings and for the friction ratios. y and x are the new and old system readings, 60 of which were taken.

a) Cone Pressures - kPa: range 0 - 1400 kPa
$y = 0{,}96\ x + 20{,}6;\ r = 0{,}77$

b) Friction Ratios- % : range 0 - 20%
$y = 0{,}68\ x + 3{,}2\ ;\ r = 0{,}30$

It will be seen that although the two systems show moderate agreement for the cone pressure, the friction ratios are not comparable. Although the site chosen for the comparison, Sea Cow Lake, could be considered as particularly severe in that the subsoil is very soft and clayey, it demonstrates that the measurement of friction ratios with conventional equipment under these conditions is impossible.

Use of improved load sensing equipment

Since the equipment is new the opportunity has arisen for only one further site to be investigated so far. Fortunately, this site, on the proposed Johannesburg Western Bypass, has a fairly wide range of soil types, from non-plastic, slightly silty sand to soft, silty clay. Five soundings were carried out in order to estimate the potential settlement of a proposed embankment.

The results in terms of friction ratio and plasticity index are shown on Figure 7 as a linear regression line. The correlation coefficient for this site is $r = 0{,}69$. Figure 7 also shows the line for all the previous samples from various sites examined with the earlier equipment.

The conclusion to be drawn from this comparison is that there is a considerable improvement in the correlation. This must be attributed to the improved measuring system since the determination of the soil parameters was carried out by the same laboratory in all cases.

INTERPRETATION OF FRICTION RATIOS

Having shown that the improved measuring system is sufficiently accurate even when the subsoil is very soft or loose, it is possible to ascertain whether or not the friction ratios measured give the same indication of material type for South African soils as Begemann (1965) deduced in Europe.

As already pointed out, the representation used by Begemann is not convenient for soils with very low cone pressures. An alternative method must therefore be used. The relationship can be shown as friction ratio against % < 16 µ size and this is done on Figure 6. There is a small discrepancy due to the use of slightly different particles sizes, but for the samples tested the difference is only about 1 or 2 % and an adjustment to this effect has been made in the plotting on the % <20 µ graph.

It is clear that the Begemann relationship is non-linear and therefore any attempt to obtain high correlation coefficients with a linear regression line are doomed to failure. However, examination of the line indicates that between friction ratio values of about 2% and 5,5% the relationship is sensibly close to linear. To avoid the complexities of statistical analysis of non-linear functions, it was decided to use linear analysis but accept that the resulting line would be unlikely to achieve correlation oefficients of the order of 0,9, a value normally considered to be satisfactory. An analysis using only those points where the friction ratio is between 2 and 5,5 would have severely reduced the number of samples available and was therefore rejected.

As already mentioned, the use of the < 16 µ or < 20 µ size as a parameter presents the problem of carrying out inconvenient hydrometer laboratory testing. Other parameters representing the material type were therefore examined. Both % < No. 200 B.S. sieve and the plasticity index expressed as that of the whole sample were used. The modified I_p is simply the standard I_p carried out on the fraction passing the B.S. No.36 sieve multiplied by the % passing the B.S. No.36 sieve.

A statistical analysis gave the following least squares linear regression equation for the total of 183 samples tested. It is of course valid to use the results of the laboratory tests on both the old and new samples since this comparison is not concerned with sounding measurements.

Modified I_p = 0,55 (% < 20 µ) - 2,36 ------- (7)
with r = 0,88

The regression lines for the earlier and latest sites separately and combined are shown on Figure 9. It is not suggested that this is in any sense a valid general relationship for all soils but only that it holds for recent alluvial deposits in local conditions and that for the purpose described here it is useful.

The Begemann line, shown on Figure 6 as a plot of friction ratio against % < 20 µ, can then be transformed to a plot of friction ratio against I_p (Figure 7) by simply converting the % < 20 µ in Figure 6, for any particular friction ratio, to an I_p by using equation 7 given above.

It can be seen that the agreement between the latest line and the Begemann line in Figure 7 is very close in the linear range. There is a little divergence at the low friction ratios and rather more divergence at high friction ratios. However, bearing in mind the purpose of friction ratio measurement, which is to provide a reasonable description of the material, these divergencies are not of great significance. If the friction ratio is outside the linear range then the description is well defined either as clean sand if F.R. < 2, or as medium to high plasticity silty clay if F.R. > 6.

For convenience a chart, Figure 10, showing friction ratios against material type subdivided into zones, is given for use with local alluvial deposits.

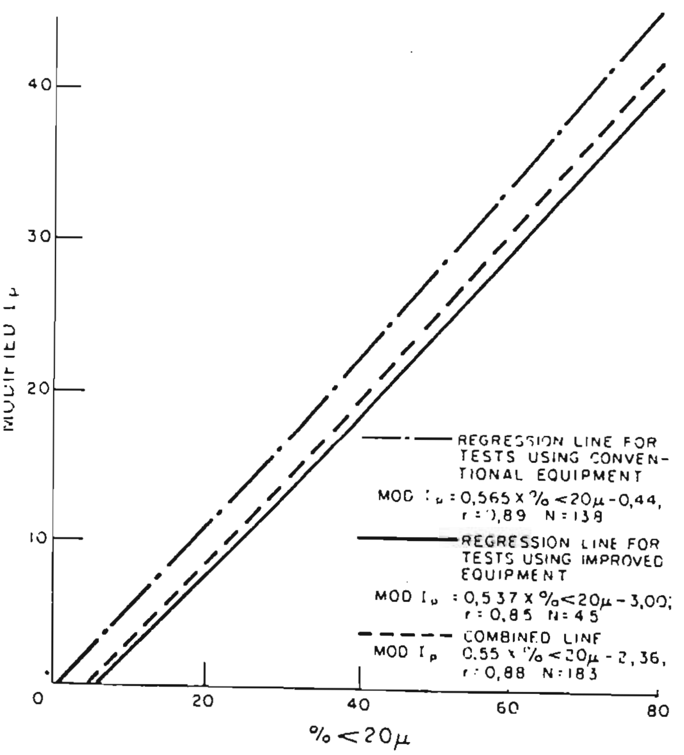

FIGURE 9.

RELATIONSHIP BETWEEN MODIFIED PLASTICITY INDEX AND PERCENTAGE SOIL < 20 µ

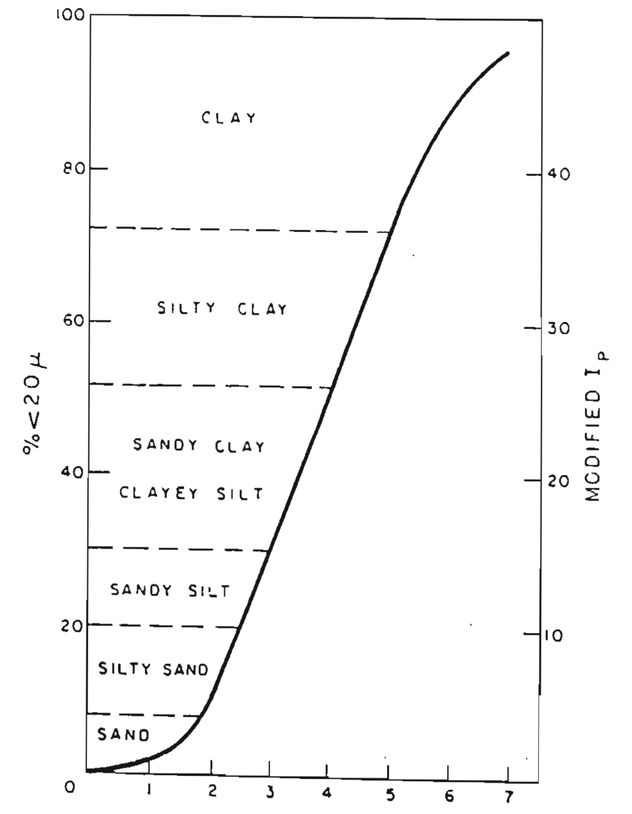

FIGURE 10.

RELATIONSHIP BETWEEN FRICTION RATIO AND SOIL DESCRIPTION

Further information should be obtained to increase confidence in the chart or modify it if necessary.

The primary purpose described herein of measuring the friction ratio, and thus providing a description of the material type, is so that settlement calculations may be modified accordingly.

However, there is a further important advantage in having a reliable material description. It is that a preliminary assessment of the time settlement characteristics of the subsoil becomes possible. There is as yet no formal way in which this can be done but it is reasonable to hope that a relationship may be established between friction ratio and C_v, the coefficient of consolidation via the plasticity index. It should be stressed that it is not the intention that sounding should replace all other testing but solely that the maximum benefit should be derived from soundings carried out so that reasonable assessments of potential subsoil behaviour can be made at an early stage of investigation.

CONCLUSIONS

1. Deep sounding is an extremely effective method for the estimation of the settlement of embankments on alluvial saturated deposits.
2. The development of the friction jacket has considerably increased the usefulness of the method because the resulting definition of the material type enables:
   a) the calculated settlement to be modified; and
   b) an assessment of the time-settlement characteristics to be made.
3. Measuring systems currently in operation are probably unable to provide the accuracy necessary for friction ratio measurements in very low density subsoil.
4. An improved strain gauge system has been devised which is cheap, sufficiently accurate, convenient to use and has the very useful facility of automatically recording the results. The equipment has been used successfully in the field.
5. Using the new equipment, friction ratios have been determined and compared with different soil parameters, these being the I_p, the % < 20 μ and the % < No.200 B.S. sieve. For the sites examined a simple relationship exists between I_p and % < 20 μ which enable the Begemann chart to be transformed into a line on a friction ratio against I_p plot. Close agreement exists between that line and the results of the field tests.
6. It is therefore concluded that a revised chart - Figure 10 - may be used to obtain descriptions of very low density or soft soil from the friction ratios, provided that the measuring system is adequate.
7. In order to increase confidence in the use of sounding friction ratios for determining material type, more sounding and associated sampling should be carried out and the results correlated.

ACKNOWLEDGEMENTS

The author would like to thank Mr. F. Reid and Mr. J. Vorster who designed and manufactured the improved load measuring device and who patiently made the subsequent modifications to produce the final version of it. Thanks are also due to the Director of Roads of the Natal Provincial Administration, to the Secretary of Transport and their Materials Engineers who made the sites and the field equipment available for much of the testing.

The paper is published with the permission of the Director of the National Institute for Road Research.

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designed bearing capacity of the material exceed 4 000 kPa. It would seem that the full potential strength of the material was not used, although in some cases I realise the size of the shafts were determined for practical reasons of installation. It would be interesting to know from the authors whether smaller sections to carry the same load at the same depth would have been used had the rock drilling equipment now available been on the market at the time of construction of the four case histories mentioned in the paper.

On Page 163 the authors state, "High grade concrete stressed to appropriate working stresses can be safely used. In general therefore less concrete is used than in piled foundations". This would not appear to be a material factor in the economics of deep founding techniques since the cost of concrete probably represents less than 10% of the total cost of installation, and with special care there is no reason why the concrete in high capacity piles should not be fully stressed as well.

CONTRIBUTIONS TO THE PAPER DEEP SOUNDING - ITS VALUE AS A GENERAL INVESTIGATION TECHNIQUE WITH PARTICULAR REFERENCE TO FRICTION RATIOS AND THEIR ACCURATE DETERMINATION BY G.A. JONES.

PROF V.F.B. DE MELLO

There is much to be discussed in this very important session of great practical implication particularly in the light of our experiences of the last thirty years during which the principal cities of Brasil have grown at incomparable rates (such as that of São Paulo with an increase of population from about 2 to about 8 million). This has involved an unprecedented rate of construction of highrise buildings because of disproportionately slow expansion of the area served by public utilities. However, because of lack of time I shall restrict myself to three principal items.

The first concerns the paper by G.A. Jones on the use of the deepsounding static cone penetrometer together with the local friction sleeve as a preliminary all-purpose tool for subsoil investigation.

When Begemann presented his suggestion of the friction ratio as "an aid in determining the soil profile" (1965), which could better have been emphasized to be a complement to visual-tactile classification of spoon-sample exploratory borings, I took the liberty to submit a discussion (6th ISSMFE, Montreal 1965, Vol. III p.294) decrying the introduction of mechanistic practices that would wipe out the painstaking gains of the fundamental principle of Soil Mechanics of requiring first the determination of the nature (classfication) of the soil type by direct sampling, and not by indirect inferences.

Mr Jones begins by pitting undisturbed sampling of soft alluvial deposits against the proposed deepsounding procedure. But the latter is an Index Parameter procedure, that should be compared with the alternate index parameter procedure of the exploratory boring: the latter assumes that by visual-tactile classification one obtains principally the determinations of parameters connected with strength as the satisfactorily dominant conditioner for problems both of compressibility and of stability. Moreover, does one forego the need to determine the water level in borings?

Indeed the lure of mechanistic proposals is extremely seductive. And doubtless there will be many a case of pragmatic success. Meanwhile, doubtless the presumed rationalizations employing the index parameters extracted from the exploratory borings do need considerable revision, to redeem them from an accumulation of criticisms on poor predictions. But, can one forego a fundamental principle of directly qualifying the materials, without running serious risks, not merely of practical failures (not yet statistically established because of the limited number of applications of the new method) but worse, of undermining the very roots of the engineering science?

The author is much commended for the improvements introduced and the carefully collected supporting evidence. I should beg leave to request, however, that much greater emphasis be attached to the restrictive hypothese; for instance, on the one hand regarding shear strength estimations, the assumption of fully saturated type of $\phi = 0$ soil (most partially desiccated and preconsolidated alluvia even when cyclically submerged would not satisfy the condition for a small pressure bulb under the cone), and, on the other hand regarding settlement computations, the presumed pseudo-elastic instaneous compression condition of the Buisman-de Beer- Schmertmann computation of sandy soils. The introduction of a new Index Parameter, such as the Modified I_p may improve statistical correlation coefficients; but, with reference to fundamental parameters and the theory of soil mechanics, is there any justification (or can and should one be sought) for the presumed trend, or is it a case of statistics at random? It must be recalled that for routine exploration, the cone penetrometer suffers from the serious drawback of being in essence too sensitive a test, subject to highly localized extreme values. If as much effort of development and correlation were expended on the sampling exploratory boring (such as, for instance, using a static penetration effort and a Swedish foil long-sampling idea), would not the returns to soil engineering both conceptually and pragmatically, have a probability of being much greater?

The second point on which I beg leave to comment is the fact that foundations and foundation design seem to be discussed at this Conference without sufficient indication of the subsoil profile (in a soil mechanics context) or of the magnitudes and distribution of column loadings and I find myself quite at a loss in the attempt to assess comparatively your practices with ours. If I may say so, you seem to be map conscious and geology conscious; in our experience, geology is merely the compulsory context within which to begin engineering investigation, but does not lead to any indices quantifiable to the degree required of engineering decisions, especially in urban foundation engineering concerned with restricted areas and depths and finer property differentiations. For instance, in paging through the Proceedings Volume I could single out two places (pages 194, 244) in which subsoil profiles are complete for foundation engineering, with classification of soil types and consistencies, and three other places (p.84, 257, 264) in which there are subsoil profiles, but without quantification of denseness or consistency.

Since we have developed all of our routine first-approximation foundation design decisions on the basis of exploratory boring with SPT indices, the boring profiles shown on pages 194 to 197 would appear to indicate conditions for economic shallow footings foundations for buildings of about 25 to 30 storeys; but one must be careful to check against possible gross differences in SPT values that can be produced by unstandardized factors of the test, leading one completely astray in comparisons (which is best avoided by comparing with static cone penetrometer values or by comparing SPT values in a standard material such as clayey fill compacted to similar specifications).

I should therefore begin by requesting that a typical exploratory boring profile of Durban be published in the discussions, alongside referenced information on static cone penetrations and/or fundamental data on plate load testing, and alongside a typical plan of column loads of a highrise building of specified number of floors.

For your reference I may summarise the following first order approximations widely used in Brasil. Reinforced concrete building loads correspond to about 1,2 tons per square meter of area in plan, per floor (thus 1,2 n t/m^2 would be the equivalent average pressure on a hypothetical raft). Rafts have never proved necessary or economincal; if the allowable bearing pressure on pad footings is higher than about 1,8 n and therefore overall pad areas add up to less than 65% of the plan area, pads are more economical than piles of the order of 10 m; the competitive limit of percentage area occupied by shallow pads continues below 100% up to the longest piles used (about 25 to 30 m). For somewhat preconsolidated silty clays and clayey sands with $3 < SPT < 25$, plate load tests have suggested allowable bearing pressures equivalanet to ($\sqrt{SPT}$ - 1)kg/cm^2 as being satisfactorily conservative, dispensing with computations of settlements and differential settlements on routine range of variation of column loads. Pier foundations have accepted nominal base pressures (assuming zero friction) of the order of 2 to 3 times the above shallow pad indications. For driven displacement piles, the pile length necessary to permit the design load is estimated on the basis of SPT values along the boring profile, with one value per meter of depth; if the layers are considered capable of concomitant contribution to friction and point, the length is such that Σ SPT = compressive stress in kg/cm^2 on the nominal concrete section (for instance, a 30 x 30 cm pile for 40 tons would penetrate to about ΣSPT = 45); if there is a significant distinction between soft upper layers and embedment into a dense substratum of point resistance, the substratum will be penetrated to where ΣSPT point = one-half the compresive stress.

Such secret unwritten rules are denied as soon as they are passed along, but have served for most preliminary designs; and since in Brasil most often construction follows rapidly upon preliminary design they may be claimed to have been proven. I submit them merely so that they may be compared, challenged, and put to shame. However, in candid contrast may I question whether the assumptions of c and Ø parameters for "upper strata" and "Cretaceous material" in the paper by Everett and McMillan are not merely cloaked with an appearance of acceptable theorization; the crucial problem of pile or pier foundation design is "how were these parameters established"?

Finally, the third point concerns the all-important "execution effects" both for lateral friction and for stress-strain behaviour of the concreted base. In particular, the effects of bentonite (and presumed bentonite cakes) on skin friction and on base compressions have drawn much attention and testing in the past few years (cf. for instance some papers at the European SMFE conference, Madrid 1973). The papers by Everett and McMillan, and by Wates and Knight, tackle a problem of the greatest interest and concern.

In Brasil, and particularly in the kilometers of deep slurry walls of the São Paulo and Rio subways we have had considerable success in general; but one must carefully guard agianst the new "philosopher's stone" complex. Each case must be examined separately; bentonite is not a cure-all, and there are many cases where it is unnecessary or may even be damaging. Of special note to this Conference is the admonition that in soils above the water table a "dry" perforation technique that preserves the benefits of capillary tension may be very much better, since despite the best of bentonite slurries the contact with free water frequently causes catastrophic damage to the soil.

DR A.P. TYRRELL

In the concluding paragraph of the section in Mr Jones' paper entitled 'Interpretation of friction ratios' it is stated that "it is reasonable to hope that a relationship may be established between friction ratio and .... the coefficient of consolidation ....".

I wish to challenge this statement for it is totally un-reasonable to have such a hope.

In general terms, the deep sounding technique is a useful tool, but it should be realised that it has severe limitations. For alluvial clays, for instance, it provides only a very superficial means of identification. There is no substitute for visual examination of these clays which have been found to exhibit a natural fabric (e.g. fine sand and silt layers and partings, rootlets, etc.,) capable of dominating mass performance in the field.

In order to be able to make realistic engineering predictions, therefore, it is essential to identify this soil fabric and then to appreciate its influence on mass behaviour (Tyrrell 1969). Descriptions such as those shown in Figure 10 of the paper viz., 'clay', silty clay' can be entirely misleading with regard to field rates of consolidation (Rowe, 1968) and highlight the limitation of the deep sounding technique.

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TYRRELL, A.P. 1969. Consolidation properties of composite Soil deposits. Ph.D. Thesis, University of Manchester.

DR B.C. VAN WYK

The first Dutch cone apparatus had a conical point which was attached to the internal pushing rods.

The main objection to this point was the sharp drop in point resistance observed when the soil broke in behind the cone. The operators had to be attentive or they could miss the maximum point resistance reading. Subsequently the jacket cone penetrometer was introduced and this gave a more continuous resistance and the maximum value was easier to determine. Some engineers were reluctant to use the jacket cone as it apparently gave higher point resistances.

The new friction sleeve cones are again geometrically different from the previous cones. I would like to describe the apparatus tested by the Delft Soil Mechanics Laboratory (Heijnen 1973). It was found that this apparatus gives the best agreement with the jacket cone. It had an 28 mm narrowed part, 200 mm long directly behind the conical point with a 15 000 sq mm friction sleeve at a distance of 300 mm behind the cone point.

The author also mentioned that the settlements predicted with the compression modulus calculated from the point resistance of the Dutch cone was found to be on the high side. Quite a number of factors contribute to this discrepency and I would like to comment on one aspect.

Tests carried out under laboratory conditions in large containers on clean homogeneous sands by Kerisel (1964) gave a particular penetration diagram.

Over the first 15 diameters there is a sharp increase of penetration resistance with depth which is attributed to the development of the complete deep foundation failure pattern, which, incidently, has not been recorded by any investigator so far. After this the penetration diagram bends down and becomes pratically constant with depth at about 30 diameters. With the overburden pressure increasing with depth and the point resistance constant, this implies that the material is becoming more compressible with depth, and that under an increasing normal stress.

The picture of a dense layer under the overburden of a soft layer follows from the work of Thomas (1968). He carried out penetration tests in a rather small container with the effect of a surcharge simulated by a water pressure on a latex membrane and a steel plate. For a constant surcharge pressure the resulting penetration resistance was constant with depth from the surface down. The penetration resistance increased with increasing surcharge pressure but tended to reach a maximum at about 550 kPa surcharge or the equivalent of 4 m of overburden. This ties in with the work of Kerisel. It also implies an increase of compressibility with depth after about 4 m.

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K. SCHWARTZ

In reading this paper, coupled with a paper by the Author and others reported in Session 5 of the Conference, it appears as if some success has been achieved in predicting settlements for road embankments using deep sounding test results with the modified Buisman-de Beer method of settlement calculations.

This method, proposed by de beer and Marstens (1957), is based on the standard Terzaghi settlement equation with the relevant relationships being given in equations 3 and 5 of the paper. These equations have in general been considered valid for saturated loose sands in which the stress increment due to the applied load is small compared with the overburden pressure, and in which Boussinesq's theory of stress distribution is valid. In addition, in assessing settlements it is necessary to distinguish between normally consolidated and over consolidated deposits.

During the field work stage of recent projects it has been necessary to carry out preliminary settlement predictions using deep soundings in saturated cohesive clayey silt soils. In these calculations the Buisman-de Beer relationship for the calculation of the compressibility modulus in saturated loose soils has been modified as follows, in an attempt to take the soil type into consideration.

$$C = 1,9 \frac{q_c}{p_o} \quad \text{----------- Equation 1}$$

is modified to
$$C = \alpha \frac{q_c}{p_o} \quad \text{----------- Equation 2}$$

where
$$\alpha = \frac{2,3}{\alpha_o} \quad \text{----------- Equation 3}$$

The methods proposed by Gielly et al (1970) have been used to estimate values for α_o. The values of α_o vary considerably with the soil type and have a direct influence on settlement calculations. No field correlations are available at present to check on the validity of the above assumptions. The Author has, however, carried out similar investigations in cohesive soils on the Sea Cow Lake site. In the settlement calculations reported by the Author and others in a paper presented in Session 5, correlations have been made with field measurements. The predictions using deep sounding and the field measurements correlate reasonably well, even although Equation 1 above has not been modified to take into account the fact that the deep soundings have been carried out in sandy or silty clays. The validity of using an α_o factor as given in Equation 3 above is questioned, and further comments by the Author would be appreciated.

The Author's work on correlations between friction ratios and material type is most interesting and could become a useful tool in site investigation if the reliability of the method could be proved. If one considers the friction ratios obtained from deep soundings on the Mtwalumi site as given in Figure 3 of the paper, in association with Figure 10, one obtains variations in soil type from a clean sand to a clay over very small vertical distances. One queries the reliability of the method when the Author states that boreholes drilled on the site gave a soil profile of 25 m of silty sand overlying Dwyke Tillite Bedrock.

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ii. GIELLY, J., LAREAL, P. and SONGLERAT, G. Correlations between in-situ penetrometer tests and the compressibility characteristics of soils. Conf. on in-situ investigation in solids and rocks. London.

AUTHOR'S REPLY:

The paper on Deep Sounding may be divided into three sections:

1. Prediction of settlement of embankments from sounding results.
2. A description of some modifications to the conventional load measuring devices.
3. The use of friction ratio measurements obtained from the mechanical friction sleeve cone.

The first of these, embankment settlement, will be discussed in greater detail in the session on embankments; however, I would like to mention that this paper gives genuine predictions for settlements of embankments before they were constructed, a practice which I am happy to see Dr Burland advocates.

The second section deals with a fairly simple modification which we, at the National Institute for Road Research, have made to the usual load sensing device for probing. This has advantages over the usual hyraulic load cell and gauges in that it is more accurate and that a permanent record is made of the output on a chart recorder. It is appreciated that very much more sophisticated devices are available but in general they are expensive and relatively complicated.

The third section has the most relevance to piling and that concerns the use of friction ratios measured by the mechanical friction sleeve in the construction of a soil profile. What we have done certainly makes no claim to be original - we have simply confirmed that for some recent alluvial deposits in this country the conventional friction ratio interpretation is applicable. We have found it convenient to express the results in a somewhat different manner and give a simplified chart of friction ratio against soil type. The relevance of this to piling is of course primarily for friction piles where a knowledge of soil type is essential for meaningful calculations to be made. The book by Sanglerat is vital reading on this subject.

Generally the main point being made is that sounding is a very convenient tool, backed by years of experience elsewhere, and we should be making more use of it. On the other hand, the last thing I want to do is to suggest that sounding is the only, or most important, technique which should be used, or that some of the semi empirical factors associated with interpretation should be viewed with mystical sanctity.

CONTRIBUTIONS TO THE PAPER TIME EFFECTS ON THE LOAD CARRYING CAPACITY OF DRIVEN PILES IN THE DURBAN AREA BY S.B. SHARRATT

DR W. NEELY

The Author has presented some interesting data on the increase in pile bearing capacity as a result of redriving at various intervals of time after initial driving. The prediction of pile capacity was based on the Hiley pile driving formula. This procedure or indeed any method of load prediction based on dynamics will only give the pile capacity immediately after driving. In normally consolidated clays the pile capacity usually increases after driving giving rise to the commonly observed phenomenon of 'set up'. In overconsolidated clays and sands the pile capacity may decrease with time.

Any appreciation of the factors which are responsible for increases in pile capacity with time may have an important commercial value in addition to providing a more meaningful guide to the time lapse between driving and testing in order to take advantage of the 'set up' behaviour. In Norway and Sweden, for example, it is established practice to test piles no sooner than a month after driving, Bjerrum, Hansen and Sevaldson (1958) have found that this is sufficient time for a pile to reach maximum capacity.

It has been well known for many years that pile driving results in an increase in porewater pressures in the surrounding soil which are then dissipated after driving by horizontal drainage. The rate of drainage is clearly controlled by the permeability of the soil although the pile properties may also have a significant influence.

The porewater pressures created by pile driving can be estimated by considering the soil displacement around the pile as illustrated in Fig. 1. Let the initial in-situ vertical and horizontal effective stresses be $J'vi$ and $K_oJ'vi$ respectively and the porewater pressure prior to driving be ui. During driving the direction of maximum displacement is radially outwards and therefore within the zone of radius R_1 the radial stress becomes the major principal stress. According to Lo and Stermac (1965) the maximum porewater pressure, ΔU_m, is comprised of two components as follows:

$\Delta U_a = (1-K_o)\ J'vi$ (due to change in ΔJ_3)

$\Delta U_s = (\Delta U/P)_m\ J'vi$ (due to shearing)

The maximum porewater pressure produced during driving can then be found provided the coefficient of earth pressure at rest, K_o, and the maximum pore pressure ration $(\Delta U/P)_m$ can be measured. $(\Delta U/P)_m$ can be determined from consolidated undrained triaxial tests with porewater pressure measurements while K_o can be estimated for normally consolidated clays from the work of Kenney (1959).

The distribution of porewater pressures around a pile is usually assumed to vary according to the relationship shown in Fig. 2.

$$\frac{U_R}{U_M} = \frac{R}{R_1}$$

This distribution has been substantiated by field measurements of porewater pressures around driven piles, e.g. Hanna (1967). The rate of dissipation

(ii) PREDICTION OF TIME FOR CONSOLIDATION FROM SOUNDING

Prediction of Time for Consolidation from Sounding

Prédiction de la Durée de Consolidation des Matériaux Provenant de Sondages

G.A.JONES Soil Engineering, National Institute for Transport and Road Research, South Africa

SYNOPSIS For road embankments over alluvial deposits the time taken for settlement is often of greater significance than the magnitude of settlement. A method of estimating the time, based on deep sounding, is proposed in which the test is carried out at a constant stress. In order to demonstrate the feasibility of the proposal, a series of laboratory tests was carried out comparing consolidation characteristics measured by constant stress penetration, with those measured by conventional consolidometer tests.

INTRODUCTION

The planning of routes for highways requires preliminary geotechnical investigations. Estimates of the amount of settlement of embankments and, equally important, the period of settlement are often required. For such investigations an indication of the order of magnitude i.e. 0,1; 1 or 10 years may well be sufficient.

In South Africa, quasi static penetrometer testing (deep sounding) is used almost on a routine basis for estimating the settlement of embankments on alluvial deposits (Jones 1975, Webb 1974). Although discussions about the theoretical interpretation of penetration testing data are continuing there is little doubt that the use of the technique on a semi empirical basis is justified. At present the drawback of sounding is that it gives no indication of the duration of settlement.

It is here reported that estimates of the required consolidation characteristics were made from penetration tests carried out at a constant stress, instead of at the more usual constant rate of penetration, on a series of laboratory samples on which consolidometer tests were also carried out.

TEST RESULTS

A standard mechanical friction cone penetrometer was mounted vertically in a frame as shown in Fig. 1. A loading platform was attached to an extended inner rod and an LVDT connected to a chart recorder was arranged to measure the movement of the platform. Soil samples were prepared in an inner bucket surrounded by a water jacket. The inner bucket had perforated sides and was lined with a filter fabric. The frame was designed so that the penetrometer could be moved to different depths in the bucket. In this way a number of tests could be carried out on the same sample. A piezometer was fitted into the head of the cone for some of the tests. It consisted of a pressure transducer located inside a cone with porous stone windows in the face.

The procedure was analogous to consolidometer testing in some respects. Loads were added to the platform and a series of graphs of deformation as a function of time was obtained. On completion of a test the cone was lowered to a new position and a further test carried out. Three soil types were used, silty sand, clayey sand and a silty clay. The first two samples were prepared in the bucket whereas the clay was an undisturbed sample extracted by an excavator from a site currently being utilized for embankment studies (Jones et al 1975).

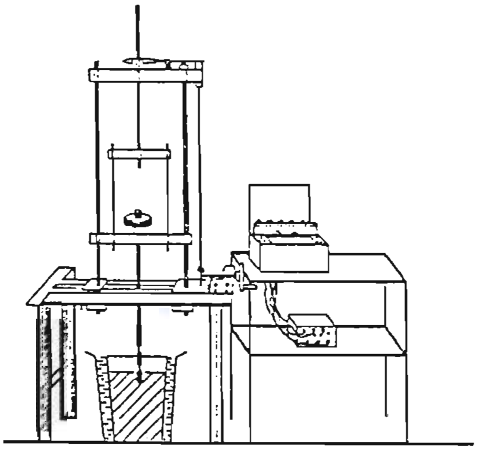

Fig. 1 Constant stress penetrometer

The classification test results on the three soils are given below:

Sample Description	Particle Size Distribution %			Atterberg Limits	
	> 60u	<60u >2u	< 2u	W_L	W_P
Silty clay	13	35	52	56	35
Clayey sand	55	22	23	33	20
Silty sand	93	7	-	N.P	

The results of standard colsolidation tests on the three samples are shown on Fig. 2.

The results of the deformation-versus-time penetrometer tests are given in Fig. 3. The tests on the clay showed evidence of what may be regarded as secondary consolidation. The end of primary deformation was graphically determined for each material and all deformations were expressed as a percentage of these and plotted against time on a logarithmic scale. Pore pressures were also recorded during some of the tests on the clay samples.

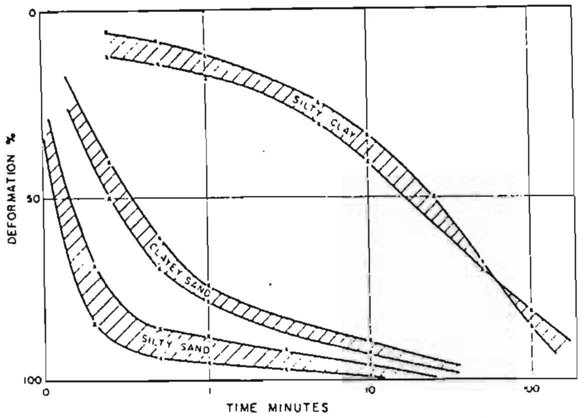

Fig. 2 Consolidometer deformation vs. time

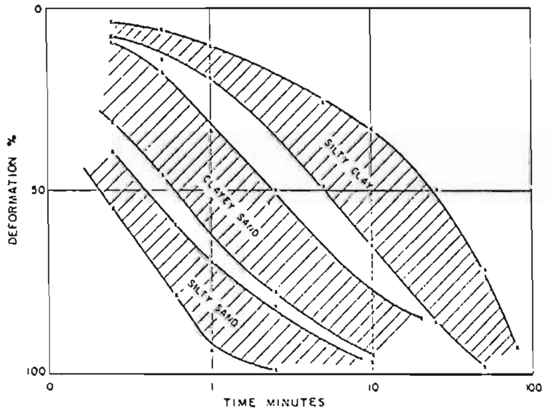

Fig. 3 Penetrometer deformation vs. time

DISCUSSION

The consolidation tests results for each material type see Fig.2, show fairly narrow bands whereas the penetrometer results in Fig. 3 show much wider spreads. The pore pressures rose immediately on application of a load after which there was a gradual decay apparently consistent with the decreasing rate of penetration. However, as is well known, such pore pressure measurements can give rise to problems of interpretation (Holden,1974; Schmertmann,1974) although many have advocated their use (Hansbo,1974; Janbu and Senneset, 1974; Ladanyi,1976). A comparison of Figs. 2 and 3 shows that consolidometer and penetrometer test times tend to agree although there are clearly anomalies which require explanation. In order to use the proposed penetrometer test with any confidence it will be necessary to resolve these anomalies and to collect considerably more data. It was observed that in all the materials the shape of the deformation time plot could be significantly altered by changing the load. This was presumably because the materials failed at higher stresses with the result that consolidation effects were masked. In this preliminary series of tests the maximum deformation was not controlled by limiting the loads and it was arbitrarily decided to discard results where the deformation exceeded 10 per cent of the cone diameter. It was accepted that the laboratory tests indicated sufficient correlation between the consolidometer and penetrometer results to justify field testing. This was found to be straightforward if the uppermost inner sounding rod was fitted a load platform. A standard 10-ton Goudsche Machinefabriek probe was used which had been fitted with an electrical strain gauge load measuring system with a chart recording device (Jones,1975). Deformation measurements were taken with an LVDT connected to this system.

CONCLUSIONS

Constant stress penetrometer tests are considered to be feasible for the preliminary field estimation of the time-settlement characteristics of alluvial deposits. Just as conventional constant rate of penetration testing has required a great deal of field correlation so will constant stress tests require similar correlations with other time-dependent tests and field performance to prove their validity.

This paper is published by permission of the Director of the National Institute for Transport and Road Research. The author also wishes to thank Messrs van Loggerenberg and Vorster of the Institute for their assistance with the design and construction of the equipment and carrying out the tests.

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(iii) EMBANKMENTS ON SOFT ALLUVIUM SETTLEMENT AND STABILITY STUDY IN DURBAN

Sixth Regional Conference for Africa on SOIL MECHANICS & FOUNDATION ENGINEERING Durban, South Africa, September 1975

Embankments on soft alluvium–settlement and stability study at Durban

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A.L. MCQUEEN *Partner of A.A. Loudon and Partners, Durban*

SYNOPSIS A series of road embankments is to be constructed over an area of soft alluvial deposits. Preliminary investigations consisting of boreholes and deep soundings have shown that settlement and stability problems are likely. The proposed construction programme requires reliable predictions of the time-settlement characteristics and since conventional investigation cannot given these, a trial embankment was built. This was comprehensively instrumented with settlement sensors and piezometers. The measured settlement of the 6 m high embankment was 1,3 m which agreed well with a prediction made from field and laboratory test results. The time in which this settlement occurred however was only about 500 days compared with the predicted time of 100 years.

The trial embankment showed that the construction programme could be adhered to without recourse to expensive drainage measures. A contract has been awarded for earthworks only and the pavement layers will be constructed two years later.

Since the stability analyses have shown that the proposed embankments will be only marginally stable, permeable blankets and toe berms will be incorporated and construction will be monitored with piezometers.

RÉSUMÉ On a établi un projet de construction d'une série de remblais routiers sur des gisements alluvionnaires tendres. Une étude préalable, effectuée à l'aide des trous de forage et du sondage à grande profondeur, a mis en évidence la possibilité des problèmes de tassement et de stabilité. Le programme de construction exige des prédictions sûres pour des caractéristiques temps-tassement. Puisque les études classiques ne peuvent pas les fournir, on a construit un remblais à titre d'essai, muni de l'appareillage complet comprenant les senseurs de tassement et les piézomètres. La mesure d'un remblais de 6 m d'hauteur a montré un tassement de 1,3 m, qui s'accordait bien avec la prédiction basée sur les résultats des expériences de laboratoire et de chantier. Le tassement n'a pourtant duré que 500 jours environ, tandis que la prédiction estimait une centaine d'années.

Le comportement du remblais d'essai a démontré que le programme de construction peut être poursuivi sans que des mesures de drainage coûteuses soient nécessaires. On a conclu un contrat uniquement pour les travaux de terre, tandis que les couches du corps de chaussée ne seront posées qu'après deux ans.

Puisque l'analyse n'a démontré qu'une stabilité marginale des remblais projetés, on va incorporer des tapis perméables et des pieds de banquette, et de plus contrôler les travaux de construction à l'aide de piézomètres.

INTRODUCTION

This paper describes an investigation, with stability and settlement analyses, which was carried out for a series of embankments to be constructed over compressible alluvium. The embankments carry part of the Durban Outer Ring Road which is shown in Figure 1. This section of the road under consideration runs from the Umgeni River northwards for about 12 km; the embankment problems occur in the southern 6 km, (i.e. chg 650-chg 830 in 100 ft chains).

The route was located to minimize the expropriation of domestic property, with the result that some considerable geotechnical problems have arisen through following a winding valley in a faulted area. The valley is low-lying and subject to frequent flooding. The meandering of the river, the Umhlangane, has resulted in four separate crossings within a distance of about 4 km.

Even along the parts of the proposed road which are not problematic because of swamps or faulting, other geotechnical difficulties occur. Although not the subject of this paper, it is interesting to draw attention to one of these parts as a good example of planning coordinated with site investigation in its various phases. In one place the preliminary road line passed along the east side of a hill in a small side cut and by so doing avoided a swamp. However, the engineering geologist responsible for the soil engineering map had noted, from interpretation of the aerial photographs, that the locally well-known problem of east dipping Ecca shale was likely to be present. A simple site inspection of the small cuttings for domestic driveways confirmed a very marked dip practically at right angles to the road line. An assessment of the situation showed that it was preferable to realign the road through the swamp area rather than incur problems of stabilising even quite small cuttings.

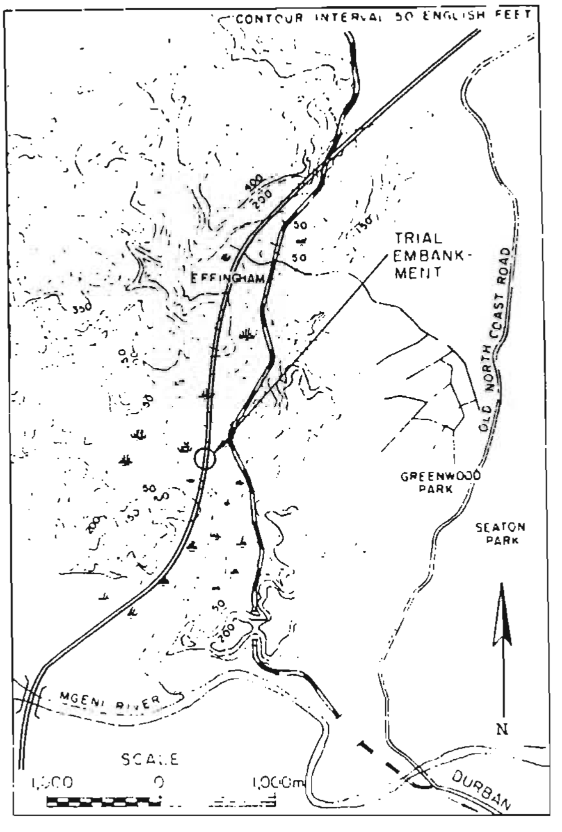

FIGURE 1

SITE PLAN

PRELIMINARY INVESTIGATION

A considerable amount of preliminary site investigation was carried out along the route. Apart from the soil engineering mapping, this consisted of sinking a number of boreholes and carrying out standard penetration tests (SPT), and later a series of deep soundings. Table 1 gives an indication of the poor subsoil conditions by listing the SPT values against depth for typical boreholes, which are numbered by their chainages along the centre line.

As a follow-up to the boreholes with SPT's. 35 soundings were carried out in the swamp areas at intervals along the centre line. During the course of this work, in-situ vane shear tests were performed so that a correlation, for the site, of the undrained vane shear strength, c_u, with the deep sounding cone pressure, q_c, could be made. This is discussed elsewhere Jones (1975). Figure 2 shows the resultant relationship, together with a typical borehole log. Factors of safety for circular arc failures could then be evaluated using assumed fill parameters and subsoil strengths, estimated from deep soundings. The Krugmann and Krizek (1973) charts and Pilot and Moreau (1973) charts were used. Since the shear strength of the fill material was assumed, the estimates could only be considered as a useful preliminary assessment.

The settlement of the proposed embankments was estimated from the sounding results by using the Buisman-de Beer relationship (de Beer and Martens (1957) in conjunction with the Terzaghi consolidation equation:

$$\delta H = 2{,}3 \times \frac{H}{C} \log_{10} \left(\frac{P_o + \sigma_z}{P_o} \right) \quad \text{-----------} \quad (1)$$

where δH = settlement of layer

H = thickness of layer

P_o = overburden pressure at mid depth of layer

σ_z = increase in stress at mid depth of layer due to load

C = compression modulus

and $$1/C = \frac{P_o}{1{,}5\, q_c} \quad \text{---------------------------} \quad (2)$$

where q_c = deep sounding cone pressure.

TABLE 1

SPT AGAINST DEPTH

Depth m	Borehole No. Chainage (100 feet)			
	666	682	688	736
1,5	1	0	0	0
3,0	7	0	5	0
4,5	0	7	8	0
6,0	5	47	7	0
7,5	59		0	8
9,0	65		1	28
10,5			4	
12,0			1	
13,5			3	
15,0			36	

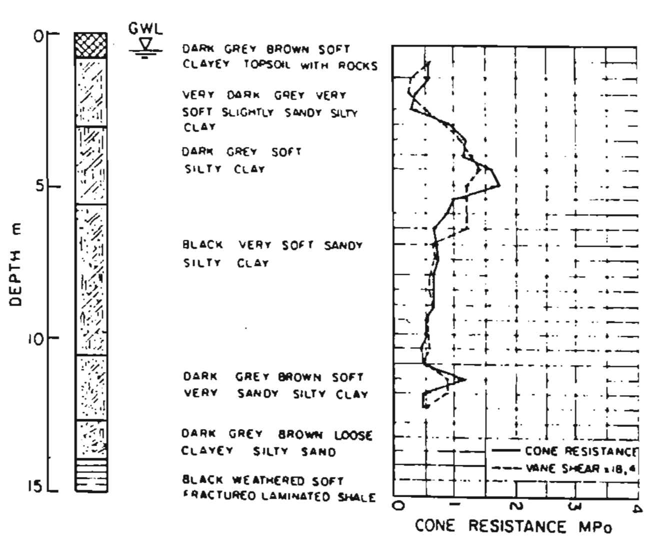

FIGURE 2

TYPICAL BOREHOLE AND SOUNDING LOGS SHOWING VANE SHEAR-CONE RESISTANCE COMPARISON

Charts were drawn up to show the predicted settlement of an embankment for various depths of compressible subsoil, heights of embankment and cone pressures. Figure 3 illustrates two of the charts. The above calculations take no account of the variability of the subsoil but nevertheless provide a guide for estimating purposes.

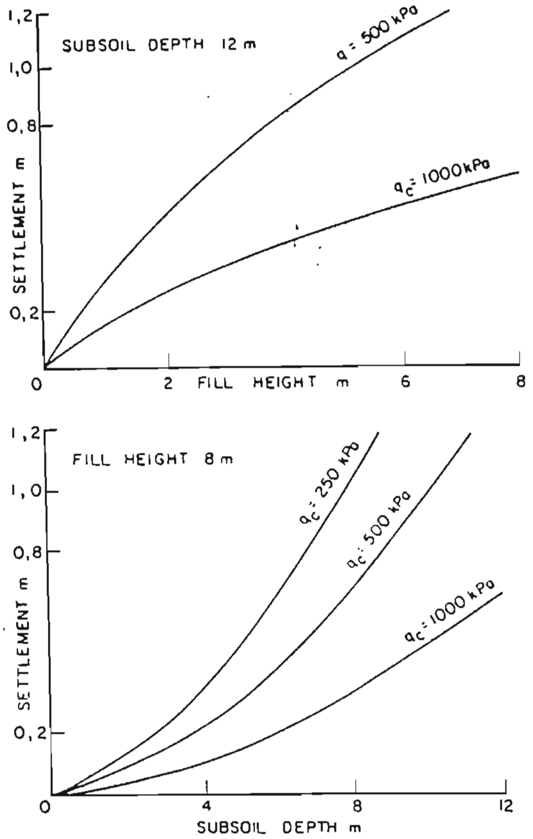

FIGURE 3

CONE RESISTANCE, FILL HEIGHT, SUBSOIL DEPTH AND SETTLEMENT RELATIONSHIP

Various modifications to the above simple equation have been proposed by a number of investigators and are mentioned elsewhere (Jones 1975).

The results of the preliminary investigation clearly demonstrated that stability and settlement problems were to be expected. The time-settlement characteristics could only be assessed from a description of the boreholes samples. Since these were predominantly medium to dark grey, slightly sandy, silty clay it was anticipated from local experience that the large settlements predicted would take about 2 years to reach 80% of the ultimate settlement (see Figure 4). This may have been acceptable for a continuous embankment, but since there were to be numerous bridges, problems of differential settlement, and possibly negative skin friction effects on the bridge piles, were important. The overall construction programme was therefore altered so that along this section of the route an early contract, comprising earthworks only, was let.

Less time was then available for the extended subsoil investigation than had originally been estimated, so that it became necessary to begin this second phase immediately. For this reason, and, also because the problem extended over a series of embankments totalling about 1,5 km in length, it was not possible to carry out a detailed investigation for the whole site.

It was therefore decided that a localised area should be chosen for the detailed investigation and the results extrapolated to the remainder using deep sounding values and basic subsoil parameters taken from laboratory tests of samples obtained from an auger survey.

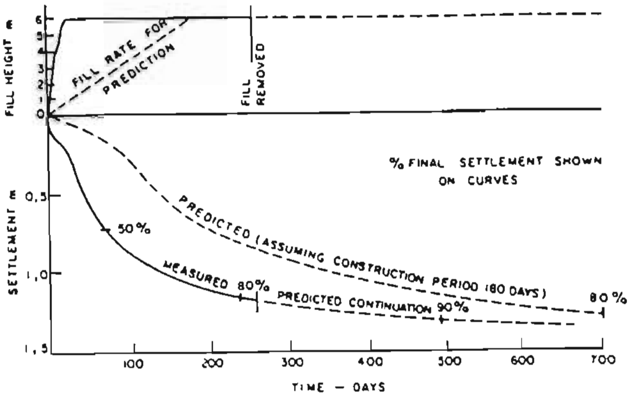

FIGURE 4

PREDICTED AND MEASURED SETTLEMENTS OF TRIAL EMBANKMENT

DETAILED INVESTIGATION

Equipment and expertise were not available to carry out the highly sophisticated sampling and testing procedures advocated by Rowe (1971) involving the use of large diameter thin-walled piston samplers. Of necessity, conventional U 102 sampling (undisturbed 102 mm diameter open drive tubes) formed the basis of the investigation procedure.

It is now widely accepted that estimates of the time-settlement characteristics of recent alluvial deposits from conventional consolidometer testing are subject to very large errors. Closer estimates are possible from Rowe Cell consolidometer testing, which permits horizontal drainage of the samples (Rowe and Barden 1966), and also from the results of in-situ permeability tests. However, in view of the proposed overall construction programme, even these methods were not thought to be accurate enough for the prediction necessary at the site. It was therefore decided that a full-scale trial embankment would be the most reliable method of obtaining the required data.

Description of trial embankment site

The site for the trial embankment was chosen, from the preliminary investigations by boreholes and soundings, as one of the areas with the poorest subsoil conditions. In addition it was very close to a proposed large cutting which was intended to supply much of the embankment material. The position of the site is indicated in Figure 1.

Although the trial embankment was to be constructed rapidly without any density control and would therefore not be included later in the permanent works, it was thought worthwhile to build it on the proposed centre line. In this way one of the worst areas would have been subjected to considerable preloading. Figure 5 shows a cross-section of the 30 m by 20 m by 6 m high trial embankment. The low natural ground-level should be noted: it was responsible for the frequent flooding and the consequent delay in installing the instrumentation.

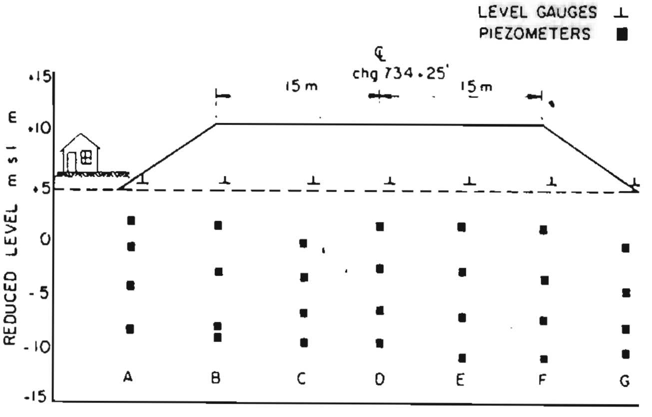

FIGURE 5

CROSS SECTION THROUGH TRIAL EMBANKMENT AND SUBSOIL SHOWING PIEZOMETERS AND SETTLEMENT SENSORS

Boreholes, with U102 sampling, and deep soundings were put down at each corner of the embankment. The boreholes were marked NE, SE, SW and NW. A typical borehole log is shown in Figure 2. Natural moisture contents, plasticity data and the results of particle size analyses are given in Table 2.

TABLE 2

Depth m	Natural m.c. %	Liquid limit %	Plastic Limit %	Clay % <2µ	Silt 60µ>%>2µ
2	57	56	35	52	35
4	50	66	24	45	42
6	93	70	27	67	30
8	91	63	35	60	32
10	72	69	32	45	41
12	57	60	30	49	48
14	28	30	15	29	32

Instrumentation of trial embankment

As shown in Figure 5, 28 pneumatic piezometers and 7 settlement indicators were installed in seven vertical profiles down to a depth of about 15 m.

The piezometers were installed at vertical intervals of approximately 3 m in 150 mm-diameter boreholes. Each instrument was placed in the centre of a column of sand about 2 m high and each column was separated from the sand above and below by layers of the local clay and rammed bentonite balls. Since four piezometers were placed in each borehole, considerable practical problems were encountered during the installation and sealing of the upper ones because of interference from the read-out tubes. However all but one of the piezometers gave readings commensurate with the installation depths.

Both the settlement gauges and the piezometers were commercially available: Terra Technology Settlement Sensor Model S-6010C and Terra Technology Pneumatic Piezometer Model P1020. The read-out for the instruments was obtained by measuring the air pressure required to operate a check valve in the instrument. In the case of the piezometer the valve balanced the applied air pressure against a diaphragm which sensed the pore pressure through a filter-element. The settlement indicator operated by balancing the applied pneumatic pressure against a head of mercury in a closed tube leading from the indicator to a resevoir in the gauge house. The pneumatic pressure was applied by releasing air from a portable, compressed-air bottle carried in a case also containing the necessary pressure gauges, (control Unit Model C-6300). Connection was made to each instrument in turn by quick-connect adaptors.

All the tubes were led to a gauge house as shown in Figure 5. The gauge house level was in turn related to a fixed bench mark installed on a nearby hillside. A feature of the instrumentation was the ease and rapidity with which measurements could be taken. It was found that with only a few minute's practice the reading of one instrument could be accomplished in as little as 1 minute, including the connecting-up time. This point is stressed because the cost of instrumentation is sometimes erroneously taken as being simply the cost of the instruments plus that of installation. Actual costing, which should include the reading time, frequently shows that the latter is the most expensive item, particularly for a long-term project.

An additional measuring system was installed at a later stage : this consisted of a large number of accurately surveyed pegs in rows parallel to the toe on three sides of the embankment. The purpose of these pegs was to measure horizontal and vertical movement at natural ground-level adjacent to the toe since at one time after the fill had been completed it was thought that the rate of settlement was increasing. It was surmised after much puzzlement that some significant deformation may have been the cause, but it subsequently transpired that, for a few of the readings, the settlement of the gauge house itself had been omitted from the calculations, thus significantly altering the shape of part of the time-settlement plot! Although very large overall fill settlement was measured, no significant horizontal movement of the pegs was detected. However, they were only monitored for a month before the temporary omission of the gauge house settlement was rectified, so that no conclusion can be drawn. In retrospect it seems unfortunate that more use was not made of this simple measuring system throughout the whole of the trial embankment period.

A further point to be borne in mind with regard to instrumentation schemes is the necessity for making them vandal-proof. Despite the fact that the read-out pipes were contained in ducts leading into the gauge house (a locked shed which was guarded) thieves managed to dig up the ducts practically at ground-level where they entered the shed, and removed a length of the plastic tubing from each instrument. It was not possible to rejoin the mercury-filled leads to the settlement gauges. Fortunately, by adding further lengths of tube to the piezometer leads and continuing with the readings, it was possible to compare before and after readings and thus deduce which tubes belonged to which piezometer.

Settlement readings were continued by placing pegs on top of the completed fill and levelling these in the conventional way with reference to the fixed bench mark.

Typical readings from these instruments are shown in Figures 4 and 6.

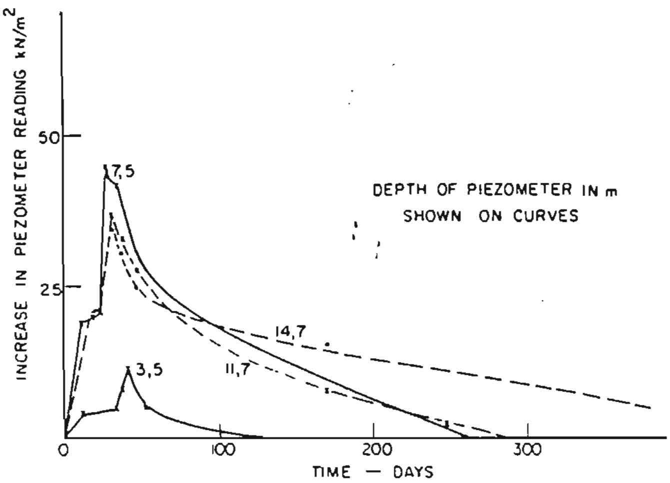

FIGURE 6

TYPICAL PIEZOMETER READINGS

Laboratory Testing

Samples were obtained from the four corner boreholes of the trial embankment and from trial pits. An auger survey was carried out along the centre line of the proposed road to obtain samples for comparative testing for the other embankment positions.

Laboratory tests consisted of quick, undrained triaxial tests, drained triaxial tests, shear box tests, conventional consolidometer tests, indicator tests and hydrometer analyses.

Tables 3, 4 and 5 summarize the results of the laboratory testing programme.

a) Quick, undrained triaxial tests. (ϕ = 0)

TABLE 3

BH No	Depth m	c_u kPa
SW	4	21,7
SW	8	16,1
SW	12,3	7,0
NW	4	26,6
NW	8	18,2

b) Drained triaxial (T) and shear box (S) tests

TABLE 4

Depth m	Test type	c' kPa	ϕ'
6	T	0	26
8,5	T	0	20
2	S	0	24
2	S	0	23,5
8,5	S	1	19,5
12	S	0	33

c) Consolidometer tests

Ten conventional consolidometer tests were carried out and the results are summarized in Table 5. Figure 7 gives the settlement vs log-time plot at 50 kPa for the sample from BH. No. NE at 4,0 m. This is typical of the results, as is the void ratio vs log-pressure plot given in Figure 8.

TABLE 5

BH No.	Depth m	M_v (m^2/kN)	Preconsolidation Pressure kN/m^2	t_{50} (mins)	t_{90} (mins)	C_v (m^2/yr)
NE	2,0	0,00126		26	150	0,40
NE	4,0	0,00041	140	9	120	1,14
NE	4,0	0,00054	130	20	170	0,52
NE	6,0	0,00094		36	225	0,29
NE	8,5	0,00172	90	23	220	0,45
NE	8,5	0,00176	80	24	215	0,43
NE	11,0	0,00015		8	79	1,29
SE	2,0	0,00076		45	210	0,23
SE	6,0	0,00103		27	195	0,38
SE	10,0	0,00055		20	118	0,52

In the above table M_v is quoted for the 100-200 kPa pressure range and t_{50}, t_{90} and C_v for the 200 kPa pressure increment. The preconsolidation pressures are estimated from the e-log p curves using Casagrande's construction. While many of the test data are still to be analysed, the indications are that the subsoil is normally consolidated at depth but that from about 4 to 5 m there is a stratum of overconsolidated material. At this depth a marked increase in shear strength is consistently shown by both the soundings and the in-situ vane tests. It may also be noteworthy that the relationship between these, which is shown in Figure 2, indicates the largest divergence from the mean value of 18,4 in this stratum.

d) Indicator tests and particle size distribution.

Table 2, given earlier, summarizes typical data from the above tests for samples from Borehole No.SE. About 120 indicator tests were carried out on the samples from the auger survey and plotted on a long section so that the poorest subsoil areas could be readily detected and compared with the trial embankment site.

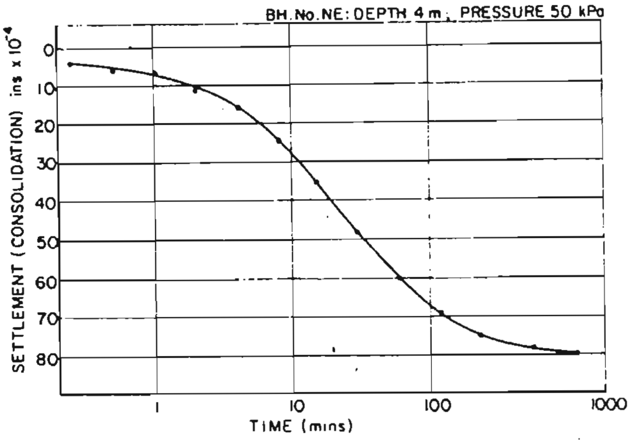

FIGURE 7

TYPICAL CONSOLIDOMETER SETTLEMENT-LOG TIME PLOT

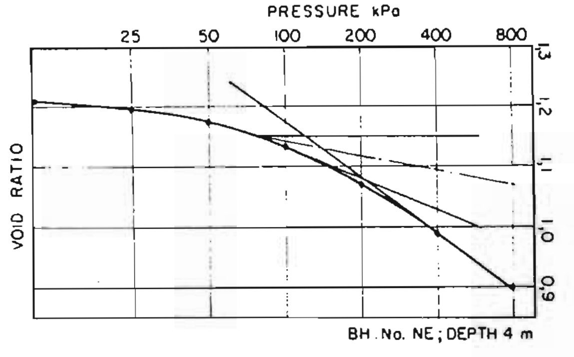

FIGURE 8

TYPICAL CONSOLIDOMETER VOID RATIO-LOG PRESSURE PLOT SHOWING DETERMINATION OF PRECONSOLIDATION PRESSURE BY CASAGRANDE'S CONSTRUCTION

CALCULATION OF SETTLEMENT

Predictions of the settlement of the trial embankment were made using the calculation method based on deep sounding cone pressures - equation (1). Four soundings were made at the proposed corners of the embankment to obtain cone pressures and friction ratios. Calculations, modified by a knowledge of the nature of the subsoil, gave the following predicted settlement under the centre:

$\delta H = 1{,}1 \pm 0{,}3$ m

The limits in the above calculated settlement reflect the variations in the four sounding readings.
The unmodified method gave the following result:

$\delta H = 2{,}1 \pm 0{,}5$ m

Another modification, proposed by Meyerhof (1965) involves the alteration of the 1,5 in equation (2) to 1,9. This alters the predicted settlement to:

$\delta H = 1{,}6 \pm 0{,}4$ m

Various other methods of calculating settlement from sounding results have been proposed particularly for clay subsoils as at this site.

For instance, Skempton (1951) has suggested the following relationship for normally consolidated clay:

$$\frac{1}{M_v} = (25 \text{ to } 80)\ c_u \quad \text{-------------------} \quad (3)$$

If the site relationship between c_u and q_c derived from the vane shear comparison with soundings is used, i.e. $c_u = \dfrac{q_c}{18{,}4}$

$$\text{then } M_v = \frac{1}{(1{,}36 \text{ to } 4{,}35)q_c} \ \text{m}^2/\text{kN} \quad \text{-----------} \quad (4)$$

Settlements can then be calculated by one dimensional consolidation theory and this gives a mean value of:

$\delta H = 1{,}6$ m

The coefficients of volume compressibility, M_v, were also obtained from the laboratory consolidometer tests and settlement calculated from these values gave the following result:

$\delta H = 1{,}3$ m.

The overall mean predicted settlement using all the calculations methods may be given as:

$\delta H = 1{,}6 \pm 0{,}4$ m.

Calculation of time for settlement

Using the classic one dimensional consolidation theory and the laboratory consolidometer coefficients of consolidation from Table 4, estimates of the time it takes for 90% of the total settlement can be made. If it is assumed that drainage takes place vertically to the top and bottom then T_{90} varies between about 50 years and 150 years !

An alternative method of estimating the end of consolidation time is by assuming that consolidation ceases when the field piezometers record zero excess pore pressure compared with their original readings. Some of these readings are shown in Figure 6. The mean for all the readings gives a time of about 500 days. It can be seen that if the actual time settlement plot in Figure 4 is projected, then settlement would level out after about the 500th day at a total settlement of about 1,3 m.

Only a superficial attempt was made to predict the time-settlement relationship for the trial embankment from the laboratory C_v values. The purpose of the trial embankment was to enable the results of a full-scale field measurement to be used to predict the time settlement for the main embankments.
The need for reliable settlement predictions from simple field testing is apparent since it not often feasible to carry out large-scale field tests.

STABILITY ANALYSES

Numerous stability analyses were carried out at various stages of the investigation. For the first analyses it was necessary to assume the parameters to be those determined from sounding readings, but for later analyses laboratory-measured values for both the subsoil and the proposed fill material became available.

Calculations were carried out using the National Institute for Road Research's SLOP 2 and SLOP 3 programs (Szendrei and Pells, 1971). Program SLOP 2 is based on Bishop's approximate method of slices for circular slip surfaces and SLOP 3 on the Morgenstern and Price method of slices for general slip surfaces. For the subsoil conditions at this site only very small differences in the factors of safety were obtained through the two computer programs and therefore the simpler SLOP 2 program was used more extensively.

The preliminary calculations showed that the 8 m-high embankment which was to be constructed at the trial embankment site would probably be unstable. Table 6 gives results of typical stability analyses using the laboratory-measured subsoil parameters of $c' = 0$ kPa and $\phi' = 20°$.

TABLE 6

Water Table m (zero at ground) level	Fill Parameters c' kPa	ϕ'	Factor of Safety
0	10	20	0,78
-1,0	10	20	0,86
0	10	25	0,93
0	10	30	1,04

A stability analysis, of conditions immediately after construction, was made using quick, undrained parameters ($\phi = 0$) for the subsoil. Cohesion values used in this analysis were deduced from the laboratory and in-situ vane and deep sounding tests. Typically for the upper 3 m, $c_u = 20$ kPa and below that depth constant $c_u = 30$ kPa. These parameters gave somewhat higher factors of safety than the long-term effective stress analyses, but if some allowance is made for the effects of probable anisotropy and the high plasticity of the subsoil, then the factors are comparable and give cause for concern.

Since flooding of the area is probable, a check was carried out for the change in factor of safety with increasing pore pressure. It was found that, for a typical set of parameters, an increase in pore pressure of 0,3 m head of water resulted in a decrease in the factor of safety of about 0,1. Since flooding to a depth of up to 2 m could be expected, the resulting decrease in the factor of safety could be highly significant.

It was concluded that a permeable blanket was required under the embankment. It was decided to construct the blanket of sandstone since this would be available from a nearby cutting.

Further analyses were made after shear box testing of fill material (Ecca shale from an adjacent cutting) had been carried out. These gave parameters of $c' = 5$ kPa; $\phi' = 38°$.

Various combinations of fill slope, rock blanket (permeable layer) thickness, and berm widths and heights were tried.

Considerable discussion arose regarding the parameters to be ascribed to the rock blanket since the stress-strain characteristics of the rock and of the underlying soft clay were clearly very different. It was eventually decided to take the view that for the purpose of analysis, the rock should have the same parameters as the selected fill, since the function of the blanket was not to provide extra strength but only to control pore pressure. Table 7 summarizes the results of the analyses and Figure 9 illustrates Case 15A which is considered to be most representative of the site conditions. All the cases in Table 7 assume that the water table is at ground-level and there is no excess pore pressure

TABLE 7

SUMMARY OF STABILITY ANALYSES

CASE No.	SLOPE GEOMETRY	SOIL PARAMETERS	c (kPa)	ϕ	MINIMUM F
1	$1:1^1/2$. No berm One layer subsoil	fill subsoil	10 0	20 20	1,06
3	$1:1^1/2$. No berm or rock Two layer subsoil, (Toplayer 3 m thick)	fill subsoil top : lower :	10 20 30	20 0 0	1,13
5	$1:1^1/2$.Berm 10x2 m	as Case 1			1,41
9	$1:1^1/2$.Berm 10x2 m	fill subsoil	10 0	25 20	1,48
14A	$1:1^1/2$. No berm	fill subsoil	0 0	38 20	1,20
15A	$1:1^1/2$.Berm 10x2 m	as Case 14A			

There are many observations possible from examination of Table 7 but probably the most important of these is the enormous difference made to the factor of safety by the addition of a berm of about $^1/3$ of embankment height. i.e. compare Case 1 with Case 5 or Case 14A with 15A where the factor increases by about 0,4. This is not altogether unexpected and is apparent from the Pilot and Moreau charts referred to earlier.

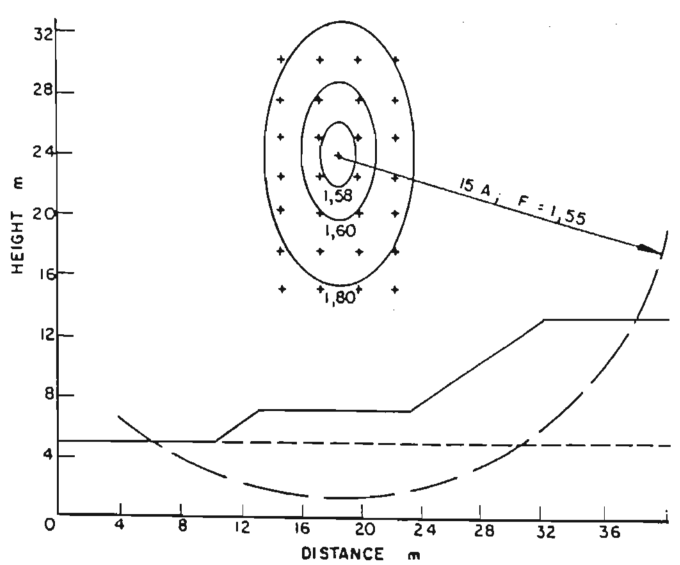

FIGURE 9

CROSS SECTION THROUGH PROPOSED EMBANKMENT AND BERM SHOWING CONTOURS OF FACTOR OF SAFETY

Many combinations of fill and subsoil strengths and geometry were analysed and in practically all cases the factor of safety was greater than unity. At this site it is fortuitous that the relatively inexpensive incorporation of berms and a drainage blanket result in a fairly high factor. It is however worthwhile speculating what would have been considered as a sufficiently high factor of safety had a cheap solution not been possible.

The piezometers installed under the trial embankment showed pore water pressure increases of up to 5 m head of water at a depth of about 8 m - see Figure 6. At the depth through which the critical slip circles pass, i.e. about 4 m, the excess pore pressure developed under the trial embankment was only about 1 m. Nevertheless, since as already mentioned, this implies a reduction in the factor of safety of about 0,3, it is of considerable concern. It was therefore decided that construction control measures were necessary and arrangements were made for piezometers to be installed under all the embankments. It would be possible to draw up control charts relating embankment height, excess pore pressures at various depths, and the factors of safety for circles passing through these depths. However, on considering the extent of the proposed embankments and the data which would have to be obtained to make the charts representative for all the different subsoil conditions, it was decided that a more pragmatic approach was called for. This is summarized in Table 8. If the indicated pore pressures are reached, construction will stop to allow time for the pressures to dissipate.

TABLE 8

Embankment Height. M	8	6	4
Max. allowable excess pore pressure. m of water	1,5	2,5	3,5

Settlement indicators will also be installed. These will be used to control construction by monitoring rates of settlement and will also serve as checks on the settlement predictions based on deep soundings.

ACKNOWLEDGEMENTS

The investigation has taken place in stages over a considerable period of time and during that time many people have been responsible for various aspects of the work. The authors would like to draw attention to the most valuable cooperation they have received from the authorities and consultants involved. The client for the road project is the Department of Transport, working through the agency of the Natal Provincial Administration. The materials engineers of these authorities, Messrs. M.F. Mitchell and P.J. Groth respectively, are particularly to be thanked for their interest and help as are the Consulting Engineering firms of Hawkins, Hawkins and Osborn and A.A. Loudon and Partners. Thanks are also due to Professor K Knight of Natal University who carried out much of the laboratory testing and stability analyses, and to Mr. A. McG. Robertson who supplied the instrumentation and gave helpful advice on its installation.

The paper is published with the permission of he Secretary of Transport, the Director of Roads of the Natal Provincial Administration and the Director of the National Institute for Road Research.

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A.P. TYRRELL

Whilst I wish to comment in detail on the paper by Mr. D.M. Clark, I should like to take this opportunity to congratulate Mr. A.A.B. Williams and Mr. G.A. Jones and their co-authors on the excellent presentation of their papers and, in particular, for providing a very full record of each of the case histories reported.

In comparison, Mr. Clark's paper lacks essential detail, and I wish to request that much fuller information relating to the case histories be provided in order that an assessment of his engineering solutions may be made, and the necessity for sand drains can be judged independently.

Specific points that I wish to make are as follows:

1. Detailed soil profiles would give a much clearer picture of the foundation conditions encountered. The closest that the author comes to fulfilling this requirement is to state that for the surface clay layer at the Langfontein site 'it would appear that there are sand lenses interbedded with the clay'. I feel sure that for a high embankment of this type, a very comprehensive investigation must have been made, and a detailed description of soil fabric (e.g. silt veins and layers, rootlets, etc.) would be informative. In this connection, close-up colour photographs of soil samples split and left partially to dry can be invaluable, and I would commend this technique strongly.

2. The details of the methods of prediction of both amounts and rates of settlement would be most interesting, together with details of the laboratory testing programme. Unfortunately, little can be gleaned from a comparison of the actual settlements recorded with the predictions as presented in the paper. For the Confidence embankment, for instance, the following figures are quoted:

   measured settlement to date = 0,2 m
   predicted settlement = 1,1 m

   Readers of the paper are therefore faced with two possibilities: firstly, that the sand drains acted as piles and were therefore a dominant influence on the actual amount of settlement; or, secondly, that the prediction of settlement was not realistic. If the latter is the case, it is possible that samples for consolidation testing were too small to be representative of the mass, or too disturbed as a result of sampling technique. As far as the prediction of rates of settlement are concerned, it is on these that the recommendations for sand drains must have been based, and it would be interesting to know what testing techniques (e.g. drainage direction) were specified in relation to the identified geological details, or fabric, of the foundation clays (Rowe, 1968). Very often, in the past, an appreciation of the relevance of the coefficients of consolidation with drainage in a vertical direction or in the horizontal direction has been lacking and a difference of several orders of magnitude is not unknown (Tyrrell, 1969).

3. Finally, it may be noted that in the author's concluding discussion, the highly variable nature of the foundation soils was appreciated. It would therefore be interesting to learn whether, for these high embankments, any consideration was given to the construction of a trial embankment in order to aid the assessment of field performance prior to final construction.

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PANEL DISCUSSION

Prof. De Mello agreed that photographs of the soil profile can be important in assessing the effect of horizontal drainage.

DISCUSSION ON PAPER - EMBANKMENTS ON SOFT ALLUVIUM - SETTLEMENT AND STABILITY STUDY AT DURBAN BY G.A. JONES, D.F. LEVOY AND A.L. McQUEEN

G.A. JONES

It is proposed that this discussion will be under two headings. The first will be settlements and to a certain extent this includes reference to the paper on deep sounding in session 3. The second section will be concerned with the evident instability of the embankment which was seen during the site visit earlier in the week.

First the bad news - the settlement predictions. Table 1 gives some very recent results of actual and predicted settlements and again I would like to stress that the predicted values appeared in print before the embankments were constructed.

TABLE 1

Embankment	Predicted Settlement (m)	Measured Settlement (m)
Mtwalumi	0,7 $\pm$ 0,2	0,8
Uvusi	0,2 $\pm$ 0,05	0,18
Sea Cow Lake Trial	1,1 $\pm$ 0,3	1,3
Sea Cow Lake Actual	1,4 $\pm$ 0,3	0,9ƒ

ƒ This embankment is still settling

$$\text{Mean Ratio } \frac{\text{Predicted}}{\text{Measured}} = 0{,}95$$

There are a number of other embankments which are being monitored but unfortunately the results are not yet available.

I should first say that there is an error in the paper on embankments on soft alluvium. On page 244, equation 2 has become inverted and should read:-

$$C = \frac{1,5q_c}{P_o}$$

Some questions were raised by Mr. Schwartz and Dr. Tyrrell and by members of the panel. I would like to answer these now.

Editors Note: These questions were raised in discussion during session No. 3 on Piled foundations.

In the paper on sounding there are a couple of points which may require a little clarification. The typical sounding log for the Mtwalumi site shown in Figure 3 indicates large variations in the friction ratio and the questioner asked if this did not show that it was a poor choice of parameter to attempt to measure. It should have been stressed that the sounding was carried out with the old style equipment and that readings of that nature - in a material which boreholes had shown to be a consistent silty sand -figured very largely amongst the reasons for deciding to build a more accurate load sensing device.

A further question concerned what value was used in the settlement prediction for the Sea Cow Lake trial embankment. The paper indicates a settlement of 1,6 ± 0,4 m. This was calculated using the method with no allowance for material type which is the same as using a constant value of 1,2. What should have been stated in addition, is that from further soundings with the modified equipment, reliable friction ratios were determined. From these, values were selected, and adjusted settlement calculations carried out which reduced the predicted settlement to 1,1 ± 0,3 m, the value given in the table above. This is a considerable adjustment and may go some way at least in explaining the discrepancies which are reported between predicted and measured settlements such as those given by Dr. Simons earlier.

Dr. Tyrrell took me to task for suggesting that time settlement characteristics could be estimated from soundings. I had intended that from the paragraph dealing with that aspect it should be fairly clear that it was not suggested that any fundamental parameter was being measured. I intended simply to say, that in broad terms, the more clayey the subsoil is, then the smaller is its permeability. As a result it is reasonable to suppose that the slower it will consolidate. All we intend to do, is to estimate the time settlement characteristics, on a classification basis, so that one may decide whether or not further investigation is required to examine these characteristics in more detail. I should perhaps stress my complete agreement with some members of the panel who have criticized over reliance on any one investigation technique. We very definitely see sounding as an excellent preliminary tool to give guidance on the necessity for further work and we would never use it as the sole means of investigation unless considerable information was already available on the subsoil conditions at that site.

I started off by calling this section the bad news - that was because the agreement between predicted and measured settlements is almost too good to be true and I shall be suspected of having concealed results which did not fit.

Anyway, now for the good news. We designed over 1,5 km of embankments in the Sea Cow Lake area and we have a dislocation of about 60 m length over half the width in one section, i.e. we have 98 per cent success.

That comment is not quite as facetious as it may seem since it must be remembered that as a result of the investigation work it had always been our intention to build the embankments early (2 years before originally contemplated) and to minimize the factor of safety; obviously not as low as we achieved in the one instance, but generally so that the cost of special construction techniques would be as low as possible.

As the dislocation has only very recently occurred we have not yet had time to fully investigate the cause. The nature of the dislocation is as illustrated in Figure 1. However, from the results of the laboratory tests carried out so far, it would appear that there are no significant changes in subsoil parameters from those originally measured. What then is the explanation for the failure?

We are having a problem deciding what the real mechanism of the failure was, because there are two complicating factors which may have to be taken into account. The first of these is that the somewhat incoherent eye witness report indicates that the first manifestation of failure was movement at what is now the large open vertical crack - our feeling is that this may be misleading and we suspect that first movement was a fairly conventional circular slip of blocks 1 and 2 as tentatively shown in the diagram, Fig. 1 and that block 3 subsequently moved. The second complication is that the trial embankment, which was removed to original ground level before construction of the main embankment, was in the position shown in the diagram, more or less at the apex of the failure. It will be remembered that the trial embankment settled about 1,3 m and what effect this might have had is not known although it is possible that it accounts in part for the large tilt of block 3.

Further stability analyses have now been carried out with some changed assumptions. The chief of these is that we take to its logical conclusion the idea expressed in the paper that some reduction in strength of embankment material should be made because of the difference in stress strain characteristics between it and the underlying soft clay. It is clear, with hindsight that the logical conclusion should be that no strength should be ascribed to the embankment if there is a possibility of full depth cracking. Using this assumption the factor of safety is reduced by about 0,2. The embankments were being monitored with settlement indicators and piezometers and limits had been given for maximum allowable pore pressures for different embankment heights. The piezometers showed that before the final lift of embankment construction, the pore pressure was in fact hovering about the limit. However all seemed well otherwise and because of the usual pressures on any construction site and the fact that the final lift was only to be half a metre and also because the

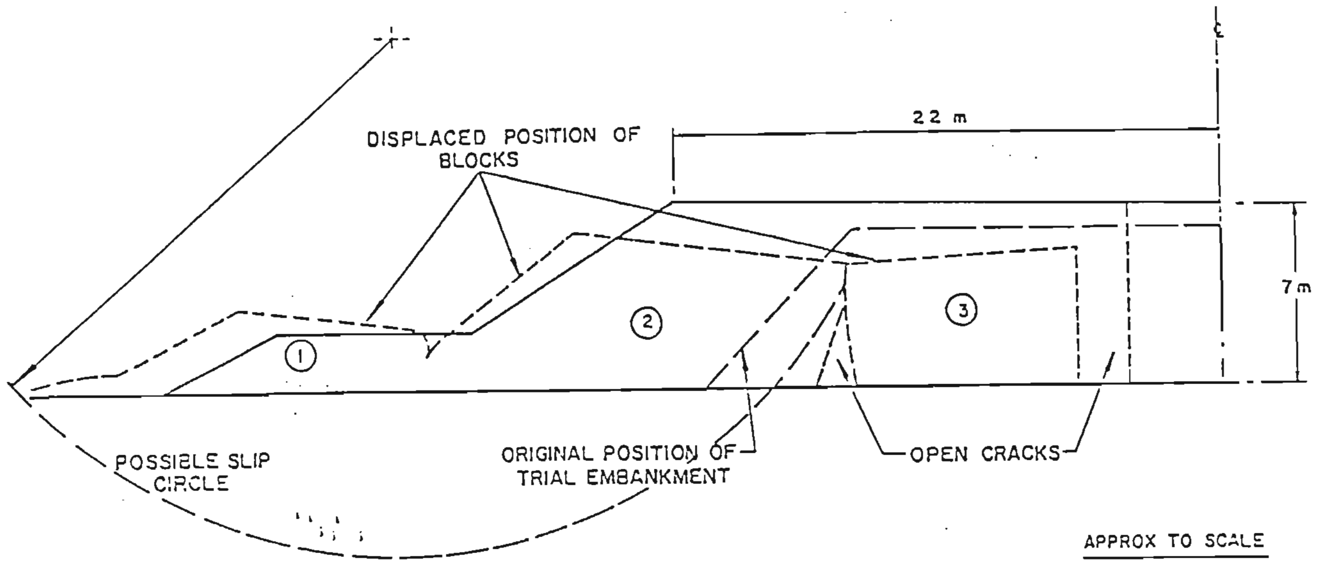

FIGURE 1

CROSS SECTION THROUGH EMBANKMENT BEFORE AND AFTER FAILURE

limit given was believed to allow some margin, it was decided to go ahead. This was done on a Friday with no untoward incident - on the following Monday the pore pressure was observed to have substantially increased to well over the pre-set limit and early on Tuesday morning the drama occured taking about two hours to reach the more or less steady deformed shape.

We have, at least for the time being, concluded that two wrong assumptions were made. Firstly the analysis using fill strength was incorrect, since it was well known that large deformations were inevitable. Secondly the limits set for allowable pore pressures during construction, which in themselves may have been correct, were not expressed in the best way for site control purposes. We now believe that there should have been a properly defined danger zone before the limit was reached, with a plan of action stating what should have been done whilst operating in that zone. It would perhaps have been better to have kept a check on rise in pore pressure expressed as a fraction of rise in embankment height.

The action taken immediately after the excessive movement was to fill the large vertical crack with sand to prevent the good side of the embankment from falling into the crack. After a rapid analysis it was decided to double the width and height of the berm - this being comparatively easy to do, since the fill came from a cut being worked only a couple of hundred metres away. In order to get more information about the cause of the problem a transverse trench was excavated which very clearly showed the size of the cracks in the base of the embankment - these are shown more or less to scale in the diagram.

We have not as yet decided what long term repairs should be carried out, but it is probable that, because of the shape of the cracks, there will be great difficulty in doing anything other than total reconstruction. At present simple cased standpipes - resulting from the sampling of laboratory testing - are being monitored. These show that fairly high pore pressures still exist therefore some time is being allowed for these to dissipate before repair work is commenced.

What is now of great interest to us is what criteria exist for deciding whether or not one should use the fill strength in a calculation of factor of safety in a stability analysis?

PANEL DISCUSSION

PROF. NASH

Referred to an embankment constructed over a swamp which failed. The embankment consisted of fly ash which when compacted was brittle. A failure occurred and it was deduced that no strength could be attributed to the stiff embankment on a soft subsoil.

PROF. DE MELLO

Commented that the stress-strain curves of the subsoil and the embankment should be compared. If they were very different, suitable adjustments should be made to the stability calculations. He referred to his paper in the 1972 Hong Kong conference dealing with the theory of settlements in Saprolites. This paper describes the effect of extreme heterogeneity of the subsoil.

MR KANTEY

Mentioned failure of an oil tank on soft subsoil in Beira. The thickness of the sand overlying the soft subsoil affected the stability.

PROF. SAVAGE

Commented that more can be learnt from a well documented failure than from many successful jobs.

AUTHOR'S REPLY

The question put to the panel resulted in no real response, presumably because it is as much an unfamiliar problem to them as it is to us.

We would suggest that by simulating the building of an embankment by a finite element technique one could arrive at tension zones in the base of the embankment the severity of which would depend largely on the ratio of the E values of embankment material and subsoil. It seems likely that this would be a fairly insensitive analysis but it may well be sufficient to enable one to say that embankments fall into three categories:-

(a) E values of same order - assume complete fill strength.

(b) E values intermediate ratio - futher investigation and check stability analysis with and without fill strength.

(c) E values out of balance - assume no fill strength.

We would be interested to have comment on this and propose carrying out a study, first of the literature and then by a desk study, to check the feasibility of establishing some fairly simple criteria.

I would like to add a brief comment on Mr. Clark's paper, particularly interesting to me since I had the opportunity of visiting the most impressive embankments which he describes. Reading between the lines, one assumes that it was difficult to decide on realistic time-settlement characteristics; the result was that sand drains were resorted to. We had the same problem at Sea Cow Lake but overcame it by building the trial embankment. This settled about 100 times faster than calculated from simple consolidometer tests and showed, as often seems to be the case, that no expensive drainage installation was necessary.

A further comment on this is that the paper by Messrs. Williams, Andrew and Glenday gives some very useful criteria for assessing the need for sand drains. Their paper also describes the underdrainage measures taken and having seen these I would say that they are a beautiful example of meticulously designed and constructed drainage. I would like to ask the authors to consider giving more publicity to this aspect of the project.

A.D.W. SPARKS

A visual method for stability analysis

The advent of computer programs for determining the factors of safety of slopes or foundations has not dimished the importance of simple hand methods for calculating these factors of safety.

Incorrect input data, incorrect usage and the tendency of computer personnel to modify programs can sometimes lead to costly errors in the results from programs which are normally reliable. Alternative simple methods for checking the magnitude of computer results are therefore necessary.

I have used a simple visual method for estimating the order of magnitude of factors of safety for slopes, foundations and wall systems. The method is an extension of previous calculation methods described in reference 1. This visual method has been used for five years for setting examination and tutorial questions. It has provided results which are within ten per cent of the final calculated factors of safety if judicious care is taken when estimating the values used in the method.

Only one aspect of this method is presented in Figure 1. Because it is sometimes difficult to estimate the average total pressure P_{av} normal to the slip surface (Step 6 in Fig.1), a better variation of the method has been presented in point 14 of this discussion.

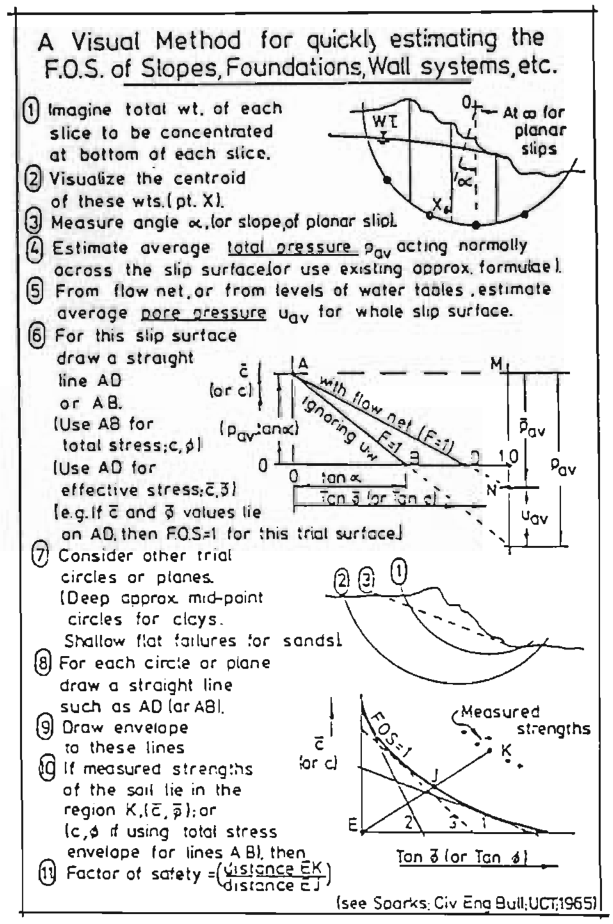

FIGURE 1

A VISUAL METHOD FOR QUICKLY ESTIMATING THE F.O.S OF SLOPES, FOUNDATIONS, WALL SYSTEMS, ETC. (NO EXTERNAL WATER PRESSURE ON SLOPE)

I would like to make the following points about this method:

(1) I have proved by mathematics and by calculated examples that the method in Figure 1 is identical to the simple method of slices if correct values are used for the average pressures shown in Figure 1.

(2) The method applies to planar or circular slip surfaces.

(3) It is a very quick method.

(4) It is possible to consider the possibility of failure along many different trial slip circles and slip planes, and to discard certain slip surfaces which will never become critical.

(iv) DESIGN AND MONITORING OF AN EMBANKMENT ON ALLUVIUM

Seventh Regional Conference for Africa on Soil Mechanics and Foundation Engineering / Accra / June 1980

Design and monitoring of an embankment on alluvium

G.A. JONES, E. RUST & H.J. TLUCZEK
Van Niekerk, Kleyn & Edwards, Silverton, South Africa

SYNOPSIS. The construction of a freeway along the Natal Coast necessitated an embankment about 500 m long and 5 m high, crossing the Umzimbazi River. The subsoil consisted of soft clays and loose, silty sands to a depth of 13 m which were expected to result in significant settlement and stability problems. Anticipated differential settlement between the embankment and the proposed river bridge posed problems of negative skin friction on the piles. The investigation and analysis therefore gave considerable emphasis to establishing the time - settlement characteristics of the subsoil and included in-situ permeability testing, together with 150 mm Rowe Cell consolidation tests. These showed that the permeability was significantly higher than measured in small diameter testing and that expensive subsoil treatment was not required. Extensive monitoring was carried out to assess the stability and maximize the rate of construction. The monitoring has shown that the embankment has performed as predicted.

RÉSUMÉ. La construction d'une autoroute le long de la côte de Natal a nécessité le franchissement du fleuve de l'Umzimbazi par le moyen d'un pont et d'un remblai de 500m. de longeur et 5m. de hauteur.

Le sous-sol se composait d'argiles molles et de sables limoneux sans consistance jusqu'à un profondeur de 13m. On aurait pu donc attendre des problèmes sensibles de tassement et de stabilité du remblai. Le tassement différentiel qu'on avait prévu entre le remblai et le pont du fleuve a posé le problème du frottement inverse du sol sur la surface des pieux.

Au cours le l'enquête et de l'analyse du sous-sol on a donc mis en relief la constatation des caractéristiques temps-tassement du sous-sol. Cette enquête comprenait aussi tant des essais de perméabilité sur place que des essais oedométriques (effectués avec la cellule Rowe à 150m. de diamétre). Ces essais ont montré que la perméabilité était sensiblement plus élevé que celle mesurée au cours des essais effectués avec des oedomètres de faibles diamètres et que le traitement coûteux du sous-sol pouvait être évité.

On a surveillé le remblai intensivement afin d'évaluer sa stabilité et de porter au maximum la rapidité de sa construction.

La surveillance a montré que le remblai se comporte comme on l'avait prévu.

1 INTRODUCTION

A national highway is being constructed along the coast of Natal on the eastern seaboard of South Africa. The narrow, fairly flat, coastal strip is intersected by many rivers resulting in a number of flood plain crossings. Although the embankments are generally only about 5 m to 8 m high, experience has shown that significant deposits of soft silts and clays frequently occur, giving rise to stability and settlement problems. In addition, potential problems were anticipated regarding negative skin friction effects at the piled structures due to the subsoil consolidation.

As is now the usual practice, the investigations were conducted in two phases. The first was to obtain an overall view of the subsoil conditions and the second to carry out a detailed investigation if it proved to be necessary.

For this project, the overall programme had definite time constraints which the construction had to meet. It was therefore vital that the investigation should produce predictions of the time - settlement characteristics which would be used with confidence in deciding the construction schedule and determining whether or not any special construction techniques, such as vertical sand drains, would be required to meet this schedule.

A number of similar sites were investigated and this paper describes a typical one, at the Umzimbazi River. This embankment will be 5 m high but is currently 3 m high and has remained at that height for about four months. This has given an opportunity to compare the measured behaviour with that predicted.

2 SITE DESCRIPTION AND GEOLOGY

The highway will consist of dual two lane carriageways with sufficient width to allow for future widening to three lanes in each direction. The embankment width will be 41 m at the top and the height will be about 5 m. The river will be crossed by two parallel bridges, one for each carriageway, each consisting of a two-span structure with an overall length of 60 m.

The site consists of a flood plain approximately 500 m wide through which the Umzimbazi River meanders before discharging via a short estuary to the sea about 1,5 km to the east. At the crossing, the river is on the south side of the flood plain but one meander is so marked that the loop cuts back along the eastern toe of the embankment, as shown in Fig. 1. As part of the road scheme the river will be canalized to reduce the incidence of flooding of the valley which is cultivated with sugar cane, usually the water table is about one metre below ground level.

Geological information of the area indicated that the bedrock was expected to be a soft to hard, well bedded, jointed, dark grey to black, argillaceous shale.

Intrusions of dolerite frequently occur in the shale and, in many cases, the river courses are determined by these intrusions. This river rises close to the coast and the sediments deposited in it are primarily transported from the nearby shale and tillite hills so that soft clays and silts were expected to predominate. Immediately adjacent to the coast line the shale is overlain by dune sands - Berea Red Sands.

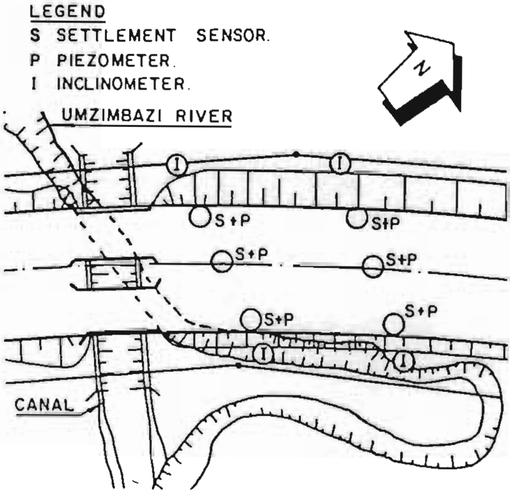

Fig 1: Site Plan

The general dip of the rocks along the Natal coast is in an easterly direction and at about 10° - 15°, but there is considerable local variation due to both faulting and distortions caused by the dolerite intrusions.

3 SITE INVESTIGATION

The geotechnical investigation was carried out in two phases. The first of these was intended to establish the local geology at the river crossing and to give preliminary indications of the strength and compressibility of the subsoil so that the magnitude of any potential problems could be assessed. Since these were significant, a second phase investigation consisting of detailed sampling and laboratory testing, was carried out to establish the necessary parameters for the design of the embankment.

3.1 First phase investigation

This consisted of four boreholes in which Standard Penetration Tests (SPT's) and thin wall sampling were carried out primarily for identification purposes. The geological section drawn up from these is shown in Fig. 2.

In addition, 10 Cone Penetration Tests (CPT - deep sounding - Dutch Probes) were performed. It has now become common practice to use the results from the CPT's to calculate settlements using the Buisman - de Beer approach (De Beer and Martens, 1957) viz:

$$\delta H = \frac{2,3 \times H}{C} \times \log_{10} \left(\frac{\sigma_z + \delta\sigma}{\sigma_z}\right)$$

where = settlement of layer

H = thickness of layer

σ_z = overburden stress at mid depth of layer

$\delta\sigma$ = increase in stress at mid depth of layer due to load

C = compression modulus

$$= \frac{\alpha qc}{\sigma_z} = \frac{2.3\ q_c}{\alpha_0\ \sigma_z}$$

where q_c = CPT cone pressure

α_0 = factor dependent on subsoil type

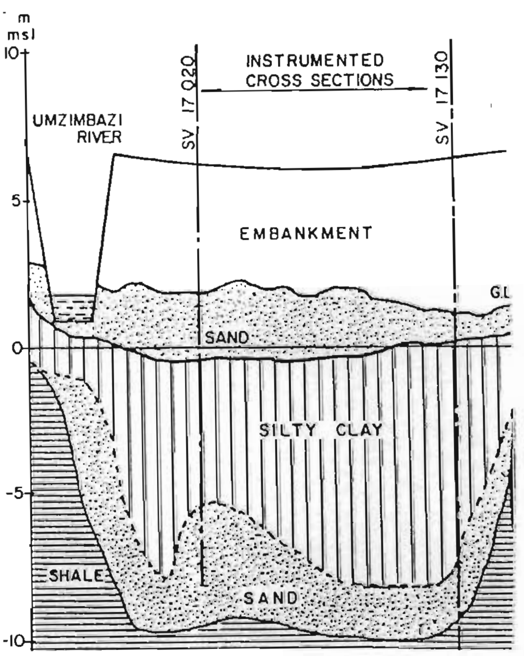

Fig. 2: Geological Section

This method is generally accepted for sands but it's use for settlement prediction for clays is less well substantiated. Nevertheless it is believed that for the initial estimates the method is of considerable value. (Bachelier and Parez, 1965; Gielly, Lareal and Sanglerat, 1970; Jones, 1975).

In addition to using the CPT results for settlement estimations, they are also used to give a preliminary indication of undrained shear strenghts (C_u) for soft clays. For this purpose, a conservative value of N = 20 is usually adopted in the following equation:

$$C_u = \frac{q_c}{N}$$

It should be noted that for normally consolidated clays in South Africa, the value for N found from comparisons of CPT and triaxial testing is generally between 15 and 20 but this may be significantly higher for overconsolidated clays.

The boreholes and CPT's (Fig. 3 & 4) showed that the flood plain subsoil consists of about 2,5 m of sand overlying 7,5 m of soft silty clay over about 3 m of sand over the shale bedrock.

Preliminary analyses of the boreholes and CPT results indicated that the estimated settlement was about 0,7 m and the undrained shear strength approximately 10 - 15 kPa. A total strength analysis utilising stability charts (Pilot and Moreau, 1973) showed that the embankment would have a Factor of Safety of about 0,9 with a height of 5 m and side slopes at 1:2.

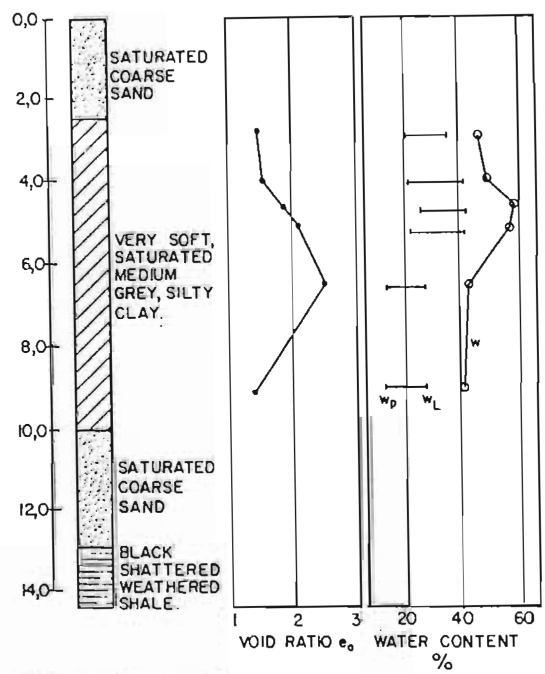

Fig. 3: Typical Borehole Log

The nature of the clay suggested that the permeability might be low, possibly giving rise to problems with rates of dissipation of pore pressures, but it was noted that numerous sandy lenses were encountered.

These results showed that further investigation was required, particularly since information regarding the permeability of the subsoil was essential.

3.2 Second phase investigation

This consisted of seven boreholes for both 100 mm and 150 mm diameter piston samples and in-situ permeability testing. The smaller diameter samples were used mainly for visual fabric assessment, whereas the larger samples were used for laboratory testing. Conventional consolidation tests on 50 mm diameter samples, and Rowe Cell tests on 150 mm diameter samples, with vertical and radial drainage, were carried out.

In order to obtain samples for the latter it was necessary to manufacture a 150 mm diameter thin wall piston sampler. In order to minimize disturbance, the tubes were kept fairly short, i.e. 500 mm. No problems were encountered in retaining the samples in the tubes during extraction from the boreholes, even when a very sandy lens was sampled.

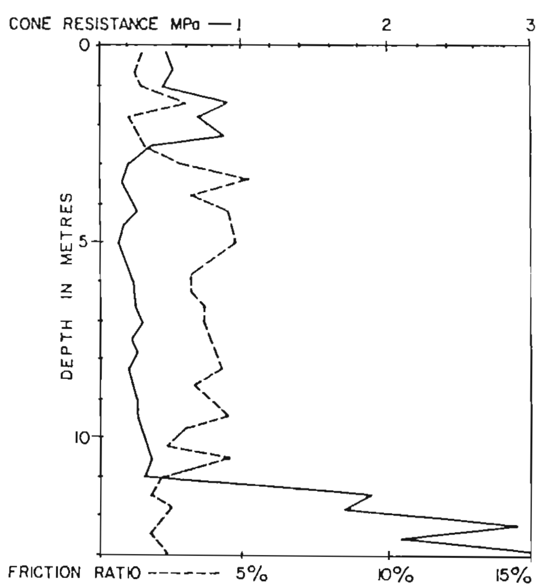

Fig. 4: Cone Penetration Test

It is frequently advocated that in-situ permeability testing is the most reliable method for assessing the time settlement characteristics of subsoil due to the advantages in testing effectively, very much larger and comparatively undisturbed samples (Lewis, Murray and Symons, 1975).

In-situ permeability tests were carried out by connecting a portable constant head permeability apparatus to twin hydraulic, porous ceramic piezometers which had been installed in a sand pocket of controlled dimensions.

The results of the in-situ permeability testing are shown as C_v's in Fig. 5, calculated from the field permeability and laboratory M_v's.

4 LABORATORY TESTING

Typical results are given in the tables below and shown in Fig. 3, 5 and 6.

Table 1. Atterberg limits, natural moisture and particle size.

Depth (m)	W_L %	W_p %	W %	%<0,075 mm	%<0,002 mm
2,75	36	20	47	61	-
3,90	41	23	51	80	13
4,50	43	27	60	71	28
5,00	43	22	58	83	24
6,50	29	13	42	53	17

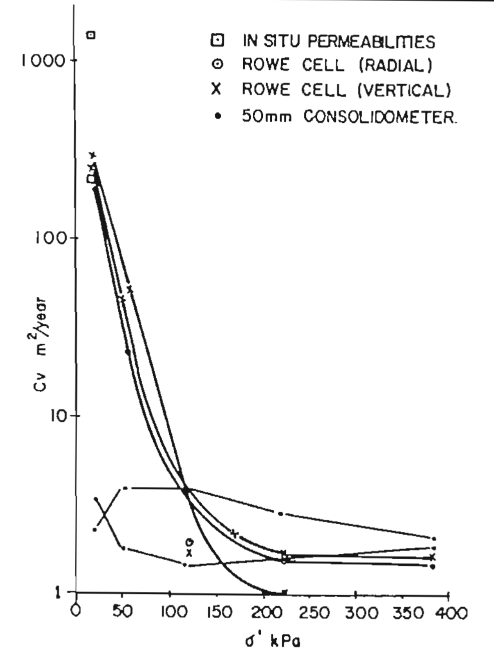

Fig. 5: C_v versus Effective Stress

Table 2. Triaxial tests - Undrained and drained.

Depth (m)	C_u kPa	C' kPa	ϕ'°
3,2	15		
5,0	11	15	25
6,0	16	20	19
6,2	12		
6,5		8	25

Table 3. Consolidometer - 50 mm diameter.

Depth (m)	M_v m^2/MN (100 - 200 kPa)	C_v $m^2/year$
2,75	1,47	0,5
4,5	0,67	1,1
5,0	1,06	0,8
6,5	0,43	2,0
8,7	0,49	2,0

Table 4. Consolidometer tests - 150 mm diameter Rowe Cell vertical and radial drainage.

Depth (m)		Effective Stress kPa 14-23	23-56	56-112	112-224	224-392
4,00 Vertical	C_v m^2/yr M_v m^2/MN	220 0,33	45 0,31	2,0 0,77	1,0 0,10	0,7 0,53
4,00 Radial	C_v M_v	162 0,30	19 0,41	2,0 1,15	1,7 1,29	1,5 0,57
6,50 Vertical	C_v M_v	31 0,41	14 0,41	12 0,71	4,0 0,71	3,0 0,37
6,50 Radial	C_v M_v	51 0,60	24 0,45	6 0,59	- -	3,0 0,53

In the calculation of the coefficients of consolidation in the above table, the square root-time method was used for vertical drainage with the Terzaghi time factor 0,848 for 90% consolidation, and for the radial case the 0,465 root method with a time factor of 0,334 (Silveira, 1953).

In addition to the above tests, dynamic consolidation tests were carried out by Techniques Louis Ménard in Paris since the Ménard dynamic compaction technique was being considered as a possible field treatment. These tests however, together with chemical analyses, indicated that the subsoil was not suitable for the Ménard method.

Geological history indicates that the material is a normally consolidated recent alluvium. The laboratory tests and the visual description of the dominant subsoil material show that it is highly compressible, very soft to soft silty clay. Local experience has often shown these deposits to have a higher strength surface crust, overconsolidated due to dessication, which can be of significance primarily because it allows reasonable trafficability on the site. The higher strength is usually discounted in stability analyses because of the possibility of tension cracks arising from prior shrinkage or settlement under the embankment. At this site however, there is no evidence of overconsolidation in the clay, both from the e-log p curves, from comparisons of C_c from these and from Liquid Limits, and from C_u measured and calculated from Plasticity Indices.

It will be observed that Fig. 3 shows that the natural water contents, almost without exception, exceed the liquid limits, i.e. the average Liquidity Index = 1,54 which is somewhat higher than was expected.

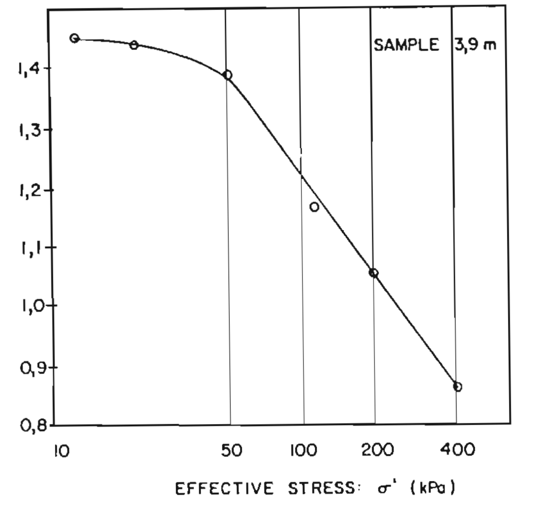

Fig. 6: Typical e - log σ'

5 ANALYSES

The analyses are considred in two parts, i.e. settlement and stability.

5.1 Settlement

The coefficients of volume change M_v, were fairly consistent both from the small diameter samples and from the large diameter tests with vertical and radial drainage. On this basis, the estimated settlement was 0,8 m, which is similar to that predicted from the preliminary Cone Penetration Tests. Since it was clear that the rate of construction would be restricted by the excess pore pressure dissipation rate, it was not possible to calculate the overall time for settlement until the construction rate had been determined. Simplified calculations however were carried out to establish the order of time, assuming instantaneous full height construction, using the C_v's from the Rowe Cell laboratory tests and those derived from the in-situ permeabilities. It will be seen from the typical results given in Table 4 that there is no significant difference in the vertical and horizontal coefficients of consolidation; because of this, and the large embankment width, the problem was therefore considered to be one of plane strain with single vertical drainage only.

These estimates indicated that the time for consolidation, would be about 2,5 years to which should be added the construction time. A similar calculation using the C_V values from the small diameter consolidometers, gave a time of about 10 years, which would not have been acceptable. The actual time for construction, calculated later as 3 months, has to be added to the 2,5 years, but it must be noted that the estimate is for 90% consolidation. The remaining consolidation settlement, say 80 mm, together with a similar allowance for secondary compression was considered to be acceptable for the embankment itself but would cause problems at the bridge due to negative skin friction. It was therefore decided to surcharge at the abutment position with an extra 2 m of fill.

5.2 Stability

Stability analyses were carried out using the Bishop Method of Slices assuming a circular failure surface. These showed that the maximum height which could be built, assuming no excess pore pressure dissipation was 2,5 m if a factor of safety of 1,2 is taken as the lowest permissible limit during construction.

Similar calculations indicated that the long term factor of safety, assuming complete excess pore pressure dissipation was F = 1,57. These calculations were made on the basis of strength parameters $c' = 5$ kPa and $\phi' = 35°$ for the fill which was shale from an adjacent cutting. The subsoil parameters were taken as $c' = 0$ and $\phi' = 25°$ although the laboratory tests showed a small cohesion, there was no evidence of overconsolidation, and this cohesion was conservatively ignored. The water table was assumed to be at surface and any benefit from the sandy layer close to the surface was ignored since this layer had been shown to be variable and practically non-existent in some areas.

The long term stability was therefore considered to be satisfactory so that the problem became simply one of determining the appropriate construction rate.

The model used was the simple one of increasing the height of the embankment, above an intitial 2,5 m height, in increments of 0,5 m. It was assumed that the actual time to construct each layer was negligible and that the excess pore pressure response in the clay was instantaneous and equalled 80% of the load increment. The factor of safety was limited to 1,2 and therefore, at any height, an allowable excess pore pressure was calculated. It was then possible to calculate the amount of dissipation needed at each embankment height such that the allowable excess pore pressure for the next load increment would not be exceeded. The times for dissipation for each height increment were calculated and summarised to give an estimate of construction time. The purpose of this was to check that even with pessimistic assumptions, the actual construction rate would nevertheless be acceptable. Since in this case the time was calculated as 3 months this was considered as reasonable.

This approach to the construction entailed a relatively high risk since the objective was to keep the factor of safety low and the rate of construction maximised so that the time available for subsequent settlement could be maximised.

The advantage of this was that artificially accelerated subsoil drainage or wide berms to increase the stability of the embankment were unncessary. An essential corollary of this overall approach however, is that the embankment must be monitored during construction to check the stability and to confirm that the actual field dissipation rates are at least as high as the predicted values.

6 MONITORING

Two lines of piezometers were put in, with three piezometers in each line as shown in Fig. 1. The piezometers were of the three lead, pneumatically operated bellows type with the leads taken to gauge houses outside the toe of the embankment. Readings were taken by connecting a portable compressed CO_2 bottle to one of the leads.

Stability control charts were drawn up as shown in Fig. 7 and these showed that the factor of safety was satisfactory. It would be imprudent to rely solely on pore pressure measurements to assess stability, and various strain measurements were considered to be an essential back up system. It is generally agreed that settlement measurements alone are insufficient and that a combination of these and horizontal deformations gives a better control. Settlement gauges and inclinometer casings were therefore installed in the positions shown in Fig. 1.

The settlement sensors were of the same type as the piezometers, i.e. pneumatic, with the bellows pressures resulting from mercury filled leads to mercury reservoirs in the gauge houses. A total of six sensors were placed approximately 0,5 m below ground level before the start of construction.

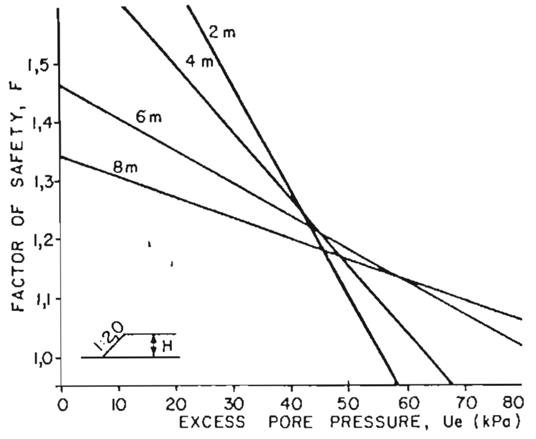

Fig. 7: Stability Control Chart

The gauge houses were levelled at the time of each reading, with reference to a nearby bench mark installed into rock. The inclinometer casings were grouted into rock at their bottom ends so that absolute values of displacement could be measured. In addition to the settlement sensors described above, simple water level U-tube sensors at different heights within the fill were also installed. In general the inclinometers showed very little horizontal movements only averaging about 20 mm. However, one inclinometer, the eastern one on the northern line, showed a movement of 70 mm with the peak value at 2 m depth.

It will be seen from the plan - Fig. 1 that this position is close to the loop of the river which was filled with sand after the diversion canal had been cut. The maximum displacement occurred at the old river bed level. As a precaution, the frequency of taking readings was increased; these showed that the rate of horizontal movement decreased rapidly as soon as loading stopped suggesting that failure was not imminent.

The ratio of horizontal strain at the toe, to vertical strain at the centre was plotted in the manner described by Matsuo and Kawamura (1977) and shown in Fig. 8. It should be noted that even the one comparatively large horizontal displacement mentioned above does not indicate any problem on this chart.

This would appear to be a useful method for describing the embankment behaviour since the latter can be seen qualitatively at a glance from the rate and direction at which the plotted line approached the factor of safety lines.

It was not necessary to curtail the rate of construction due to any danger signs but this was probably because the rate of construction was considerably slower than the maximum calculated since it was convenient to use the earthmoving equipment at an adjacent similar site. A typical plot of height of fill, pore pressure and settlement is given in Fig. 9. The settlement is close to the predicted levels and the rates of pore pressure dissipation and settlement are in reasonable agreement with the predicted values.

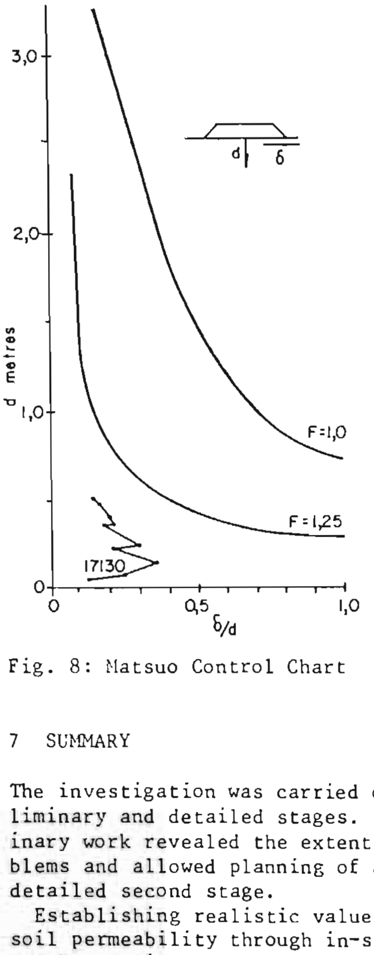

Fig. 8: Matsuo Control Chart

7 SUMMARY

The investigation was carried out in preliminary and detailed stages. The preliminary work revealed the extent of the problems and allowed planning of an economical detailed second stage.

Establishing realistic values of the subsoil permeability through in-situ testing and large diameter consolidometer (Rowe Cell) tests was a vital contribution to the overall design.

The decision to build the embankment at a monitored controlled rate was based on the predicted behaviour and on comparisons of costs with other schemes such as installation of drains and utilising flatter slopes or stabilising berms.

The monitoring has shown that the embankment has behaved as predicted and has more than justified the cost of the extensive investigation through the minimizing of overall construction costs.

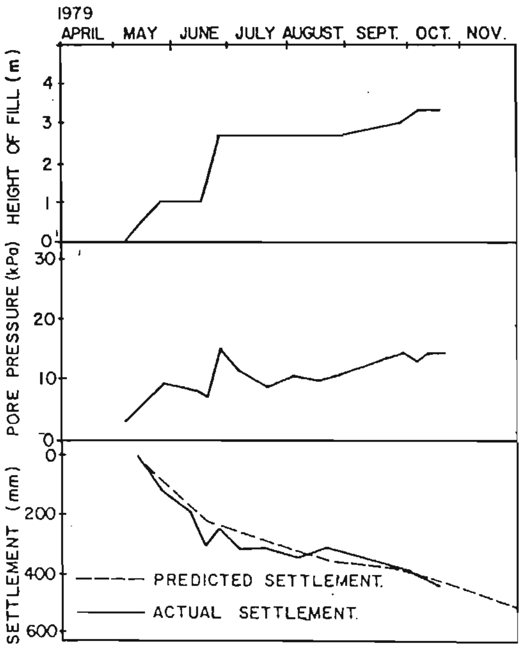

Fig. 9: Typical Monitoring Record

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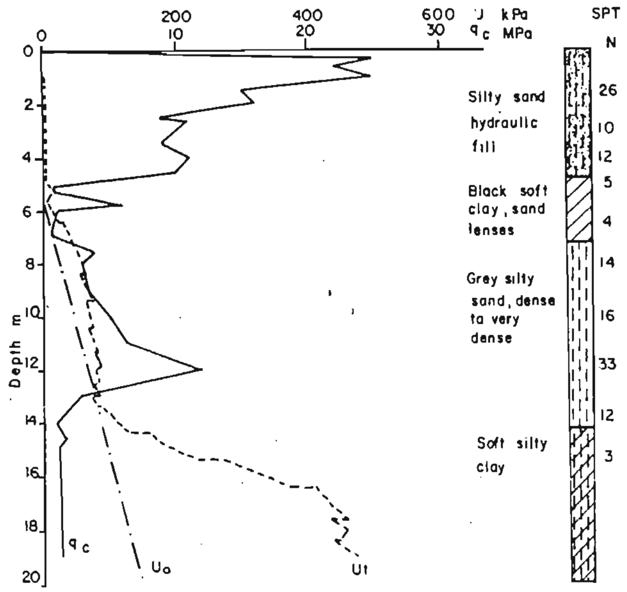

Fig. 4 Typical result under storage shed

(c) Gypsum tailings dam. An existing and practically complete dam was tested to obtain data on the state of consolidation since the possibility of reclaiming the dam was being considered. A typical probe result with borehole log is shown in Figure 5.

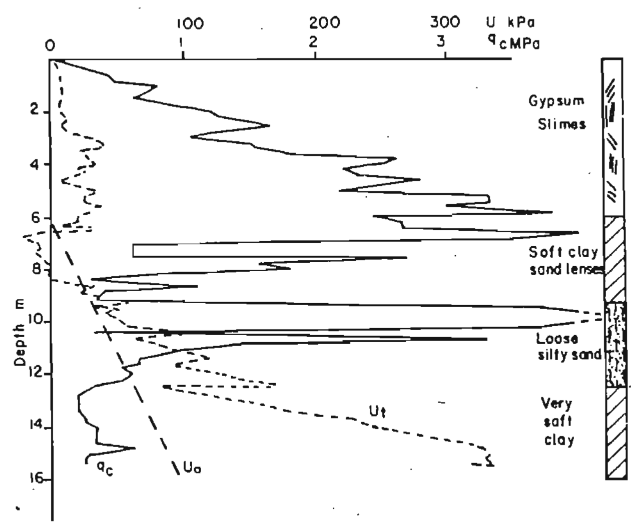

Fig. 5 Typical result through gypsum tailings dam

(d) Gold mine tailings. A dam was probed, primarily to establish the layering and relative permeability of these layers, so that the most appropriate modelling of the dam could be made for the purpose of stability analyses. Typical results are shown in Figure 6. No borehole log was available for the subsoil, nor was this penetrated by the tests.

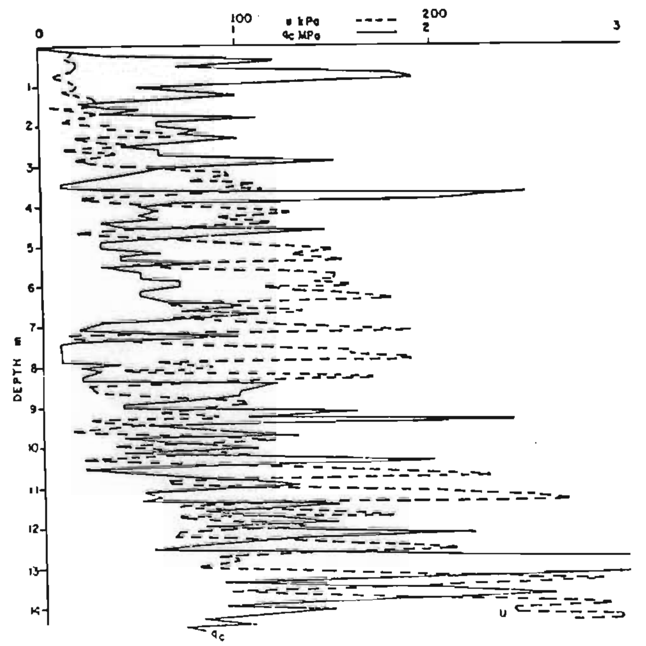

Fig. 6 Typical result from gold mine tailings dam

(e) Platinum tailings. A dam was probed for the same reasons as the gold tailings dam and typical results are shown in Figure 7. Information on the subsoil was available and this consisted of stiff, black, shattered and slickensided slightly sandy, silty clay derived from residual decomposed norite with W_L = 65%; I_p = 38% and a clay (0.002 μm) fraction of 60%.

DISCUSSION

(i) Consolidation. It is emphasised that the primary purpose of the test programme was to establish semi empirical correlations of field and laboratory consolidation data.

Many field dissipation tests were carried out and facsimiles of parts of typical results are shown in Fig. 8. The upper part of the figure shows the pore pressure response during penetration, i.e. the chart is on a depth base. The lower part shows dissipation tests of pore pressure against time. Superficially the results are similar to laboratory consolidation test data and a similar processing method is therefore adopted.

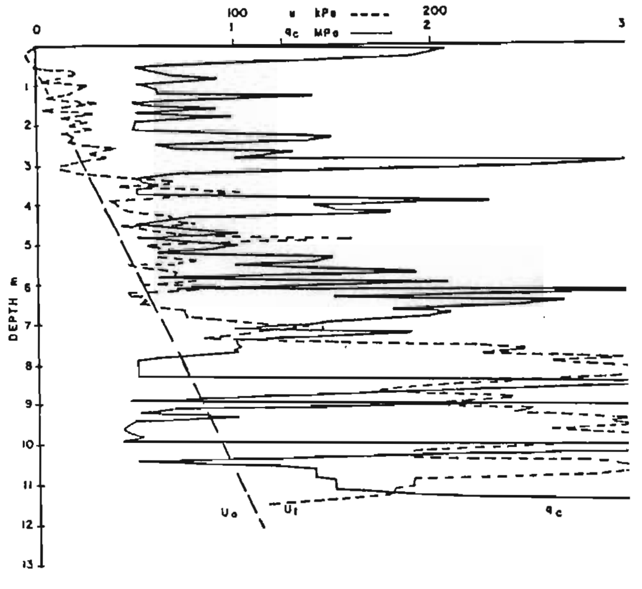

Fig. 7 Typical result from platinum mine tailings dam

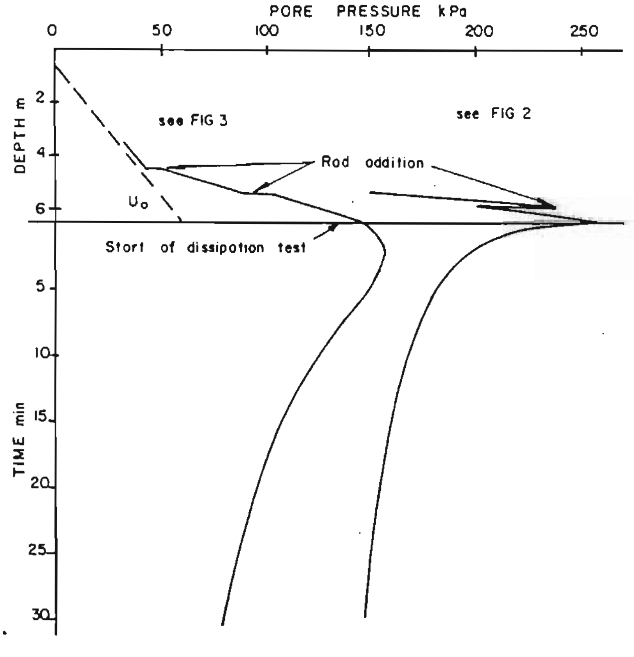

Fig. 8 Typical dissipation test results

The right hand part shows results taken from the probe test shown in Figure 2. It will be seen that on stopping penetration there was a practically instantaneous decrease in pore pressure and that this portion was sometimes a significant part of the total dissipation. This was seen very clearly on the actual charts of test results and it was necessary to switch the chart recorder to the time control immediately before stopping penetration to correctly record this. It was tentatively suggested that this immediate decrease was a dynamic effect due to the stopping of penetration and should therefore not be considered as part of the dissipation. A data correction procedure was therefore adopted of plotting the results on a square root of time basis as shown in Figure 9.

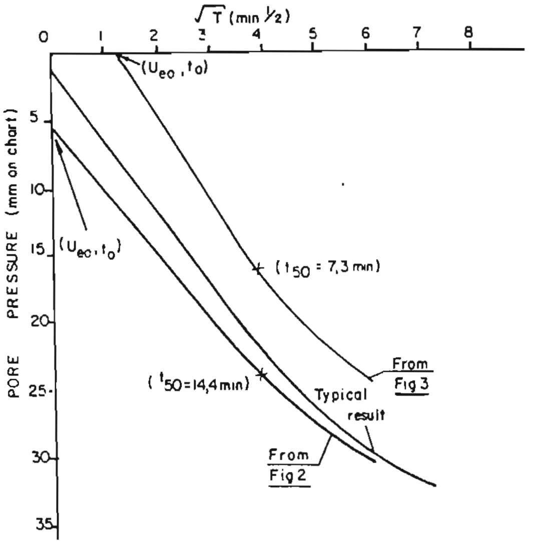

Fig. 9 Typical pore pressure against square root time

A practically straight line part to the curve resulted, which intersects the pore pressure axis. This point was taken as the dissipation pore pressure datum, u_{eo}, with the time axis unchanged and the datum as t_o. Final dissipation was taken as t_{100}, μ_{e100}, and the parameter for the subsoil consolidation characteristics, t_{50}, was defined as the time at which half the dissipation had occurred u_{e50}. The t_{50}'s were read off the pore pressure against square root of time plots.

The left hand side of Figure 8 shows the result from the probe shown in Figure 3. On stopping penetration the pore pressure increased before subsequently decreasing. The reason for this was not established but it was presumed to be partial blocking of the filters which in this case were the face mounted elements possibly with some loss of saturation. It was noted that if the results were plotted on the same square root of time basis as the previous case, then a straight line section intersects the time axis at a real time.

This was taken as the time datum, t_o, by shifting the time axis; the pore pressure datum, u_{eo}, was taken as the maximum pore pressure recorded. Because of the possible doubt concerning the validity of these results they were not used for correlations with laboratory data, it was however interesting to observe that the slopes of the straight line portions, and the t_{50}'s deduced, appeared to be in close agreement with those derived from the much more usual dissipation tests in which immediate decrease of pore pressures occurred on stopping penetration. It is possible therefore, that although partial blocking of the filter elements inhibits response during penetration, it may not be significant for the much longer times for dissipation tests.

In the type of tests described above, the time for full dissipation may be long and inconvenient, an alternative method of arriving at t_{50}, or an equivalent parameter, was desirable.

If the tests were being carried out at a site where the in-situ pore pressure regime was simply the hydrostatic head, then if this was known from the water table, the u_{es0}, can be taken from the root time plot without the full dissipation being achieved, provided that it was continued beyond the t_{50} time. This made a considerable difference to the time required for testing and made the procedure very much more convenient for use as a routine field test.

If the pore pressure regime was not known, it was considered possible that a deduction of the permeability could be made from the slope of the straight line part of the dissipation results, (see Figure 9). It was found however, that although the slopes, which are the initial rates of dissipation, gave a relative indication, it seemed to be too insensitive to use as a quantitative measure. It also implied a square root time modelling which may not be valid and was therefore not pursued further.

Typical t_{50}'s from the results of dissipation tests carried out at a site at which comprehensive laboratory large diameter (150 mm) as well as standard 50 mm diameter, consolidation test data was available, are shown plotted against the reciprocal of the coefficient of consolidation in Figure 10. A best fit straight line through the origin was drawn and resulted in the following equation :

$$t_{50} \text{ (mins)} = \frac{50}{c_v} \text{ (m}^2\text{/year)}$$

It is suggested that this equation should be seen as a preliminary attempt to establish a useful field relationship and that much more data is needed to refine this or similar correlations. The ease and economy of cone pore pressure testing (CUPT) should be borne in mind so that the advantage of being able to obtain consolidation data from simple field testing is fully exploited. It may also be noted that considerable scatter usually occurs in laboratory test consolidation data resulting in difficulty in selecting appropriate values for analyses. A greater number of field tests may provide a better statistical selection procedure. It was incidentally noted that the field t_{50}'s were generally about an order of magnitude higher than laboratory t_{50}'s where the consolidation test equipment had a drainage path length of 10 mm.

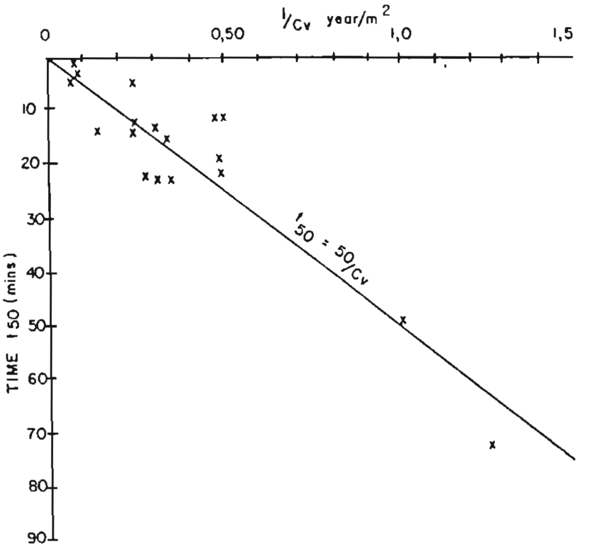

Fig. 10 Correlation of field dissipation times with laboratory consolidation data

This implied that the effective length of the field drainage path was about 35 mm which intuitively seemed reasonable for a 35 mm diameter cone.

(ii) In-situ pore pressure measurements. A secondary purpose of the field testing was to measure excess pore pressures under an embankment which was being constructed. A number of the piezometers had shown that high pore pressures existed and that although settlement appeared to be as expected, the pore pressures showed very little dissipation. Holes were drilled through the embankment and CUPT's carried out into the subsoil. These showed high pore pressures under the embankment with the values in agreement with the piezometers. They also showed that the t_{50}'s were higher for material under the embankment than for material at the same depth outside the embankment which was expected, since the laboratory work (Jones and Rust, 1981) had shown c_v to be highly stress dependent.

(iii) Subsoil Identification. All the results of CUPT's, where independent information on the nature of the subsoil was available, confirmed observations by authors referred to previously that the pore pressure response is extremely sensitive to the nature of the subsoil. Sufficient data has not been analysed to develop quantitative relationships but if the suggestion by Baligh et al (1980) was adopted of using u/q_c as an identification parameter, then the results from natural materials shown in Figures 2 and 4 indicated that for soft clays u/q_c varies from about 0.4 to 1.0 which agrees with the Baligh et al results. Loose sands showed u/q_c values of about 0.2 to 0.5 and dense sands have lower values.

The gold mine tailings dam u/q_c values were extremely variable (as shown in Figure 6) from about 0.02 where the cone pressures were highest, to about 0.3 for the lower cone pressure zones. These were assumed to reflect coarser and finer layers respectively, resulting from the method of construction of the tailings dam.

The platinum tailings dam results (see Figure 7) showed similar results except that the variation was considerably smaller, both in the thickness and number of layers and in the u/q_c value which were typically from 0.02 to 0.10. The results from both these dams showed remarkably good definition of layers indicated by the virtually totally consistent results of low cone pressures accompanied by high pore pressures and vice versa. Figures 6 and 7, because of the size of the drawings, are much less detailed than the field chart results where the reversals of cone and pore pressures were strikingly obvious. The charts enabled layers of about 25 mm thickness and less, to be clearly distinguished.

The gypsum slimes dam generally showed less pronounced layering with an overall u/q_c of greater than 0.5

The layering in these dams, which varied spatially and in consistency, undoubtedly has a most significant influence on the overall permeability. This may be taken into account in the development of the most appropriate flow nets for dam design purposes and could also affect construction practice. However, despite the apparent confirmation of the use of u/q_c as a material identification index it is suggested that some caution is required in that in many cases the selection of representative values from the field results in a multi layered subsoil system is difficult because of the rapidly changing values of both pore pressure and cone pressure. A further potential difficulty is that in some cases (see Figure 2) the pore pressures, in a layer which appeared to be very consistent, increased much more rapidly than the cone pressures so that there was no unique u/q_c value. This suggested that a measure of the rate of increase of pore pressure would be a more appropriate parameter, which would appear to contradict Baligh et al's findings. It should be noted however, that the rapid increases in pore pressure with depth, referred to above, occurred between stoppages for dissipation tests; a method of correction for these was not defined, nor was the overall effect of the repeated stoppages clear.

Because the emphasis of the work described here was on dissipation tests from excess pore pressures, u_e, down to hydrostatic pore pressures, u_o, the authors suggest that u_e may be a more sensitive measure than u_t, for defining a material identification parameter; this could lead to a normalized subsoil index, I_s, defined as follows :

$$I_s = \frac{(u_t - u_o)/u_o}{(q_c - \sigma_{vo})/\sigma_{vo}}$$

where σ_{vo} = vertical total stress.

Insufficient data has been processed for a thorough comparison of u/q_c values with the above. The values would obviously be different and in many cases would be negative. This could possibly be an advantage in that it would lead to a clear indication of dilatant materials. However, it may be noted that the expression given above is effectively a measure of the pore pressure parameter at failure, A_f, and it seems unlikely that this would usefully define the nature of the soil but would define the state of consolidation. An alternative materials index may perhaps be simply the ratio of excess pore pressure, μ_e, to the hydrostatic pore pressure, μ_o, i.e. $(\mu_t - \mu_o)/\mu_o$.

CONCLUSIONS

The use of a pore pressure measuring sensor in a probe to give a cone with pore pressure penetration test (CUPT), considerably enhances the data obtained from probing.

At present there is insufficient theoretical basis for the full interpretation of the pore pressure information obtained. It is therefore suggested that efforts should be made to establish semi empirical correlations.

In-situ pore pressure dissipation tests were carried out and the results compared with laboratory data. These led to the following relationship :

$$t_{50} \text{ (mins)} = \frac{50}{c_v} \text{ (m}^2\text{/year)}$$

The pore pressure probe was also used for measuring pore pressures under embankments during construction and gave results in agreement with in-situ piezometers.

Data from the CUPT's was also used to confirm the use of u/q_c as a materials identification parameter. Although the results are encouraging, it is suggested that this parameter may be more suitably defined in terms of excess pore pressures rather than total pore pressure.

ACKNOWLEDGEMENTS

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(vi) DESIGN AND MONITORING OF AN EMBANKMENT ON SOFT ALLUVIUM

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DESIGN AND MONITORING OF AN EMBANKMENT ON SOFT ALLUVIUM
LE DESSIN ET LA SURVEILLANCE D'UN REMBLAI CONSTRUIT SUR TERRAIN ALLUVIONNAIRE MOUS

Session No.

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SYNOPSIS The construction of a freeway along the Natal Coast, South Africa necessitated an embankment 400 m long and 5 m high, crossing the Umgababa River and flood plain. The subsoil consisted of loose sands and soft silty clay to a depth of 23 m. Analyses indicated that stability and settlement problems were to be expected. Since the planning had allowed time to overcome these potential problems, it was not necessary to provide expensive subsoil drainage systems. However, extensive instrumentation was required to control the rate of construction and to monitor the embankment performance. This showed that surcharging had to be provided to minimize differential settlements between the structure and the embankment, but nevertheless, provision had to be made for negative skin friction and lateral loads on the piles.

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INTRODUCTION

A six lane dual carriageway is currently under construction along the Natal coast of South Africa. The rugged topography inland, restricts development to the narrow, fairly flat, coastal strip. This is intersected by numerous valleys with soft alluvial deposits of considerable depths. Due to these, the height of embankments across the flood plains are minimized, and are generally dictated by expected flood levels. Nevertheless, the 5 m high embankment over 23 m of soft sands and clays, was expected to cause problems, not only for the embankment, but also for the structures.

The traffic growth in the area required the opening of the highway about 3 years after the start of construction. One of the principle objectives of the site investigation was to enable times for settlements to be predicted with sufficient confidence to devise a suitable construction programme.

SITE DESCRIPTION AND GEOLOGY

The road crosses the Umgababa River flood plain in a north-south direction.

As shown in Figure 1, the Site Plan and Geological Section, the length of the embankment is about 400 m. The width at road level is 41 m and the height varies from about 5 m in the centre to 8 m at the ends. At the southern end there will be a secondary road underpass, and in the centre, the main river bridge. At present the river runs along the northern edge of the flood plain. It will be realigned to provide a less skew crossing at the proposed bridge, and also to improve the alignment of the river at the existing main road bridge some 200 m downstream. This part of the coast has a recent history of bridges, and approach embankments to these bridges, being washed out at times of extreme flooding.

The flood plain is very flat and about 2.6 m above mean sea level; the sea is about 1.5 km to the east and causes tidal variations in the river level of about a metre, at the embankment position.

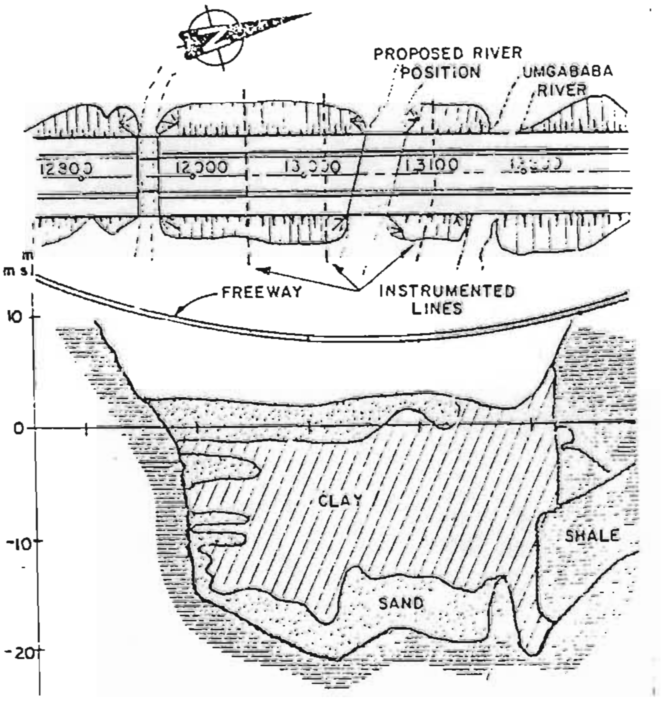

Fig.1 Site Plan and Geological Section

The geology consists of alluvial deposits overlying shale bedrock at about 23 m depth. The alluvium is variable, comprising both sands and very soft organic silty clays containing shells. The rock is a dark grey soft to medium hard, well bedded and jointed argillaceous shale. The shale generally dips towards the east at about 10° to 15° and this has given many problems in the stability of cuttings, particularly on the east facing cut slopes.

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The shale is frequently intruded by diabase dykes and sills, and since in many cases these weather at a different rate to the shale, river courses are often determined by these intrusions.

SITE INVESTIGATION

The investigation was carried out in two phases. During the first, the overall geology was established and the strata at the bridge sites determined for structural design purposes, by carrying out Cone Penetration Testing (CPT) and NX diameter boreholes (see Figure 2). Undisturbed samples 50 mm diameter were taken using thin wall Shelby Tubes, and laboratory tested for preliminary assessment of strength and compressibility. Standard Penetration Tests (SPT) generally gave very low values (N=1) in the dominant soft clay layer.

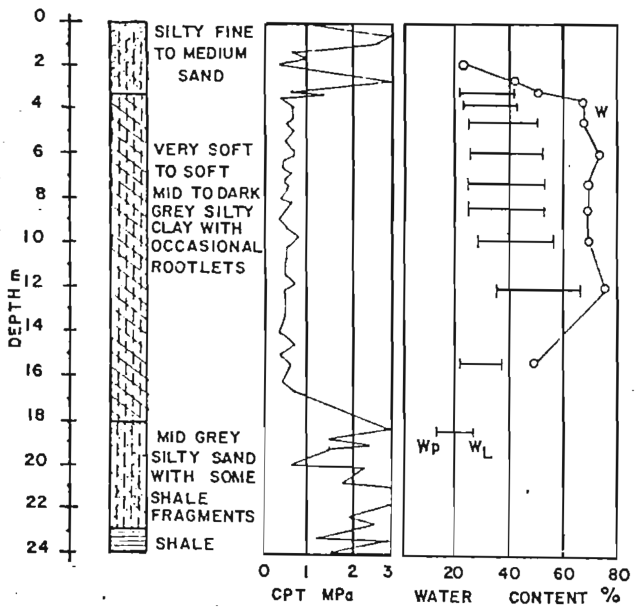

Fig.2 Typical Borehole and CPT Log

The first phase of the investigation indicated that severe potential problems existed and therefore a second phase was necessary. This consisted of further boreholes with 150 mm diameter piston sampling.

Undrained quick triaxials, drained triaxials, 50 mm diameter conventional consolidometers and 150 mm diameter Rowe Cell consolidation tests, permitting both vertical and horizontal drainage, were carried out.

Typical results are given in Table 1, and Figure 2. In-situ constant head permeability tests were carried out using twin tube porous ceramic piezometers carefully installed in sand pockets of controlled dimensions. Coefficients of consolidation, c_v, calculated from these, together with laboratory c_v's from both small and large diameter samples, are shown in Figure 3, plotted against effective stress.

TABLE 1

Laboratory Test Results

Depth m / Sample dia. mm		Effective Stress kPa 14-28	28-56	56-112	112-224	224-392
3.1	c_v m^2/yr				1.7	1.6
50	m_v m^2/MN				0.44	
4.3	c_v				5.0	6.0
50	m_v				0.70	
3.1	c_v	256	50	12	0.9	0.8
150	m_v	0.50	0.31	0.67	1.49	0.51
4.3	c_v	377	21	9	5	0.5
150	m_v	1.94	0.48	0.67	1.40	0.80

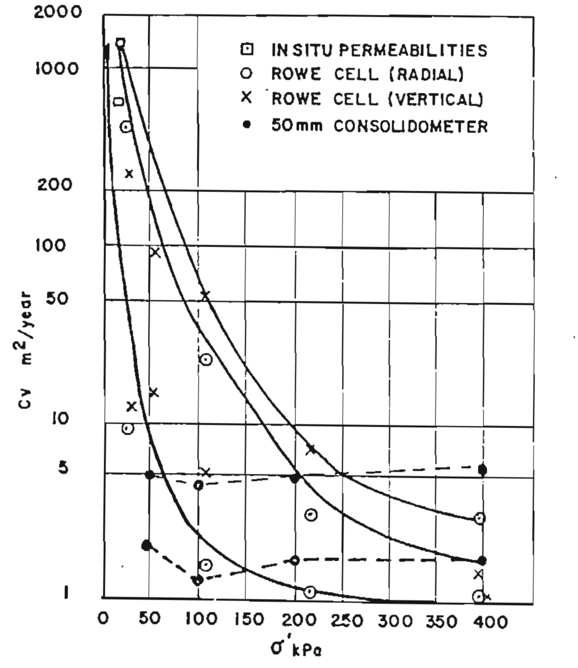

Fig.3 Coefficients of consolidation versus effective stress

ANALYSIS

The stability and settlement analyses were carried out following each of the two phases of the investigation.

First Phase Stability

Total stress analyses were carried out using the Bishop Method of Slices and also from stability charts, (Pilot and Moreau, 1973) with shear strengths derived from Cone Penetration Testing and from quick undrained triaxial tests. These showed that the embankment height was limited to 3 m for a factor of safety of 1.2.

The mean cone reading, $\overline{q}_c$, was 392 kPa, the standard deviation was 43.9 kPa and the coefficient of variation, V_{qc} was 11%.

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The undrained shear strength was calculated from (Lunne, Eide, and de Ruiter, 1977)

$$q_c = N_k \tau_f + \gamma z$$

where N_k = cone factor

τ_f = undrained shear strength

γ = soil unit weight

z = depth

From the above, c_v = 20 kPa if N_k = 15, which is a commonly accepted value for this type of normally consolidated soft clay.

The laboratory tests showed c_u to be lower, but this was assumed to be due to sample disturbance.

First Phase Settlement

Settlement calculations were based on the CPT results, using the Buisman - de Beer approach, (de Beer and Martens, 1957), but modified as shown below, (Bachelier and Parez, 1965; Gielly Larel and Sanglerat, 1970; Jones, 1975).

$$\delta H = \frac{2.3\ H}{C} \log_{10} \left(\frac{\sigma_z + \delta\sigma}{\sigma_z}\right)$$

where $C = \dfrac{2.3\ q_c}{\alpha_0\ \sigma_z}$

where α_0 = factor dependent on subsoil type, assessed from CPT friction ratios and sample description.

The calculations showed that settlements of about 0.9 - 1.2 could be expected for an embankment height of 5 m.

These preliminary analyses indicated that significant stability problems could be expected, and that unacceptable differential settlements between the embankment and the structures would occur.

In order to plan a construction programme to minimize these problems, it was necessary to determine with more precision, the subsoil strength and consolidation characteristics and their variations.

Second Phase Stability

Although the emphasis of the second phase of the investigation was on determining the consolidation parameters, laboratory tests also provided effective stress parameters for more detailed stability analyses.

The mean values of the laboratory effective stress parameters were $\overline{c}'$ = 5 kPa and $\overline{\phi}'$ = 25°. However contrary to the very small variation shown by the CPT's, the laboratory tests were highly variable, i.e. V_c = 40% and $V_{\tan \phi}$ = 20%.

For the purpose of analysis, $\overline{c}'$ was conservatively taken as zero throughout, since it was small and variable. Although the factor of safety is not very sensitive to changes in the fill parameters, these were taken also conservatively, as $\overline{c}'$ = 5 kPa and $\overline{\phi}'$ = 35 with V_c = 10% and $V_{\tan \phi}$ = 5%, for the selected compacted soft rock shale fill.

Probability calculations using normal and beta distributions of the subsoil parameters (Harr, 1977), indicate probabilities of failure of 0.096 and 0.104 at a factor of safety of 1.2, i.e. the construction limit. If, however, the factor of safety increases to 1.6 after some dissipation of excess pore pressures, then the probability of failure is less than 0.01. It must be emphasised that the estimation of probabilities of failure, was to give a basis for comparing the relative safety of situations represented by conventionally calculated factors of safety. Since all the assumptions are pessimistic the calculated probabilities are also pessimistic. For instance the variation of tan ϕ' is expected, on the basis of the CPT results, to be small and if this is taken as say 10%, instead of 20%, then the probabilities of failure for the normal and beta distributions reduce from 0.096 and 0.104 to 0.005 and 0.0006 at a factor of safety of 1.2.

Construction control stability charts were drawn up (as shown in Figure 4), using conservative assumptions with a simplified subsoil model consisting of a single clay layer.

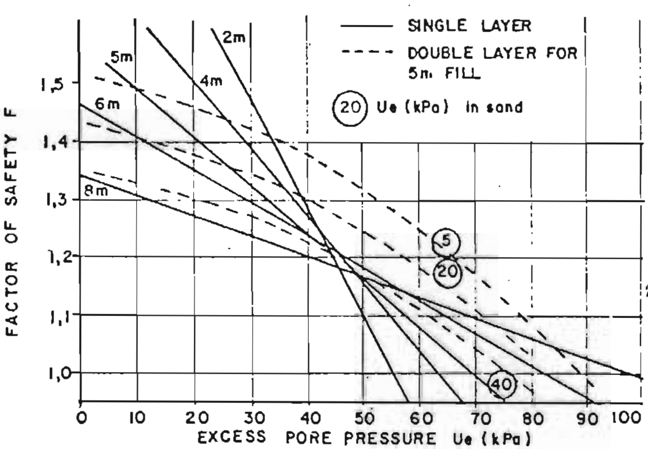

Fig.4 Site Stability Control Charts

The allowable rate of construction was expected to be a function of the pore pressure dissipation and a provisional programme was drawn up based on the predicted dissipation rates from consolidation tests and in-situ permeability measurements.

The time for construction was arrived at by modelling the embankment as a series of instantaneous 0.5 m lifts, with each increment being added after the previous excess pore pressure had dissipated sufficiently to allow the next increment. A number of calculations were carried out with different assumptions regarding the pore pressure response, δu_e, for additional loading, $\delta\sigma_v$, i.e. $\delta u_e = \delta\sigma_v$ and for $\delta u_e = 0.8\ \delta\sigma_v$ and $0.6\ \delta\sigma_v$. These resulted in considerably different predicted construction times. On the basis of these results, tempered with judgement, it was concluded that the required time would be from about 6 months to 1 year.

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Second Phase Settlement

Settlements were predicted from CPT's, as described earlier, and from coefficients of volume change, m_v, from large and small diameter consolidometers. The estimated settlements, for a 5 m embankment, were 0.9 - 1.2 m from the CPT's and 1.2 - 1.5 m from the laboratory results.

It will be seen from the Geological Section, Figure 1, that the bridge at the southern end could be founded on footings - eventually this bridge was moved some 10 m further south to minimize the foundation depth - and settlement of the embankment would therefore only create the problem of a distortion at the interface of the embankment and the structure. However, for the river bridge at the centre, post construction settlements would not only do this, but also induce additional stresses in the piles due to negative skin friction and lateral loading.

Heavy flooding may cause scour, estimated at up to 10 m deep, at the piles, and therefore stiff, large diameter bored piles are preferable to smaller diameter driven piles.

In order to minimize post construction differential settlement between the embankment and the structure, the bridge position was preloaded by building the embankment across the future river position. To obtain the maximum benefit from this, the river diversion across this section was delayed for as long as possible and this had the added advantage of allowing construction of the bridge in the dry. The delayed filling in at the initial river position was expected to cause differential settlements along this section of the embankment, but since these would not be adjacent to structures, they were considered to be acceptable.

A number of different schedules were examined before deciding on the one described above. An important factor in evaluating the alternatives was that this section of freeway would be completed before the adjacent section to the south, so a temporary link to the existing main road was to be constructed about 1 km to the south of the embankment. It was therefore valuable to have the option of constructing the embankment with temporary pavement and surfacing, which could be completed later, after most of the settlement will have occurred. This allowed more than usual flexibility in the overall planning and was instrumental in the adoption of the proposed programme. Nevertheless it was important to predict rates of settlement for the various operations.

The rate of settlement is essentially governed by the dissipation of excess pore pressures in the clay layer from about 4 m to 18 m deep. The Rowe Cell tests had indicated little difference in the horizontal and vertical permeabilities and vertical double drainage was assumed.

The time for 90% consolidation, t_{90}, can be taken as :

$$t_{90}\ \text{(years)} = \frac{0.85 \times 7^2}{c_v\ (\text{m}^2/\text{year})} = \frac{42}{c_v}\ \text{years}$$

Figure 3 however, shows c_v's to vary from about 15 to 3 m^2/year for the range of final effective stress between top and bottom of the clay. The higher values, at lower stresses, tend to be confirmed by the c_v's obtained from the in-situ permeability tests.
It will be noted in Figure 3, that the c_v's obtained from the small diameter consolidation tests appeared to be much less dependent on the effective stress, and generally gave lower c_v's in the relevant stress range.

The more realistic c_v's obtained from the large diameter testing - and later confirmed by the embankment performance - more than justified the special provision of the sampling and testing equipment for this project.

It was estimated that the 5 m embankment would take approximately 7 years to achieve 90% consolidation (see Figure 5). Alternatively, that after 2 years, only about 70% of consolidation will have taken place, so that about 300 mm of settlement would occur subsequent to starting the river bridge construction, and a further 200 mm after opening the road. Despite the advantage of the proposed temporary surfacing, this residual amount of settlement was considered to be excessive at the river bridge.

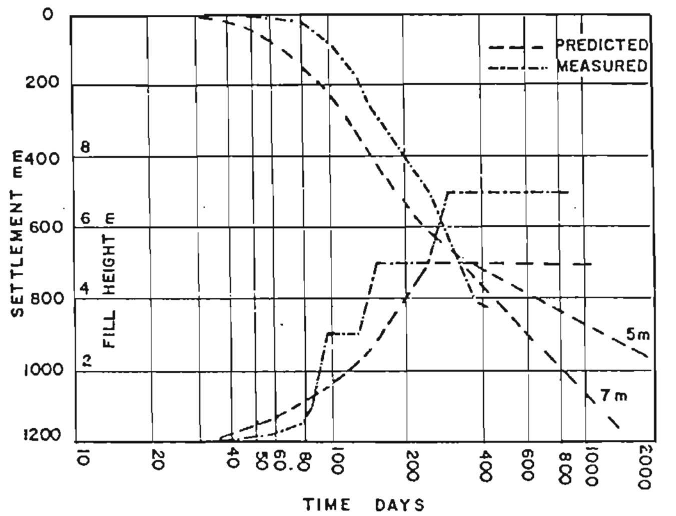

Fig.5 Predicted and Measured Settlements

At the design stage it was therefore decided to make provision for the option of surcharging the centre section by adding about 2 m of fill. If the surcharge was used then after 2 years about 950 mm of settlement would have occurred - see the 7 m line on Figure 5. The surcharge could then be removed, leaving an acceptable amount of consolidation settlement - about 100 mm - to occur after completion of the bridge in a further year. It may be noted however, that without recourse to subsoil drainage systems, not enough consolidation could be achieved before piling, for negative skin friction to be discounted.

MONITORING

It was clear from the various analyses, that despite considerable care in obtaining the basic data, a high level of confidence could not be placed on the rate of settlement predictions due to the variability in c_v's.

By implication it was necessary to monitor the performace of the embankment to control the rate of construction and also so that should the settlement be slower than predicted or should anything unexpected occur, appropriate steps could be taken.

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The instrumentation was confined to three lines across the embankment (see Figure 1). Two of the lines were adjacent to the river bridge position and placed so that they would not be affected by the removal of fill and construction of the bridge. The third line was about 100 m south of the bridge where the embankment height increased to about 6 m.

The instruments consisted of six inclinometers taken into the shale bedrock, i.e. one at the end of each line, at the toes of the embankment; five pneumatic remote reading mercury head settlement sensors placed symmetrically in each line about 0.5 m below initial ground level and seventeen pneumatic piezometers were installed in the three lines at depths of 3 m and 8 m; the instrument leads were taken to brick built gauges houses at the eastern end of each line. Fifteen standpipe piezometers were installed later, five in each of three boreholes (as shown in Figure 6). Typical results from the instruments are shown in Figures 5, 6 and 7.

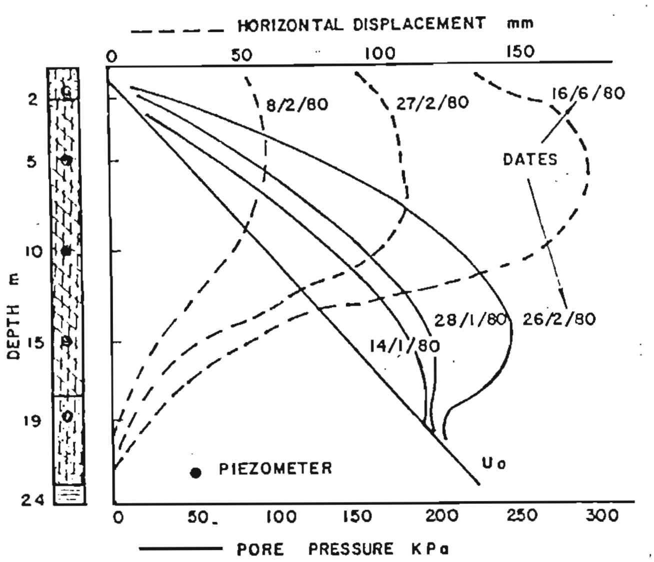

Fig.6 Typical Piezometer and Inclinometer Results

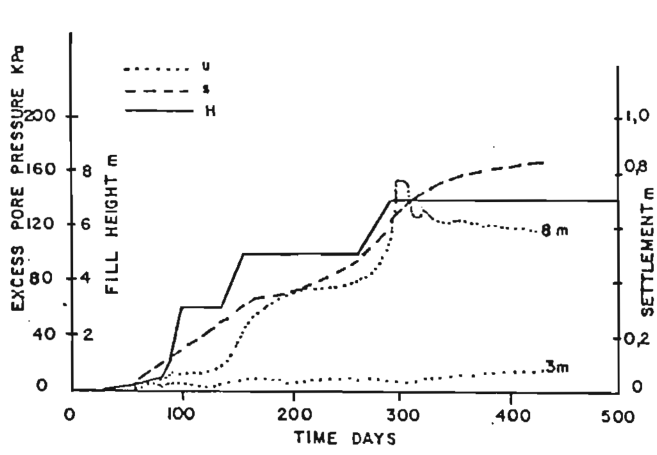

Fig.7 Pore Pressure Response

DISCUSSION

Two distinct steps occurred during construction (as shown in Figure 7). The first of these was quite short, at a height of about 3 m, and was due to adverse weather and the requirement to use the earth moving equipment elsewhere. The second step, at about 5 m height, was primarily for geotechnical reasons.

The typical result (see Figure 7), shows that the rise of excess pore pressure in the clay at 8 m depth, follows immediately on loading; that it is practically 100% of the additional stress and furthermore, that no dissipation was discernible. In contrast, the excess pore pressure at 3 m depth, in the upper sand layer, was small and reflected only the settlement of the piezometers. Settlement followed the loading quite closely and, with the inclinometers, gave expected values of deformation. These deformations were plotted as recommended by Matsuo and Kawamura (1977), i.e. vertical deformation against the ratio of horizontal to vertical deformation.

These deformation measurements gave no cause for alarm, yet according to the initial single layer stability charts (Figure 4), the factor of safety was below the construction limit of 1.2. This was unexpected, and unlike the performance of a similar embankment on the same project, which was constructed first because more severe problems had been expected (Jones et al, 1980).

In view of these developments the situation was re-analysed. The later stability analysis used a two layer subsoil model to take cognizance of the undoubted benefit of the upper sand layer. The distribution of excess pore pressures under the embankment was modified from a simple system into a multi layered - both vertically and horizontally - model to fit the measured response.

Modified stability charts were drawn up, and one of these, for a fill height of 5 m, is shown on Figure 4. The modified charts are less conservative and presumably more realistic. For example, for a typical embankment say 5 m high, a u_e in the clay of 60 kPa, and in the sand of 10 kPa, gives a factor of safety of 1.23 compared with 1.08 if the single layer chart is used with u_e of 60 kPa.

The highest excess pore pressures were measured at the centre line of the embankment at a depth of about 10 m. In practice the failure surface is not likely to be as deep as this, although the circular arc analytical method does indicate that it could be up to 8 m deep. The use of pore pressures from the deeper piezometers is therefore pessimistic.

In addition to the measured data indicating that stability was a potential problem, the lack of pore pressure dissipation also suggested that the rate of settlement could be adversely affected, which would necessitate a change to the programme. It was decided to instal further instrumentation to check the performance. Fifteen standpipe piezometers were put in at various depths in the clay layer, as well as into the upper and lower sand strata. These piezometers consisted of ceramic filters attached to 10 mm diameter semi rigid black plastic pipes, with valves at the top to which a pressure gauge could be connected. These proved to be remarkably effective although most embarrassingly unsophisticated in appearance.

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Immediate response was not necessary, since these piezometers were intended both as a check on the existing pneumatic system, and also to give a more complete picture of pore pressure distribution within the subsoil. This is shown in Figure 6. It can be seen that the assumed double drainage model is valid for the clay layer. In addition to this extra instrumentation, in-situ pore pressure measurements were also later carried out using a CPT probe equipped with a piezometer, which confirmed the standpipe piezometer measurements, (Jones and van Zyl, 1981).

The measured settlements were compared with the predicted (as shown in Figure 5). This showed that the actual settlement lagged behind the predicted. It was therefore necessary to proceed with the designed surcharge as soon as possible to minimize post construction settlements.

Final design of the river bridge foundations had not been completed since the option was kept open of neglecting negative skin friction had full consolidation occurred. This was clearly not the case, and the piles had to be designed to include negative skin friction which, for the 0.9 m piles was about 15% of the working load. The problem of lateral loads on the piles due to the embankment was examined using the semi-emphirical method described by de Beer and Wallays, (1972).

A major consideration is the presence of the sand layer in the upper 3 m to 4 m. If this can be considered as dense, then the piles may be modelled as being fixed at both ends; on the other hand, if the layer is loose, then the piles must be taken as free at the top and this dramatically increases the bending moments. However, because the inclinometers showed significant lateral displacements in the upper sand (as shown in Figure 6), and also since excavation for the new river channel would remove most of the sand layer in front of the abutments, the piles were considered as fixed only at the bottom. This resulted in excessive bending moments, so a system of near horizontal anchoring of the pile cap has been designed using cables through the fill, attached to deadman concrete anchor blocks cast in the fill, some 30 m behind the abutments. A monitoring system consisting of measuring the anchor cable stresses, the movements of the pile cap and anchor block, and soil deformations by inclinometers close to the piles, will be installed.

CONCLUSIONS

This case history demonstrates the value of adopting a philosophy of flexibility both in the design and construction in problematical areas.

The investigation showed that there was a reasonable probability that the embankment could be built without the installation of extensive subsoil drainage measures. Since forward planning had provided some scheduling options, and since the temporary pavement link to the existing road permitted some post construction settlement, it was decided to proceed without subsoil improvement other than surcharging at the bridge.

ACKNOWLEDGEMENTS

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(vii) MINE TAILINGS CHARACTERIZATION BY PIEZOMETER CONE

MINE TAILINGS CHARACTERIZATION BY PIEZOMETER CONE

GARY A. JONES*, DIRK VAN ZYL, AM.ASCE** AND EBEN RUST*

ABSTRACT

The piezometer cone is particularly useful for the characterization of different soils. The typically layered nature of mine tailings impoundments lends itself to the use of the piezometer cone for the identification of the various layers. This paper describes some of the details of a piezometer cone and its application in mine tailings materials. Results of field testing in tailings impoundments from gold and platinum mines are presented and discussed. It is shown that the results can be used to identify layering and thereby the spatial variation of geotechnical parameters.

The possibility of characterizing mine tailings materials (and granular soil in general) by using indices based on pore pressure measurement during penetration is discussed. Finally, it is shown how the piezometer cone can be used as a means of monitoring the stability of a tailings impoundment throughout its operating life.

Introduction

The characterization of soil materials through relatively inexpensive in situ testing is one of the geotechnical engineer's dreams. Many attempts have been made in the past to develop and perfect instrumentation for this purpose, e.g. the Standard Penetration Test (SPT), the Dutch cone (CPT) with friction sleeve (Begemann, 1956) and the screwplate (Janbu and Senneset, 1973). The Dutch cone has been used with considerable success in Europe for the characterization of soil deposits through the identification of soil types, the estimation of undrained shear strength of clays, as well as friction angle and compressibility of sands, and the prediction of pile capacities.

The development of the piezometer cone into a useful soil investigation tool has brought new possibilities for soil characterization through continuous testing (Tortensson, 1975; Wissa, et al., 1975; Parez, et al., 1976; Peignaud, 1979; Baligh, et al., 1980; Campanella and Robertson, 1981; Gillespie and Campanella, 1981; Jones and van Zyl, 1981). Most of the studies reported in the literature have been conducted on natural materials. However, the present paper describes piezometer cone testing in tailings material and the particular concerns of such testing.

Various soil indices are defined and their usefulness is investigated. Finally, some comments are made about the use of the piezometer cone as a monitoring instrument.

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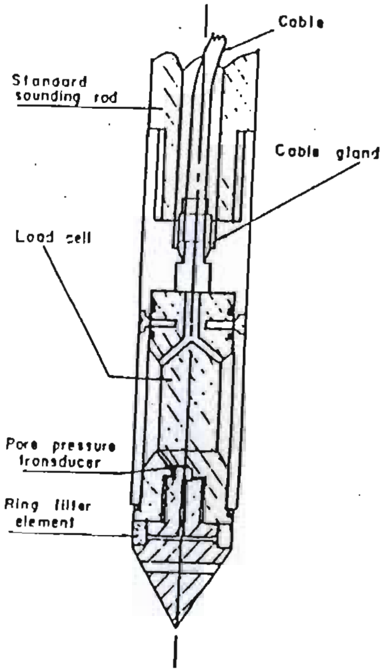


igure 1. Details of Piezometer Cone

Piezometer Cone

The piezometer cone used in this study is described in detail by Jones and van Zyl (1981). Figure 1 shows the general arrangement of the piezometer cone. The cone used is the conventional shape of 35.7 mm diameter (10 cm^2 area) with 60° apex angle. The cone pressure is measured by an electric resistance strain gauge load cell, while the pore pressure is measured by a pressure transducer. The chamber for the latter is filled with de-aired water and connected to a ring filter element (porous stone) located directly behind the cone tip. Glycerin has been successfully used as a pore pressure chamber fluid (Campanella and Robertson, 1981). The electrical outputs are recorded on a chart recorder, which is controlled as required: either by the depth penetrated or by time and chart speed. A data logger system is currently being fitted which will facilitate computer processing of the test results. Reaction for the conventional 100 kN Goudsche Machinefabriek probe is provided by short screw auger anchors. The rate of penetration is maintained at the standard 20 mm/sec. The effect of penetration rate on the test results is discussed later.

Care was taken at the beginning of each test to de-air the porous :one and to saturate the pore pressure chamber.

me Characteristics of the Tailings Deposits Investigated

Tailings impoundments constructed by hydraulic filling techniques e typically layered due to the construction process. Some of the yering can also be a result of changes in the ore body being mined, changes in the milling process.

During hydraulic filling of upstream constructed tailings im- mdments, the slurry is deposited through point discharge or spigot scharge. In the first case, a concentrated slurry stream exits the ivery pipe and distributes over the already deposited tailings to m a beach. Spigot discharge is done through regularly spaced out- s from a tailings delivery line on the crest of the tailings im- mdment. Sedimentation of tailings takes place along the beach with coarsest material settling close to the discharge point and the er material further along the beach. The finest material is

usually deposited in the pool area at the end of the beach. Considerable layering usually results from hydraulic filling. The layers, which are not necessarily continuous, determine the spatial variation of material characteristics such as shear strength and permeability. The pore pressure distribution in a tailings impoundment is also strongly influenced by the layering. Figure 2 shows a typical cross-section of a tailings impoundment constructed by the upstream technique.

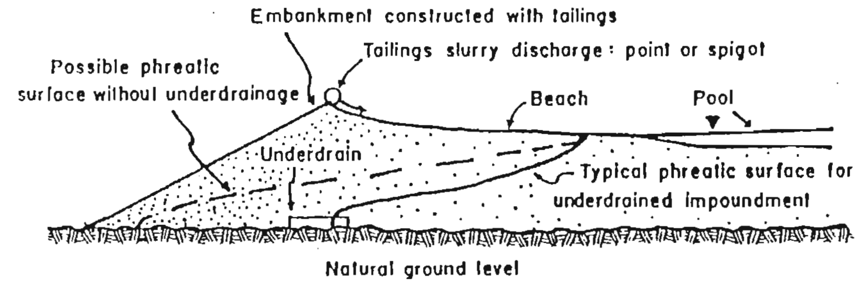


Figure 2. Typical Section of Upstream Constructed Tailings Impoundment

The field testing described in this paper was performed on platinum and gold tailings. The platinum tailings are deposited through a spigot system, while the gold tailings are distributed through a point discharge and paddock system (a description of this system is given by Ruhmer, 1974). Typical grading curves of these tailings materials are presented in Figures 3 and 4. The gold and platinum tailings are granular materials. There are differences in the characteristics of the tailings material forming the various layers and this is reflected in the grading envelopes shown in Figures 3 and 4.

Purpose of Testing

The purpose of the testing program is to provide information for the modeling of the geotechnical characteristics of the impoundment and to use this information in evaluating the stability of the structure. Information on the spatial variation of the geotechnical parameters may also be used in the probabilistic modeling of the impoundment. One considerable advantage of the electric probe is that it permits continuous testing with depth.

The testing described here was performed for four main reasons:

(i) Identification of layering
(ii) Measurement of in situ (excess) pore pressures
(iii) Estimation of consolidation parameters through pore pressure dissipation tests
(iv) Estimation of effective strength parameters and relative density.

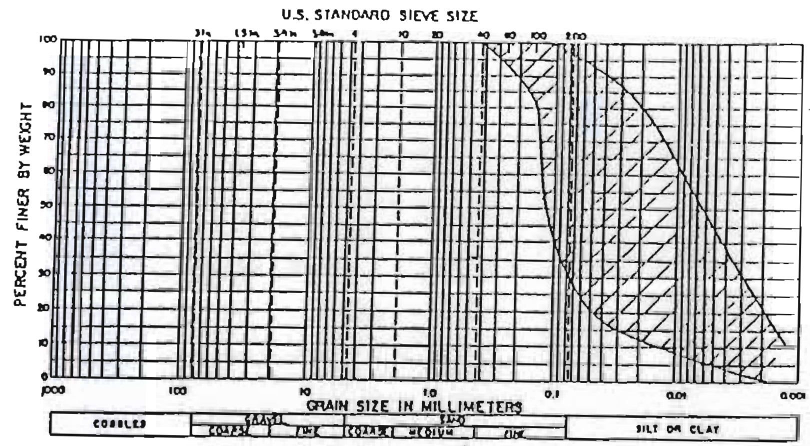


Figure 3. Grading Envelope for Gold Tailings

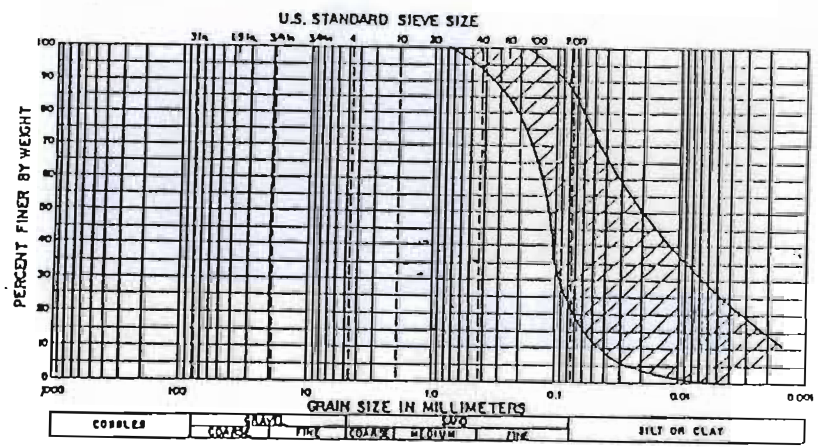


Figure 4. Grading Envelope for Platinum Tailings.

This paper will discuss the test results obtained from typical tests in gold and platinum tailings. It is shown how these results are interpreted and the concerns with such interpretations.

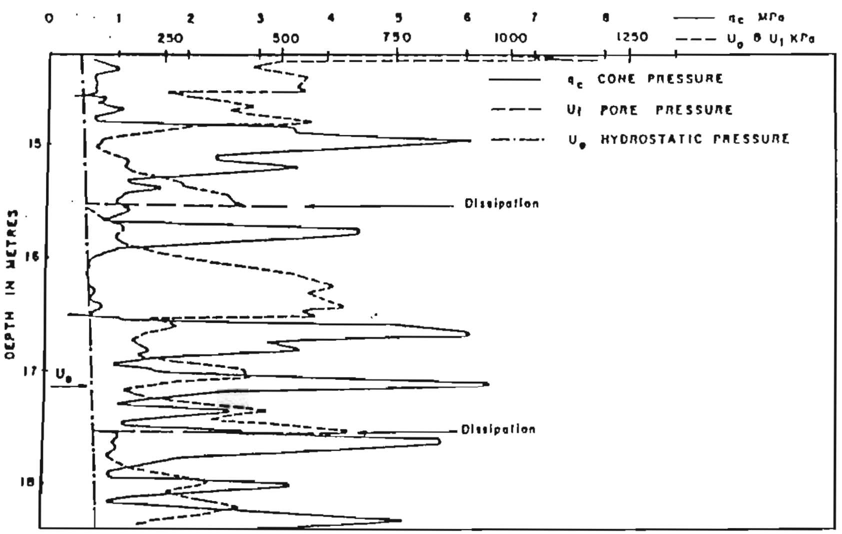
Typical Test Results

Portions of the pore pressure and cone pressure profiles obtained from the testing of gold and platinum are presented in Figures 5 and 6. The following general observations can be made from these results.

(i) There is a marked relationship between cone pressure and pore pressure response, i.e. higher point resistances and lower pore pressures occur at the same depth while lower point resistances are observed with higher pore pressures (e.g. 16 to 16.5 m, Figure 5). This is assumed to indicate the layering of the material since, during penetration, the denser coarse material tends to dilate, resulting in a decrease in pore pressure, whereas a positive pore pressure is caused in the less dense finer material. It may be noted that layer thicknesses in Figure 6 average about 0.25m but individual layers of about 0.05m and less can be readily detected.

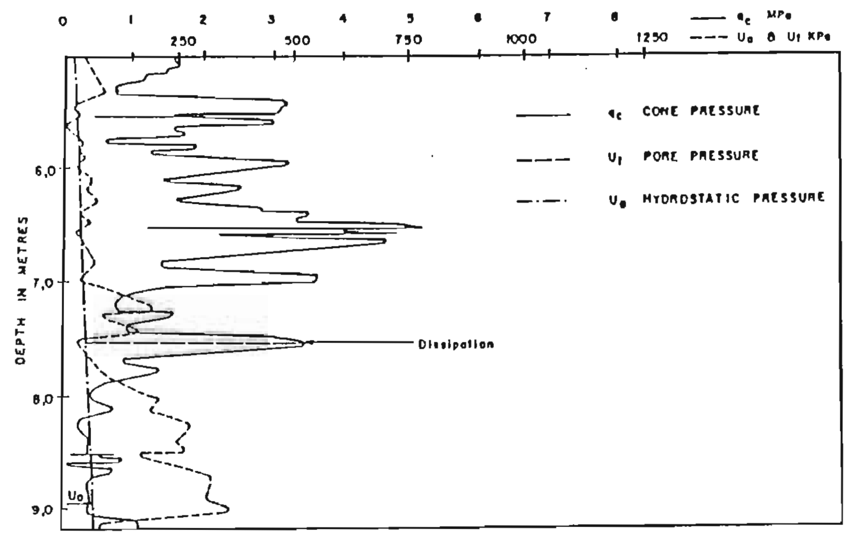
(ii) Complete saturation of the pore pressure system is of concern in interpreting piezometer cone data. It is difficult to identify a 'soft' system from testing results in variable material such as tailings. There may be some indications in Figures 5 and 6 that the cone was not fully saturated; note slow response between 17.5 and 18.0m in Figure 5, as well as the apparent smaller and slow response of pore pressure in Figure 6. Extrapolating the hydrostatis pressures in Figures 5 and 6, water table depths of about 6.4m and 3.2m are obtained respectively. These depths were therefore penetrated before any positive pore pressures were measured. It is possible that some desaturation could take place. The correct combination of porous element air entrainment value and cone fluid, such as glycerin, controls the saturation of the cone when penetrating unsaturated materials. From other testing performed on tailings by the authors, the results in Figures 5 and 6 are considered representative of testing in tailings under fully saturated conditions. The further interpretation of the data will therefore assume saturated conditions.

(iii) During probing it is necessary to stop after each meter to add a further rod. This stop takes about one minute and during the period, the generated excess pore pressure almost completely dissipates. These positions are indicated by horizontal lines e.g. at 15.5m and 17.5m in Figure 5. At these, or at other positions, complete dissipation is sometimes allowed so that the in situ pore pressure can be measured.



Depth (m) (1)	Cone Pressure q_c (kPa) (2)	Hydr Pressure U_o (kPa) (3)	Total Pore Pr. U_t (kPa) (4)	Total Vert.Pr. $\bar{\sigma}_{vo}$ (kPa) (5)	Soil Type (6)	ϕ_o (degrees) (7)	ϕ'_o (degrees) (8)	Rel. Density D_r (%) (9)
14.0	600	78	330	211	Clay	14	23	0
15.0	6,100	87	100	227	Sand	33	34	47
15.8	4,400	93	150	240	Silty Sand	30	33	34
16.4	600	99	650	250	Clay	13	25	0
16.6	6,000	100	180	253	Silty Sand	32	35	44
15.9	1,000	103	450	258	Clay	17	26	0
17.1	6,300	106	156	262	Silty Sand	33	34	45

Figure 5. Piezometer Cone Results for Gold Tailings.



Depth (m) (1)	Cone Pressure q_c (kPa) (2)	Hydr Pressure U_o (kPa) (3)	Total Pore Pr. U_t (kPa) (4)	Total Vert.Pr. σ_{vo} (kPa) (5)	Soil Type (6)	ϕ_o (degrees) (7)	ϕ'_o (degrees) (8)	Rel. Density D_r (%) (9)
5.4	3,167	22.7	23	79	Sand	34	34	46
5.7	667	25.7	43	84	Clayey Sand	23	24	0
5.9	3,000	27.7	33	87	Sand	33	35	43
6.2	1,500	30.7	67	92	Clayey Sand	29	32	19
6.5	5,000	33.7	23	97	Sand	37	36	59
7.2	667	40.7	200	109	Silty Clay	21	30	0
7.4	833	42.7	176	112	Silty Clay	24	30	0
7.5	3,300	43.7	30	114	Sand	33	33	42
8.3	333	51.7	283	128	Clay	14	25	0

Figure 6. Piezometer Cone Results for Platinum Tailings.

Interpretation of Test Results

Figures 5 and 6 show interpretations of the piezometer cone test results at selected depths. The first five columns in the tables give depth from surface, cone pressure (q_c), hydrostatic pressure (u_o), total measured pore pressure (u_t) and total overburden pressure (σ_{vo}) taking dry unit weight at 13.3 kN/m^3 (85 lb/ft^3) and saturated unit weight at 16.5 kN/m^3 (105 lb/ft^3). The methods used for interpreting the results are discussed in this section. Also discussed are the concerns with these interpretations.

(1) Material Identification

Various methods have been proposed for the identification of materials from cone penetration test results. Begemann (1965) introduced a semi-empirical method based on friction ratios which has been shown to be a very reliable system. However, because of the small thickness of the layering in tailings impoundments (usually less than 50 mm), friction ratio is not a practical way of identifying tailings materials since the friction sleeve is itself 134 mm long.

Baligh, et al. (1980) suggested the use of a u_t/q_c ratio as a materials identification parameter and showed that such a ratio is useful in the identification of clay layers with different overconsolidation ratios. It was also concluded that u_t/q_c decreases with decreased cone angle of the penetrometer. Jones and van Zyl (1981) suggested that indices based on the excess pore pressure (u_e) may be a more sensitive measure. Note that $u_t = u_o + u_e$, where u_e can be positive or negative.

Three different soil indices will therefore be examined for the identification of materials:

$$I_{s1} = u_t/q_c \qquad (1)$$

$$I_{s2} = \frac{u_t - u_o}{u_o} \qquad (2)$$

$$I_{s3} = \frac{(u_t - u_o)/u_o}{(q_c - \sigma_{vo})/\sigma_{vo}} \qquad (3)$$

In order to compare these indices, they are plotted against cone pressure in Figure 7 to 9. The data points in these figures are from test results in gold and platinum tailings below the water table.

It can be seen that Figures 7 to 9 are all similar and take the form of fairly narrow hyperbolic bands for the tailings material tested, and there would appear to be no marked advantage in any one of the plotted indices. As previously mentioned, it is believed that the use of u_e, which may be positive or negative, has the advantage of indicating materials which are dilatant, but has the disadvantage that it is necessary to know the natural hydrostatic conditions to calculate the

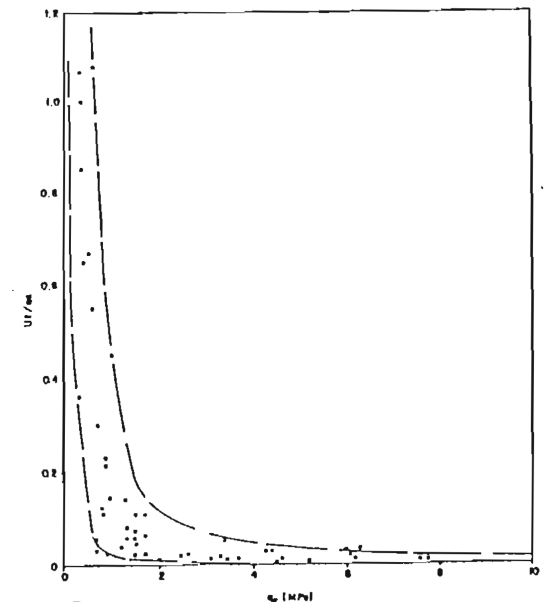


Figure 7. q_c vs. I_{s1}

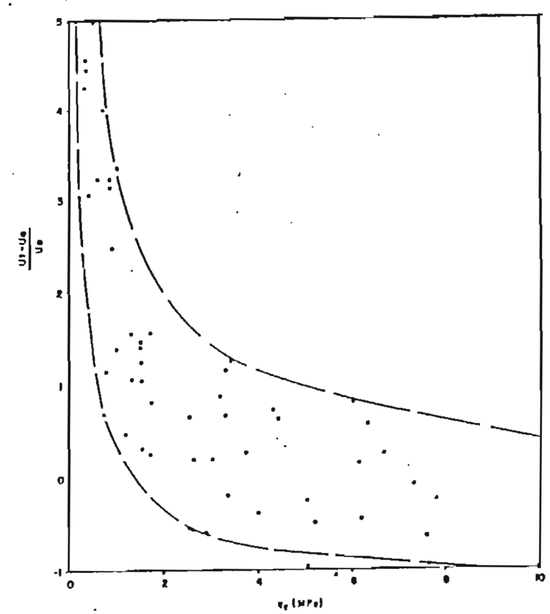


Figure 8. q_c vs. I_{s2}

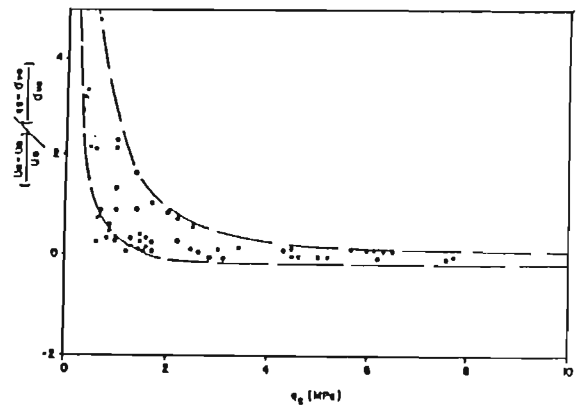


Figure 9. q_c vs. I_{s3}

change in pore pressure. The hydrostatic pressure can be obtained with the piezometer cone, as discussed later.

Although there are differences in the band width in Figures 7 to 9, all three indices are apparently fairly insensitive. From the test results in Figures 5 and 6, it is apparent that the fine materials will plot along the vertical part of the bandwidth (high pore pressure and low strength), while coarse materials will plot along the horizontal leg. Figures 8 and 9 show some negative values of the pore pressure factors due to the effects of the coarser dilating materials.

To further examine the identification of material types with the piezometer cone, a plot of $(u_t - u_o)/u_o$ vs. $(q_c - \sigma_{vo})/\sigma_{vo}$ was compiled as presented in Figure 10. This contains the results from gold tailings of two different mines, platinum tailings and natural materials (varying from soft, highly plastic clays to dense medium grained sands). The cone results (q_c) in the tailings, indicate marked highs and lows (refer to Figures 5 and 6). The high cone readings have been distinguished on Figure 10 by a diagonal stroke through the site symbol. These values plot generally along the cone pressure axis.

It is evident from Figure 10 that highly plastic, soft clays plot on a band close to the pore pressure axis, whereas the dense sands are close to or below the cone pressure axis, and materials intermediate between clays and sands seem to fall rationally between these.

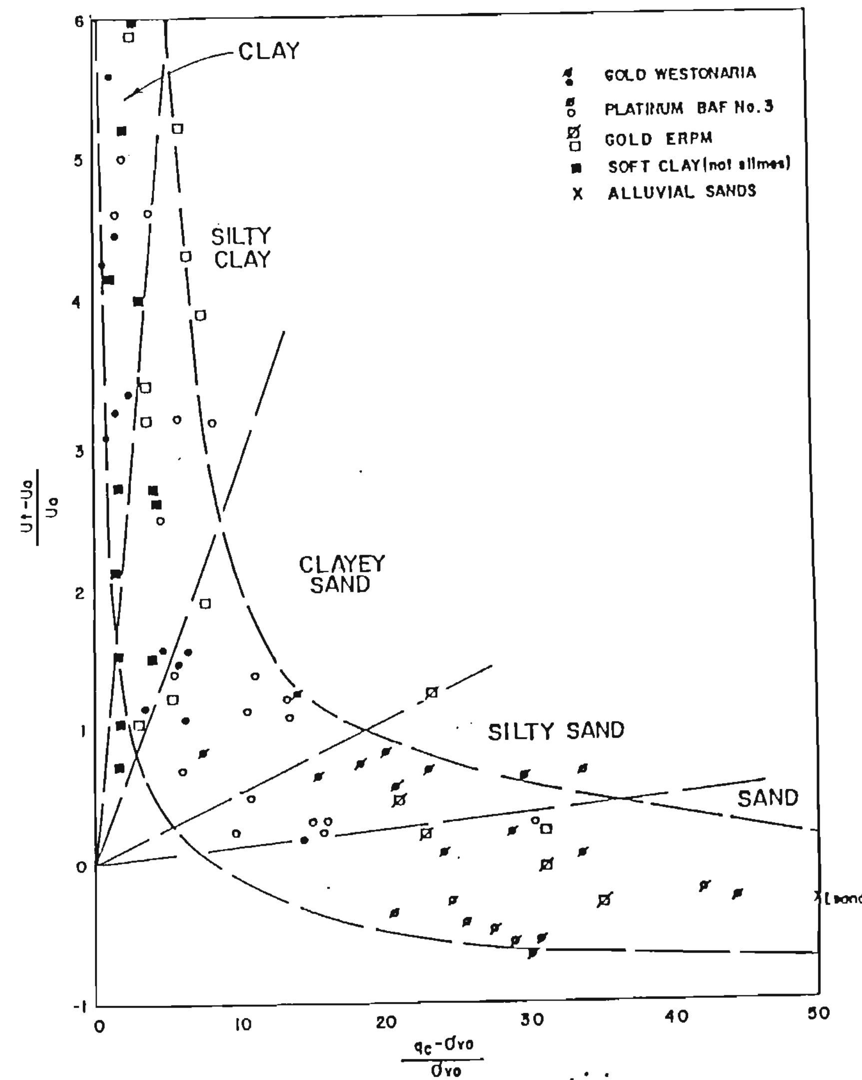


Figure 10. Soil Identification from Piezometer Cone Results for Normally Consolidated Saturated Material

Generally it is confirmation that higher cone resistances together with smaller pore pressure responses are expected in sands, as compared to soft, normally consolidated clays, where small cone pressures and high pore pressure responses would be anticipated. The fairly narrow band of results, with comparatively small overlap of different materials within the band, suggests that this plot can usefully be developed as an indication of the effective grain size of the material.

It is tentatively suggested that Figure 10 can be used in the form presented to identify normally consolidated saturated materials from testing with a piezometer cone having a 35.7 mm, 60° tip. Sand materials are larger than the no. 200 sieve, while clay is smaller than 0.002 mm (refer to Figures 3 and 4). It must be emphasized that the divisions shown in Figure 10 are tentative. These divisions can be refined by well controlled laboratory testing.

The absolute values of the parameters in Figure 10 are strongly dependent on the measured total pore pressure and cone resistance, so that it is a function of the shape of the cone, the position of the filter, and the rate of penetration.

Figure 10 was used to interpret the data in Figures 5 and 6, the resulting soil types are given in Column 6. As expected, considerable variation of soil types is obtained.

(ii) <u>Pore Pressure Measurement</u>

Figures 5 and 6 demonstrate that the cone pressure and pore pressure readings change very rapidly. A careful analysis of the field recorded charts indicates a slight offset of the peaks and troughs (in cone pressure and pore pressure readings). The pore pressure response seems to lag approximately 40 mm behind the cone pressure reading. The center of the ring filter element is about 40 mm higher than the cone tip (refer to Figure 1) so that, superficially at least, it would appear that the response lag is reasonable. The inclusion of an electronic data logger system should help in the investigation of this effect. It should be possible to investigate this problem by laboratory testing of a carefully prepared layered sample.

There has been considerable speculation regarding the optimum position of the filter element. Torstensson (1975) demonstrated that this has a marked effect on the magnitude of the pore pressure measured during penetration. Parez, et al., (1976), however, observed no such differences. However, these results were comparing relatively close positioning of the filters. Jones and van Zyl (1981) also found no differences with similar close filter positioning and experienced frequent blocking of the face mounted filters in soft clays, which made the face position impractical.

Levadoux and Baligh (1980), present a theoretical analysis of pore pressure generated during penetration. Figures 11(a) and 11(b) show the theoretical pore pressure distribution (normalized with respect to effective overburden pressure) and the normalized pore pressure variation along the tip of a 60° cone. The results in Figure 11(b) indicate that the maximum excess pore pressure occurs along the tip of the cone.

From a practical point of view, it is better to measure the pore pressure just above the tip because of problems with smear and damage of filter elements.

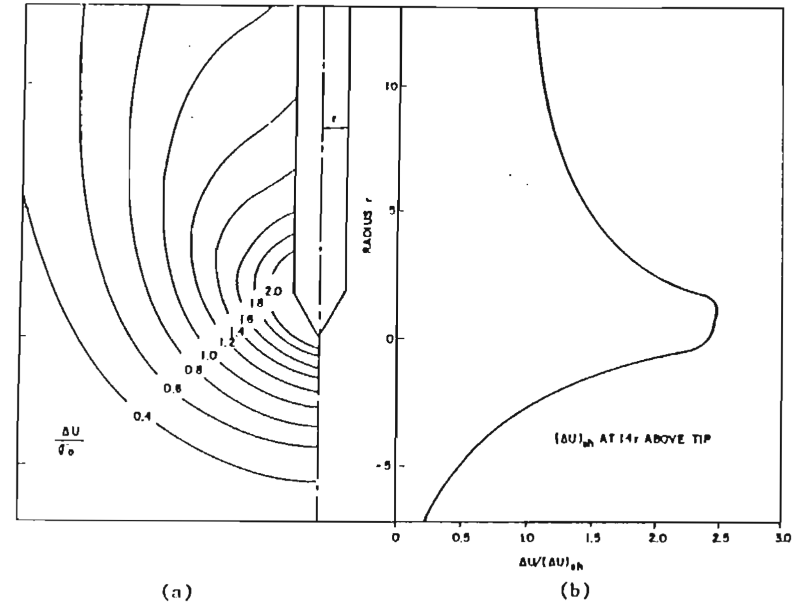


Figure 11. Theoretical Pore Pressure Distribution for Probe Testing (60° cone)(from Levadoux and Baligh, 1980)

It is essential that some standardized form of probe be developed before a meaningful comparison of material identification indices, based on generated pore pressures, can take place. It is suggested that the design shown in Figure 1 is the most suitable for practical use based on the previously stated reasons. It also conforms to the recommended shape for a Dutch cone given in the ISSMFE standard (1977).

a. <u>Measurement of Hydrostatic Conditions</u>

In the investigation of tailings impoundments it is necessary but often difficult, to establish the pore pressure conditions which influence the stability. This is due to the complex layering system which may result in numerous perched water tables. Furthermore, flow is near-horizontal in the pool area and becomes drawn down at the embankment (see Figure 2). Many fixed piezometers would be required to monitor these conditions completely, and, because of the thin layers, it would be necessary to ensure that any piezometer only measure the pressure within one layer. In practice this is very difficult to achieve.

However, the piezometer cone can measure the pore pressure at any point by simply stopping penetration and allowing dissipation, even within thin layers since the filter element has been deliberately reduced to a width of 5 mm.

Figure 12 shows pore pressure measurements taken in this manner at the edge and at the center of tailings impoundments with the water tables (measured with standpipe piezometers) superimposed for direct comparison. It can be seen that at the center the measured pore pressures in meters of water (shown dashed) increases in exact correspondence with increased depth which conforms to uniformly horizontal or zero flow conditions. Close to the embankment, however, the measured pore pressures increase with depth at a lower rate -- indicated by a solid line in Figure 12. It is believed that this is a reflection of the changed flow conditions at the embankment which can be shown by an appropriate flow net (refer to Figure 2).

Since it is the pore pressure conditions close to the embankment that control the stability of the dam, it is believed that the piezometer cone will be invaluable in the assessment of existing impoundment stability.

b. <u>Pore Pressure Dissipation Following Penetration</u>
Typical curves of pore pressure dissipation in soft clays following penetration were presented by Jones and van Zyl (1981). Figures 5 and 6 indicate that pore pressure dissipation takes place each time a rod is added (e.g. at 15.5 m and 17.5 m in Figure 5).

In the testing of clays it has been observed that the excess pore pressure actually increases after penetration has stopped, after which dissipation takes place (Schmertmann, 1975; Torstensson, 1975; Peignaud, 1979; Gillespie and Campanella, 1981; Jones and van Zyl, 1981). This effect was not observed in tailings materials. Gillespie and Campanella (1981) attribute this effect to an unsaturated cone. However, it seems that it is also a function of the permeability of the material tested.

The time dependent pore pressure dissipation results are similar, superficially at least, to laboratory consolidation test data (Torstensson, 1975; Jones and van Zyl, 1981). The highest pore pressure at the beginning of dissipation and the hydrostatic pore pressure are known. A plot of the results can therefore be used to obtain t_{50}, t_{90}, etc.

Gillespie and Campanella (1981) report considerable success in the interpretation of consolidation characteristics from dissipation test results by using expanding cavity theory. A more direct interpretation can be to use the theory in the Appendix to this paper. Figure A.2 presents the time factor, T, for consolidation of a spherical body. However, the value

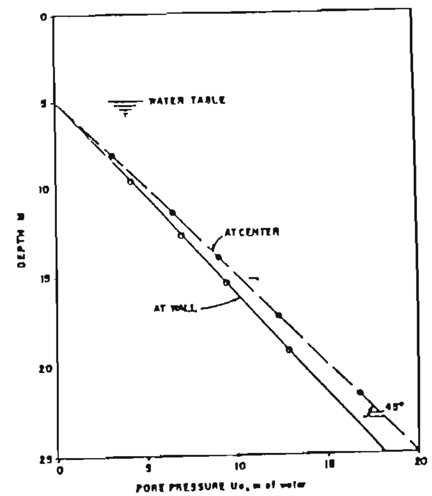


Figure 12. Pore Pressure Variation with Depth and Measurement Location

of a, the radius of the sphere, must be assumed. No data is available at this point for comparison of these methods.

c. <u>Influence of Penetration Rate</u>
The penetration rate influences the magnitude of the cone pressure as well as the pore pressure generated during testing (Parez, et al., 1976; Baligh, et al., 1980; Campanella and Robertson, 1981). In order to measure drained conditions it would be necessary to reduce the penetration rate so that no excess pore pressure is generated. The question is what penetration rate would satisfy this condition.

The allowable penetration rate can be estimated by using the model in the Appendix. Although this approach is simplified, the conclusions are useful. The results obtained from the analysis in the Appendix indicates that the testing rates presently used cannot lead to the measurement of fully-drained conditions. However, it is impractical to use penetration rates as low as those calculated. For the present reliance must therefore be placed on empirical corrections for the estimation of fully-drained parameters from piezometer cone testing data.

(iii) <u>Effective Strength Parameters</u>
The tailings materials tested can be assumed to have a c' parameter of zero. Harr (1977) used the general expression for the ultimate bearing capacity of a footing to estimate the operative angle of friction. By assuming that the width of the penetrometer is negligible in comparison to the testing depth, the following relationship between cone pressure (q_c), effective vertical stress (σ'_{vo})

based on hydrostatic conditions and operative angle of friction (ϕ_o) is obtained:

$$q_c/\sigma'_{vo} = (1 + \tan \phi_o) \tan^2(45 + \phi_o/2) \exp (\pi \tan \phi_o) \qquad (4)$$

This expression can conveniently be plotted and is given in Figure 13.

It was observed by previous workers (Parez, et al., 1976; Baligh, et al., 1980), that the cone pressure is dependent on the penetration rate. Or stated differently, the excess pore pressure generated (and other factors contributing) during penetration, affects the measured cone reading. Baligh, et al. (1980) suggested a linear relationship between q_c and penetration rate, q_c increasing with increasing penetration rate. Parez, et al. (1976) suggested the following relationship:

$$\Delta q_c = \Delta u \, (N_q - 1) \qquad (5)$$

for granular materials, where N_q is the classical bearing capacity factor. The same factor was used in equation 4, and therefore Figure 13. The vertical axis of Figure 13 therefore plots N_q.* It must be noted that $\Delta u = u_e$, the excess pore pressure, so that equation 5 can be written as:

$$\Delta q_c = u_e(q_c/\sigma'_{vo} - 1) \qquad (6)$$

The effective operative angle of friction is then calculated from equation 4 by increasing q_c with Δq_c when u_e is positive and by decreasing q_c when u_e is negative.

A useful way of presenting the data is to plot ϕ_o and ϕ' vs I_{S3} (equation 3). Correlations can then be established from such plots for obtaining ϕ'_o.

Results of ϕ_o and ϕ'_o are given in Figures 5 and 6 in columns 7 and 8 respectively. The ϕ'_o values correlate generally well with the soil types, e.g. in Figure 5 sand and silty sand have $\phi'_o \simeq 34$, while for clay $\phi'_o \simeq 25^o$.

The ϕ'_o values obtained from the piezometer cone results for materials described as sand compare well with laboratory test results on gold and platinum tailings which are:

gold $\phi' = 34^o$

platinum $\phi' = 37^o$

The selection of an effective angle of friction for stability analyses depends on the relative frequency of occurrence of the various layers. Engineering judgment would be used for the selection of a value for deterministic analyses. The results obviously lend itself well to probabilistic analyses. A discussion of the methods used in estimating the variability of effective angle of friction for such analyses from piezometer cone results is beyond the scope of this paper.

*Parez, et al. (1976) presents a plot of ϕ vs (n_q-1) which differs slightly from Figure 13. The difference is in the expression used for N_q in the preparation of these figures.

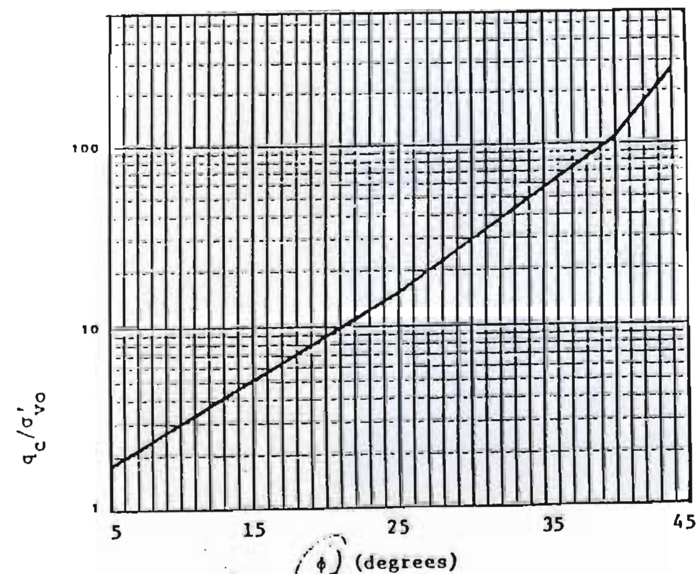


Figure 13. Graphical Representation of Equation 4.

(iv) Relative Density

After extensive sophisticated laboratory testing, Schmertmann (1978) found an empirical relationship between cone pressure (q_c), vertical effective stress (σ'_{vo}), and relative density (D_r), which "applies to normally consolidated uncemented, primarily quartz, geologically recent, saturated, fine SP sands in situ, when using the Fugrotype, 10 cm^2, 60^o, cylindrical tip, advanced continuously at 2 cm/sec." For q_c and σ'_{vo} in kgf/cm^2 ($\simeq$ 100 kPa):

$$D_r\% = \frac{100}{2.91} \ln (q_c/12.31\,\sigma'^{\,0.71}_{vo}) \qquad (7)$$

The values of relative density calculated from equation 7 are given in column 9 of Figures 5 and 6. Negative values were obtained where $D_r = 0$ is given. The relative density values are in general very low and compare with other results obtained by the authors in tailings impoundments from Dutch cone and piezometer cone testing. This is of concern because the relative density is a very important parameter in assessing the seismic stability of tailings impoundments. This specific area requires further work before definitive conclusions can be reached about relative density values obtained from equation 7.

The Piezometer Cone as a Monitoring Instrument

The piezometer cone can be used rather inexpensively to obtain geotechnical parameters from in situ testing. Interpretation of the test results can be done by using the methods described in this paper. Undoubtedly these methods will be refined in the future.

The construction of tailings impoundments continues during the mine's operational life. There are, therefore, opportunities to change construction procedures, if necessary, to improve the stability. The use of relatively inexpensive in situ testing procedures, such as piezometer cone testing, holds considerable promise in the monitoring and design of such improvements.

Piezometer cone testing results can be used as the basis for stability analyses of tailings impoundments. Careful monitoring of pore pressure and movements of the impoundment should follow such stability investigations. Piezometer cone testing can then be repeated on an annual (or longer interval) basis to re-evaluate the stability as well as the effects of changes in construction procedures. Such monitoring will help in identifying possible trouble spots and will form a good data base for the understanding of the behavior of the tailings impoundment.

It is furthermore possible that the mine can own piezometer cone equipment for intermittent testing. Such equipment can be mounted on the back of a pick-up to ensure easy access. The piezometer cone testing holes can also be used for the installation of standpipe piezometers.

Conclusions

The piezometer cone is a very useful geotechnical site investigation tool which is now sufficiently developed for general field investigations. The piezometer cone can be used inexpensively in mine tailings impoundments to:

(i) Identify the materials in the tailings impoundment, especially the layering. Considerable knowledge about the spatial variation of material characteristics can therefore be obtained rather inexpensively.

(ii) Estimate the effective strength parameters and relative density of the tailings material.

(iii) Estimate the consolidation parameters through pore pressure dissipation tests.

Because piezometer probe testing can be conducted inexpensively (in relation to undisturbed sampling, if such sampling is possible at all), it can become a useful monitoring tool for mine tailings impoundments. Such testing can reveal trouble spots such as layers of lower strength or excess pore pressure. It can therefore assist considerably in the evaluation of tailings impoundment stability and the planning of changes in construction procedures if required.

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APPENDIX

Estimation of Penetration Rate for the Measurement of Drained Conditions

A method to estimate an allowable vane shear rate to measure fully drained shear strengths of silts, was presented by Blight (1968). The same method will be used here to estimate a penetration rate for the piezometer cone for the measurement of drained conditions.

Despite the obvious limitations of the model used here in the case of cone penetration testing (such as neglecting expansion of material around tip during penetration, etc.), it is interesting to note the conclusions of the analysis.

The following assumptions are necessary to carry out the analysis:

(i) As the penetrometer is advanced, excess pore pressure is set up within a cylinder of influence of radius a. See Figure A.1. This cylinder is effectively made up of a series of spheres of radius a. The pore pressure is assumed to be uniform within these 'spheres of influence.'

(ii) The pore pressure on the surface of the 'spheres of influence' remains equal to the hydrostatic pore pressure at all times. The surface is thus assumed to represent a drainage surface.

(iii) The pore pressure in the individual spheres of radius a is set up during penetration of the distance 2a, i.e. from the time that the cone tip 'enters' the sphere until it 'leaves' the sphere. The distance 2a should therefore be divided by the time for the required drainage to take place in order to obtain the allowable penetration rate.*

For the boundary conditions:

$$\text{when } t = o,\ u = o \text{ and} \qquad\qquad \text{when } t \geq o,\ u = o, \text{ at } r = a \qquad (A.1)$$

t can be shown that the following solution is valid for pore pressure in the sphere with radius a (Blight, 1968):

$$U = 1 - \frac{1}{T}\left[\frac{1}{8} + \frac{4}{\pi^3}\sum_{n=1}^{\infty}\frac{(-1)^n}{n^3}\sin\frac{n\pi}{2}\exp(-\pi^2 n^2 T)\right] \qquad (A.2)$$

$$\text{where the time factor, } T = \frac{c_v t_f}{a^2} \qquad (A.3)$$

and U - degree of consolidation in sphere

t_f - time for required degree of consolidation

One more assumption is necessary to solve the problem; namely, the relationship between a and the diameter of the probe. Figure A.2 presents a plot of T vs. U.

*The assumption of pore pressure set up over a penetration distance of 2a is made here to allow the analysis to be conducted. Reference to Figure 11a shows that the assumption of a sphere may not be that unreasonable.

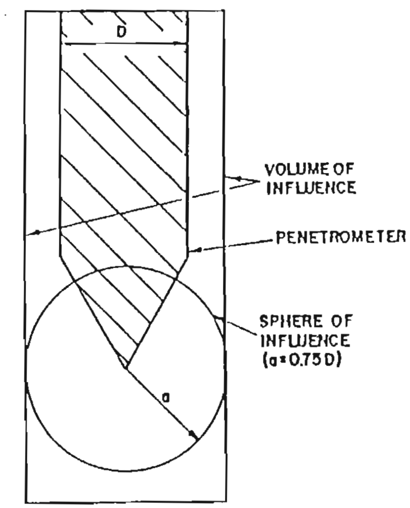

Figure A.1. Volume of Influence During Penetration

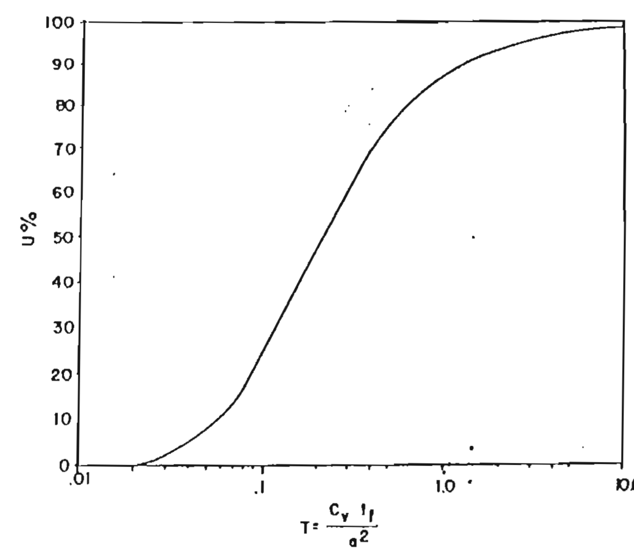

Figure A.2. T vs. U for Consolidation of a Sphere.

Example

Find the allowable penetration rate for gold tailings based on the assumptions above and for a = 0.75D, a = D, where D-diameter of piezometer cone.

Solution

From Figure A.2, for U = 90%, T = 1.30. Furthermore, from Blight (1968), for gold tailings, average laboratory d_v = 600 mm^2/min. For the cone D = 35.7 mm. From eq. A.1 for a = 0.75D, t_f = 1.6 min., which translates into an allowable penetration rate of about 35 mm/min. For a = D, a penetration rate of about 26 mm/min. is obtained. These penetration rates are much lower than the standard for the testing; namely, 1,200 mm/min.

(viii) PIEZOMETER PENETRATION TESTING CUPT

Proceedings of the Second European Symposium on Penetration Testing / Amsterdam / 24-27 May 1982

Piezometer penetration testing CUPT

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1 INTRODUCTION

Pore pressure or piezometer penetration testing (CUPT) has been in use in South Africa on a fairly routine basis for about three years.

The equipment and systems have undergone considerable changes in that period in a planned development programme intended to introduce gradually, more reliability and sophistication particularly in the data logging and processing. The methods in use are briefly described and it is interesting to note the similarity between independently developed systems mentioned by a number of authors in Cone Penetration Testing and Experience, ASCE 1981 and elsewhere. It is hoped that these similarities will allow a consensus to be readily achieved on rationalisation and standardisation of piezometer penetration testing.

The main objectives of penetration testing are to estimate strength, compressibility and consolidation parameters and to permit an adequate description of the subsoil to be made.

Some results from three sites recently investigated are given with an indication of the derived design information.

Results from these, and many other sites, are combined to enable a soil classification chart to be drawn up based on a relationship between the measured generated pore pressures and the cone pressures.

2 PIEZOMETER CONE AND SYSTEM

The piezometer cone and friction sleeve currently in use are shown in Figure 1. They comply with the European Sub Committee (1977) recommendations. The 60° cone is 35,7 mm diameter and is followed by a 150 cm^2 friction sleeve. A 5 mm wide porous plastic filter is interspersed between the cone and sleeve. A further identical sleeve serves as a housing for a load cell, which measures the cone plus sleeve load; a very similar load cell within the friction sleeve measures the cone load independently. It is of course appreciated that this sytem necessitates a decrease in sensitivity for friction sleeve measurements, as compared with an independent sleeve load measuring system; however the resolution obtainable in strain gauge load cells makes this an academic rather than practical consideration, and it is believed that the simplification in manufacturing and assembly is more than justified. Both loads cells have identical dimensions and have 8 foil strain gauge full bridges. Different load capacity probes are obtained by using load cells with identical external dimensions, but with different axial cable hole sizes. The cone load has a recess into which is cemented a Kyowa PS 10 KA (1000 kPa) pressure transducer. The transducer is miniature, measuring 5 mm diameter by 0,75 mm thick, and the chamber is the minimum practical size into which the transducer can be fitted. It may be noted that not one of these transducers has given any problem over thousands of metres of probing.

The outputs from the transducer, and the

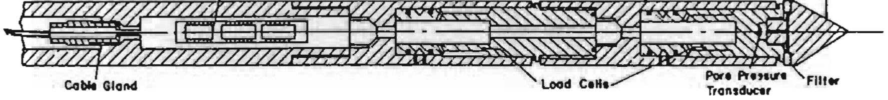

Fig. 1 Piezometer Cone

two load cells, are led to the third cylindrical section of the probe which contains voltage regulators, signal conditioners and amplifiers. It has been found that with short cable lengths, this down the hole amplifier system is unnecessary, but under more difficult conditions it has considerable advantages. The system is arranged so that in an emergency the amplifier can be easily removed or by-passed.

The overall length of the probe is arranged to be 500 mm, to simplify depth recording. The depth penetrated is measured using a rotating disc, optical linear encoder, driven by the rod movement. The encoder pulses at about $10H_z$, at the standard 20 mm/sec penetration rate, and this controls both chart recorders and a Sharp MZ80B Microcomputer. The latter has been modified to include 5 A/D converters (3 in use for probing) and gives a continuous display of cone pressure, friction ratio and pore pressure against depth. This data is recorded on tape and later processed on the same system using diskettes and a printer.

The probe is calibrated in the laboratory using a conventional triaxial loading frame and cell. The latter has been modified using a special top plate with a 36 mm dia hole with O-ring seals. The loads and pore pressures can be readily calibrated and the effect of cell pressures on the cone load measured. This effect has been discussed by Baligh et al (1981), and is larger than may be generally appreciated when high pore pressures are generated; this problem could be considered as similar to that of including the mass of inner probing rods in the mechanical probing method, i.e. the effect is measurable, but only of real significance in particular circumstances. It could of course be accommodated within the software for the data processing system.

Several authors, Campanella and Robertson (1981), Tumay et al (1981), Jones and van Zyl (1981), have noted that during dissipation tests, after ceasing penetration, pore pressures sometimes increase before dissipating. Although incomplete saturation, or lag in the measuring system could cause this, the dynamic effects referred to by Schnertmann (1974), at ESOPT I, are believed by the authors to be a more rational explanation, since the effect has been observed even after the most stringent de-airing procedures.

In the calibration system described above, it is very simple to check the relative time response of the cone and pore pressures by introducing a pressure pulse into the water filled triaxial cell. Negligible response time differences were observed, with or without the cone and filter in place, or even when no attempt was made to de-air the filter element. While certainly not advocating carelessness regarding de-airing procedures, the authors feel bound to remark that up to now they are not aware of having been confronted with problems attributable to lack of saturation even in tailings dams where the upper few metres are frequently unsaturated. It has however, as noted previously, been observed that in a multilayered soil system there is a measurable lag in the field between say a peak cone reading and an equivalent low pore pressure reading. This lag is equivalent to about 40 mm of penetration, and at this stage is believed to be due to the particular geometry of the probe, where the filter element is approximately 40 mm behind the tip.

3 FIELD RESULTS

Typical CUPT results from three sites are given below to illustrate the different uses of the technique. The sites are a waste ash dam, a mine tailings dam and a river crossing.

3.1 Waste Ash Dam

It was intended that an existing waste ash dam should be substantially raised in height, and an investigation was carried out to measure the strength parameter and the pore pressure regime, so that the dam wall stability could be analysed. The dam measures about 350 m from the ash/water discharge point to the wall, which is about 25 m high. The wall itself is constructed of coarse waste, and is fairly permeable, showing considerable seepage in the downstream face. Standpipe piezometers had been installed at an earlier stage of construction to monitor the water regime, but these had shown apparently anomolous readings.

A series of six CUPT's were carried out in a line from the discharge pipe to the standing water pool at the wall. Dissipation at stops for rod additions was very rapid, so that the ambient pore pressures were easily etablished. These are shown in Figure 2, a diagrammatic cross-section through the dam. A typical cone pressure (q_c) result (No 3) is also shown on this section. Table 1 gives the mean q_c values, together with the result of sieve analyses on samples taken at ground level at the six CUPT positions - only % passing 0,075 mm are shown.

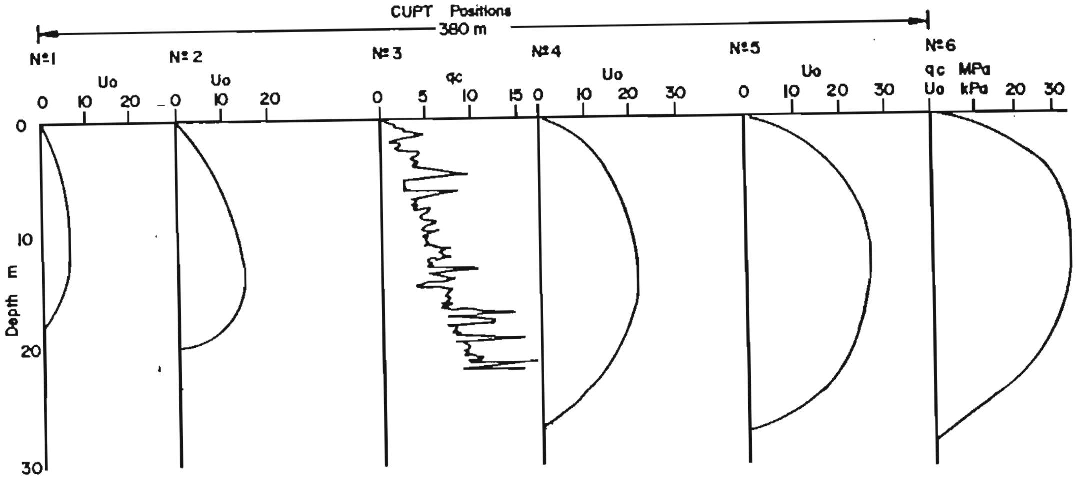

Fig. 2 Section showing pore pressure and typical cone pressure

Table 1 Summarized cone pressures and gradings

CUPT No	1	2	3	4	5	6
Cone Pressure MPa	10	7	6	4	1,5	1
% passing 0,075 mm	60	55	88	21	98	-

It would be expected that the coarsest material would be deposited close to the discharge point, and the finest nearest the wall. The above gradings generally reflect this, but the anomaly at No 4 is due to a surface local change due to the construction of an access track of coarser material. The cone pressures show a very definite trend across the dam, of higher values close to the discharge point becoming lower towards the wall.

Shear box tests were carried out on a number of samples and gave a mean $\phi = 34^o$.

Various methods of deducing ϕ and ϕ' from cone pressures were used, (Durgunoglu and Mitchell, 1975. Janbu and Senesset, 1974; Meyerhof, 1976) and as expected these gave a range of values. A method of calculating effective stress parameters suggested by Parez et al, 1976, was also used. This method takes account of the generated pore pressures and since these were small near the discharge pipe, ϕ and ϕ' are very similar. Close to the wall, where significant pore pressures were generated, calculated ϕ' become larger than the equivalent ϕ values.

The various methods of calculating ϕ, gave a range of about 30^o to 38^o; with the Meyerhof method giving the lower values and the Janbu method giving a mean value of 35^o

Despite the change in fineness of material, in all cases the dissipation times were so rapid that t_{50} dissipation times were a fraction of a second. No realistic measure or estimate of permeability was therefore possible.

The ambient pore pressures shown in Figure 2 indicate a consistent pattern. It is believed that this pattern, of low pressures near the surface reaching a peak at about mid depth and becoming low again at the base of the dam, is due to fairly free drainage through the base, so that hydrostatic conditions are never established despite a continuous inflow and a standing pool near the wall. It will be noted that the higher pressures are close to the wall where the material is finer and less permeable, and it is assumed that this overrides any draw down effects due to the comparitively free draining wall.

It may be ironically observed that despite recording rational ambient pore pressure conditions, it is unlikely that a dam would be deterministically designed on this basis.

3.2 Mine Tailings Dam

It has been proposed that the current generation of tailings dams, about 30 m high, should be extended to heights of up to 100 m. It was thought that consolidation of the material within the dam could produce significant negative skin friction, or other drag forces, on slender concrete incrementally constructed penstock drainage shafts. An investigation was carried out on an existing dam by piezometer probing and Hughes Self Boring Pressuremeter Testing (HSBP). The latter was carried out by arrangement with Dr Hughes of Situ Technology, Vancouver. Disturbed and undisturbed piston samples were taken for laboratory testing. A series of CUPT and HSBP holes was put down from the dam wall to the penstock. At this type of waste disposal facility, discharge is from the walls towards the centre penstock drainage and hence the coarser material is deposited close to the wall and the fines close to the centre. This is illustrated in Figure 3, which shows the undrained shear

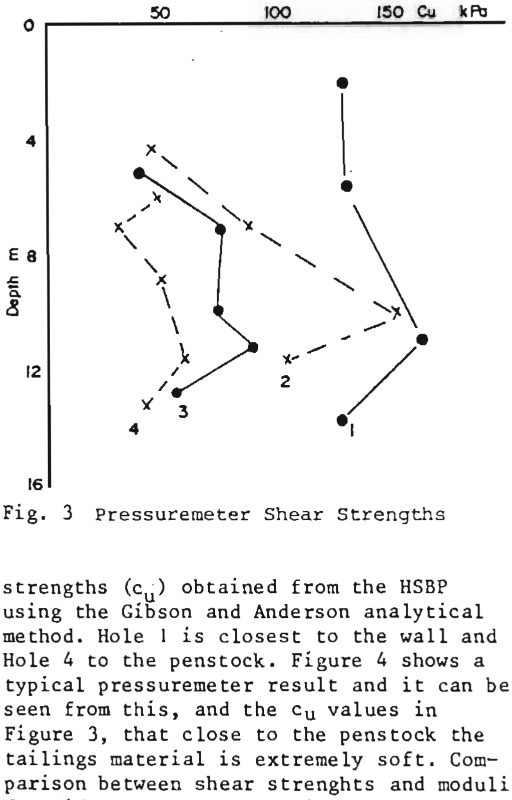

Fig. 3 Pressuremeter Shear Strengths

strengths (c_u) obtained from the HSBP using the Gibson and Anderson analytical method. Hole 1 is closest to the wall and Hole 4 to the penstock. Figure 4 shows a typical pressuremeter result and it can be seen from this, and the c_u values in Figure 3, that close to the penstock the tailings material is extremely soft. Comparison between shear strenghts and moduli for different methods of interpretation are being carried out.

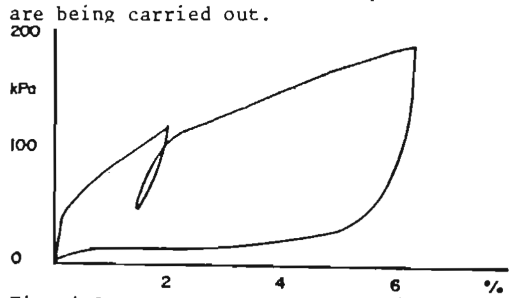

Fig. 4 Pressuremeter Stress Strain

It is common for tailings dams to be constructed over natural stiff fissured clay deposits. Stability analyses generally show that the critical failure surface is wedge shaped, with the near horizontal section passing through the underlying clay. Knowledge of the pore pressure regime within the clay is therefore essential. CUPTs were carried out through the tailings and into the underlying clay. The results of two such tests, carried out at a six months interval, are shown in Figure 5. During this period the dam was not raised in height and it can be seen from the figure that although the hydrostatic height changed by 0,4 m (4kPa), the excess pore pressure in the clay decreased by a further 10kPa. With a suitable model the macro permeability can be estimated. Since dissipation tests were carried out at halts in the penetration testing, micro permeability can also be estimated, (Jones and van Zyl, 1981; Schmertmann, 1978). These were orders of magnitude different, which is assumed to be due to the fissuring in the clay increasing the macropermeability - it should be noted that dissipation tests are usually carried out where the generated pore pressures are highest, which implies a non-fissured position, i.e. the probe dissipation tests are a biased sample.

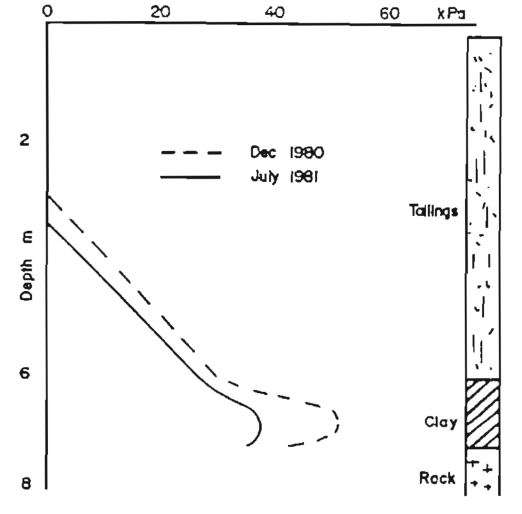

Fig. 5 Pore pressures in tailings and subsoil at 6 month interval

Laboratory consolidation tests were carried out on samples of the tailings and these gave coefficients of consolidation of about 1-5 m^2/year, whereas the piezometer probe values were about 50-250 m^2/year.

This anomaly has not been resolved, but it seems probable that the field values are more realistic. The very low laboratory results are assumed to be a measure of vertical permeability through very carefully selected samples containing lenses of the finer deposits. Whilst the values are in themselves correct they are not representative of the tailings material generally.

Gradings were carried out and the results used to add information to the classification chart described in the discussion section.

3.3 River Crossing

Two lines of CUPTs, each consisting of 15 positions, were put down across a river bed about 350 m wide, to determine the conditions for cut off walls for a proposed coffer dam, to allow construction of a dam. Considerable prior investigation, consisting of seismic surveys, boreholes and in situ permeabilities, had taken place and the objective of the piezometer probing was to confirm rock levels, provide strata identification and to give relative measures of permeability.

For comparitive purposes some mechanical friction sleeve probing was also carried out. Adjacent mechanical and electrical probe results are shown on Figure 6.

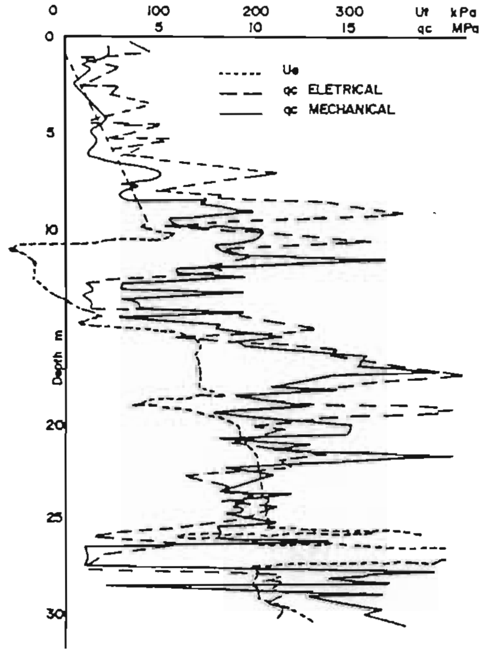

Fig. 6 Mechanical and Electrical Cone Pressures and Pore Pressures

These show general agreement of the q_c values, except that in the upper few metres the electrical readings have significantly higher peak values, in addition to being generally higher. As well as being due to inherent differences between continuous and discontinuous system, the probes were about 10 m apart and the upper sand banks undergo considerable changes due to the river movements. Figure 6 also shows the total pore pressures recorded during penetration. It will be seen that down to 10 m depth the pore pressure is hydrostatic, then between 10 and 15 m negative excess pressures are generated. From 15 m to 18 m the pore pressures return to practically hydrostatic, before reducing again for about a 1 m thick layer at 19 m depth. From there to 25 m the pressures are practically hydrostatic again before a zone occurs of fluctuating values including some very high excess pore pressures down to 30 m.

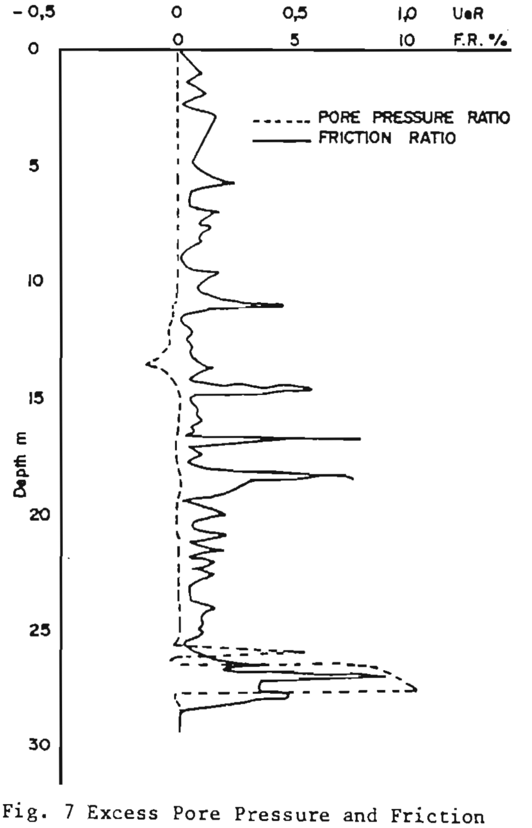

Fig. 7 Excess Pore Pressure and Friction Ratios

Figure 7 shows both mechanical friction ratio, FR, and excess pore pressure ratio, UeR, $\left(\frac{U_e}{q_c - \sigma_{vo}}\right)$ plotted against depth, where

U_e is the generated excess pore pressure. Although both ratios show generally similar results, there are some discrepancies. For example FR shows two peaks between 10 and 15 m, indicating clayey material with sand between, whereas the U_eR shows a clayey layer for this full layer thickness. Similarly the FR shows peaks between 15 m and 20 m which are not reflected in the pore pressure ratio. The second of these peaks coincides with a low pore pressure value accompanied by a high cone value (see Figure 6) which is taken to indicate not a clay, but a very dense silt. Between 25 m and 30 m both systems indicate a clay layer which according to the pore pressure, appears more extensive than the mechanical friction ratios indicate.

Drilling carried out in the vicinity confirmed about 10 m of clean medium to coarse sand, overlying very stiff clayey silt and softer clays to 15 m, followed by sand to 25 m then firm clay.

4 DISCUSSION

The site work described above demonstrates the ease with which CUPT probing can be carried out even in difficult circumstances, e.g. at the river site about half the probes were in a fairly rapidly flowing but shallow water. A standard 100 kN Goudsche Machinefabriek machine was used which was winched into position with four wheel drive vehicles. Electric cables were submerged most of the time and the electronic data logging system was either placed on folding camp tables, if the water depth permitted, or on the back of a vehicle.

Considerable discussion has already taken place on the interpretation of pore pressure readings, e.g. ESOPT I (1974), Raleigh (1975), St Louis (1981), and there is a consensus on qualitative evaluations of the results. Recently there has been more emphasis on the use of the generated pore pressure to obtain soil classifications. The authors presented a classification chart (Jones et al, 1981) based on normalized parameters, $\frac{U_t-U_e}{U_o}$ and $\frac{q_c-\sigma_{vo}}{\sigma_{vo}}$,

This appears to have no significant advantage over simply using (U_t-U_o), and $(q_c-\sigma_{vo})$, and in fact if the former is used, close to or above the water table, the pore pressure parameter becomes very large; similarly, close to ground level, the cone pressure parameter also becomes very large for small overburden pressures and log plots would be necessary to accommodate these values. It is possible that a log-log representation would be an improvement but at this stage it has been found that the chart presented in Figure 8 is satisfactory..

There are three aspects which should be emphasized.

Firstly, the accumulation of data for such a chart requires controlled results from many sites so that some of the material type boundaries shown on the chart are not yet well defined, although it is believed they are generally correct. Since the pore pressures are a function of the position of the sensing element some standardisation is urgently required if progress is to be made.

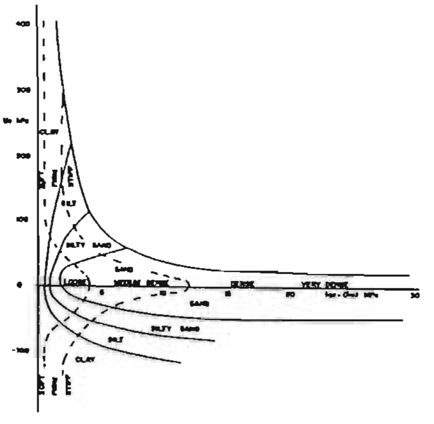

Fig. 8 Soil Classification Chart from Excess Pore Pressures and Cone Pressures.

Secondly, it is strongly urged that the ratio used should be that of the excess pore pressure, and not the total pore pressure, to a cone pressure function. The latter could be simply the cone pressure, since the overburden pressure does not usually have a large effect.

Thirdly, it is suggested that a chart format is much more useful than a single ratio, since it introduces a further dimension. It is of interest to note that Begemann (1975) mechanical friction ratios first started as a chart, then became generally used as a simple ratio, but have now returned to a chart format, for more meaningful interpretation.

5 CONCLUSION

Piezometer probing - CUPT - has now been sufficiently developed to require some standardisation in the equipment and procedures so that more co-operation and information exchange can take place. It would appear that agreement on the positioning of the filter element is the most important issue.

The field results given, demonstrate the various uses of pore pressure penetration testing, including the measurement of shear strength and consolidation parameters, in situ pore pressure conditions and soil classification

A soil classification chart based on excess or generated pore pressures and cone pressures is given.

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(ix) PIEZOMETER PROBE (CPTU) FOR SUBSOIL IDENTIFICATION

SYMPOSIUM INTERNATIONAL ESSAIS EN PLACE IN SITU TESTING VOLUME 2 PARIS 1983

PIEZOMETER PROBE (CUPT) FOR SUBSOIL IDENTIFICATION

PENETROMETRE PIEZOMETRIQUE (CUPT) POUR L'IDENTIFICATION DES SOLS

JONES Gary A.*, **RUST** Eben*

Summary

A piezometer probe system is described which is used inter alia to identify subsoils.
The piezometer tip is similar to the standard 35 mm diameter electrical cone (CPT) but in addition has a pore pressure sensor. This allows measurement of the excess pore pressures generated during penetration.
A chart has been drawn up relating these excess pore pressures, and the cone pressures, to the type and consistency of the subsoil. Simple theoretical justification is given for the chart.
Examples are given illustrating the use of CUPT field results to identify the subsoil and hence to draw up detailed geological sections.

Résumé

On décrit un pénétromètre piézométrique, utilisé en vue de l'identification des sols. La pointe du pénétromètre est identique à celle du pénétromètre standard de 35 m de diamètre, mais possède en plus un capteur de pression de pore ; ce montage permet la mesure de la surpression de pore générée pendant la pénétration.
On a mis en évidence la relation entre surpression de pore, pression et pénétration et type et consistance du sol. Une justification théorique élémentaire est donnée à cette relation.
Des exemples sont donnés illustrant l'emploi du pénétromètre piézométrique pour l'identification des sols et la réalisation de coupes géologiques détaillées.

1. Introduction

Subsoil identification is a necessary part of all geotechnical investigations. For this, borehole sampling is used, but in many cases this is both difficult and relatively expensive due to the subsoil conditions. Because of these problems there has been a growing emphasis on *in situ* testing techniques. One of the more widely used techniques is Cone Penetration Testing (CPT). Recently the CPT equipment has been developed to continuously measure, through electronic systems, pore pressures as well as penetration resistances.

This paper describes a method of subsoil identification using Piezometer Cone Penetration Testing (CUPT).

The method was developed semi empirically by correlating CUPT data with independent soils classification tests. From these correlations a chart was drawn up which relates the cone pressure and the excess pore pressure developed during penetration, with a description of the material type and consistency. It is shown that the method has theoretical justification.

An example of the use of the chart is described in which the complex subsoil profile across a river bed is drawn up from CUPT results.

It is emphasized that this subsoil identification is additional to the convention parameters obtained from piezometer cone penetration testing.

2. Piezometer cone system

Fig. 1 shows a cross section through the piezometer cone. In the interest of standardisation, the common size, 35,7 mm diameter, 60° cone, forms the basis of the instrument. The cone load is measured with a strain gauge load cell housed in a sleeve which is identical in external dimensions to the standard friction sleeve used in electrical penetration testing i.e. 35 mm diameter by 133,7 mm long. A similar load cell and sleeve – not shown in the drawing – is mounted above the lower sleeve and thus records a total cone plus sleeve reading, enabling conventional friction ratios to be deducted. Some electrical cones measure the sleeve friction separately, but the system described is believed to provide a simpler and more robust design. It is noted that Schaap and Zuidberg (1982) now favour this method.

Considerable discussion has taken place regarding the optimum position of the filter element. However, the filter position shown is the most satisfactory, since it provides the best compromise between sensitivity, convenience, wear and clogging (Jones and Van Zyl, 1981).

The filter rings are 4 mm thick and are easily cut from readily available porous plastic sheets. This filter thickness provides good response time and layer resolution.

The pore pressure measuring system consists of a miniature transducer mounted in a recess of the least practicable dimensions. The two load cell and pore pressure transducer outputs are amplified in the penetrometer tip, with high

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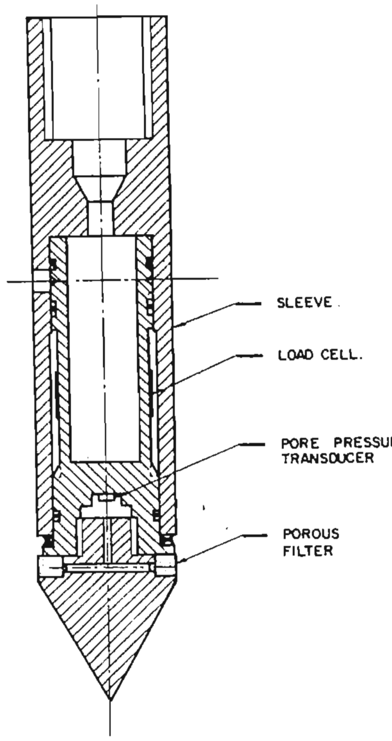

Fig. 1 : Piezometer probe

quality instrumental amplifiers, since down the hole amplification improves the signal to noise ratio. This has the advantage that the cable length is not limited by electronic considerations. A surface control unit with variable gains, conditions the analogue signals and feeds them to a chart recorder and to a data acquisition unit. It is strongly advocated that this dual system should be used, since the chart recorder allows on going assessment of the results. Both the chart recorder and the data acquisition unit are controlled by an optical shaft encoder which senses the depth of penetration.

A Sharp MZ 80B personal computer, adapted with analogue to digital 1/0 boards is used as the acquisition unit. The computer has 64k RAM, 2k ROM, two graphic areas, dual floppy discs, CRT display and tape drive. The system is soft ware controlled and therefore allows exceptional flexibility. For normal probing the sampling rate used is 4 Hz i.e. one reading of each channel at 5 mm intervals; if higher resolution is required the system can be operated at 20 Hz, i.e. 1 mm intervals, or more, without difficulty. A flat bed Watanabe 6 pen plotter is connected to the computer so that final report quality CUPT logs can be produced immediately.

The entire system is easily portable in separate sub units and can be readily connected to any probing rig.

3. Piezometer probe results

Typical CUPT results are shown in fig. 3 and 4. It can be seen that in sandy materials the total pore pressures measured (u_t) are close to the hydrostatic pressures (u_o) i.e. the excess pore pressures generated (u_e) are practically zero :

$$u_e = u_t - u_o$$

On the other hand, in soft to firm clays fairly high pore pressures are generated and in stiff clays or silts, which dilate on shearing, negative excess pore pressures are generated.

Since the excess pore pressures are material dependent, this factor can be used to identify materials. Extensive correlations of pore pressure responses and cone pressures, against independent identification of many soils, have enabled a chart to be drawn up which is described in the following section.

A wide range of other applications of the CUPT has been well established. This includes the derivation of effective stress parameters (Janbu and Senesset, 1974 ; Sugawara and Chikaraishi, 1982) ; consolidation parameters (Baligh *et al.*, 1981 ; Campanella and Robertson, 1981 ; Campanella *et al.*, 1982 ; Jones and Van Zyl, 1981 ; Torstensson, 1975) and *in situ* pore pressures. From the latter it is possible to derive flow nets in tailings dams and the state of consolidation and stability under embankments. Since the measurements are continuous, the variation of these and other parameters can be assessed and used in risk and probability analyses.

4. Soils identification chart

A chart shown in fig. 2 has been drawn up by correlating the excess pore pressures (u_e), and the cone pressures (q_c) minus the total vertical stress (σ_{vo}), against soil type and consistency obtained from laboratory tests of undisturbed samples.

Where similar piezometer cone results and sample descriptions were available in the literature, these too were taken into account.

The majority of the data points used in compiling the chart were from geologically recent alluvial deposits and from tailings dams.

The following points regarding the chart should be noted :
(i) The vertical axis represents the excess pore pressure (u_e) generated during penetration and may be either positive or negative with respect to the ambient pore pressure (u_o).
(ii) u_e may be below minus 200 kPa, the limit shown on the chart, but in the authors' experience, in this zone, infrequent and insufficient information is presently available to allow further detail on the chart. However, u_t, the total pore pressure, cannot exceed minus one atmosphere.
(iii) u_e may be above the 400 kPa shown on the chart, in which case material producing such a high u_e is clay.
(iv) The horizontal axis ($q_c - \sigma_{vo}$) only shows values up to 25 MPa, because the authors' data has comparitively few points beyond this range.
(v) The consistency boundaries are defined using the conventional correlations between cone pressure and strength or density.
(vi) The material type boundaries should be seen as transition zones and not as definite boundaries; the positions of these may be modified slightly as further information becomes available.
(vii) These are two aspects which are dependent on the particular piezometer cone used : these are firstly that the position of the filter element controls the portion of the generated pore pressure which is actually measured. Experience suggests that provided the filter element is either on the cone face, or within one diameter above it, then the difference is insignificant as regards the use of the chart. The second aspect is that the cone pressure should be corrected by the net area ratio as described by Campanella *et al.* (1982) and De Ruiter (1981). In practice this correction only becomes significant for materials with relatively high pore pressures and low cone pressures.
(viii) A zone of negative excess pore pressure and low cone

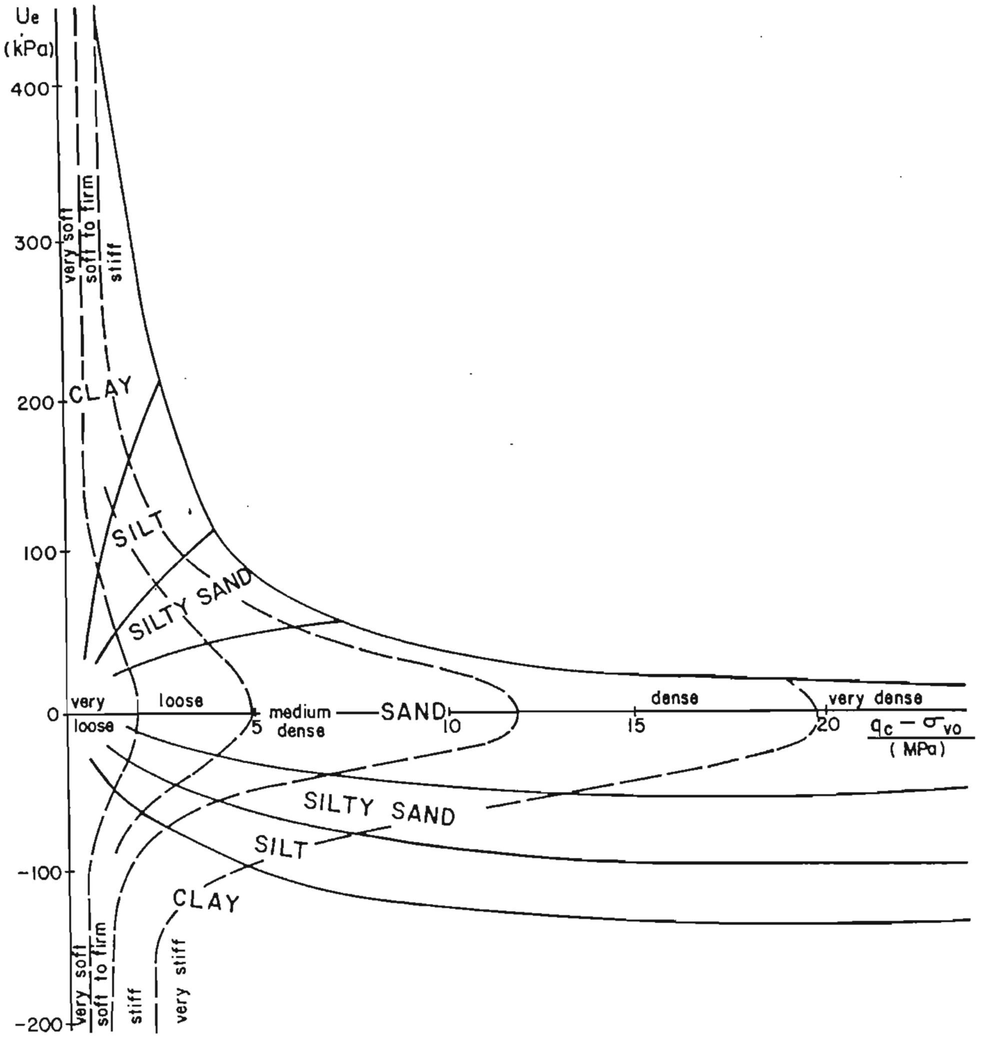

Fig. 2: Soils identification chart

readings is shown as soft clay. These results have been recorded in practice, although the theoretical justification is not clear.

(ix) The chart has been drawn up only from results obtained from fully saturated subsoils and tailings.

5. Interpretation of CUPT results

Fig. 3 and 4 show two CUPT results taken from the thirty probes put down in two lines, each about 400 m long across the Sabi River in Zimbabwe, at the site of a major irrigation dam. These were part of an investigation for the feasibility and design of diaphragm walls to be used as coffer dams, and detailed definition of the subsoil was required. Comparison with CPT's using the Standard Dutch Mechanical friction sleeve tip are discussed by Jones and Rust (1982). Boreholes were also put down to obtain soil samples and to allow diamond drilling into the bedrock.

The use of the chart to interpret the CUPT field results, shown in fig. 3 and 4, is demonstrated below :

Fig. 3 : depth 1,0 m-6,5 m : note water table measured at 0,8 m depth, therefore at 6,5 m depth, hydrostatic pressure (u_o) is 56 k.Pa. Measured u_t is 63 k.Pa and therefore u_e is 7 k.Pa.

Average cone pressure (q_c) in this zone is 4 MPa, overburden pressure (σ_{vo}) is about 60 k.Pa, therefore ($q_c - \sigma_{vo}$) is approximately 4 MPa. The chart shows the material to be a loose sand for the upper 6,5 m and ranges from very loose at the top to medium dense at the base of the stratum.

Depth 6,5 m-10,0 m : average u_o is 75 k.Pa, average u_e is 190 k.Pa and mean q_c is 1,5 MPa. Therefore the material is firm silty clay.

At two positions in this stratum there are market decreases in pore pressures coincident with increases in cone pressure. If these are plotted in the same manner as described above, it will be seen that they are silty sand and sand lenses.

Depth 10,0 m-13,3 m : u_o is 105 k.Pa, average u_t is -10 k.Pa therefore the average u_e is -115 k.Pa. Mean q_c is 10,0 MPa and σ_{vo} is 0,18 MPa, therefore ($q_c - \sigma_{vo}$) is about 9,8 MPa and the material plots as a very stiff silt practically on the clay-silt boundary, hence the soil may be described as a very stiff clayey silt.

Depth 13,3 m-14,7 m : average u_o is 130 k.Pa, u_t is 105 k.Pa, and therefore average u_e is -25 k.Pa.

Mean q_c is 9,8 MPa and σ_{vo} is 0,22 MPa, so ($q_c - \sigma_{vo}$) is about 9,6 MPa and the material plots as medium dense silty fine sand.

Depth 14,7 m-16,5 m : average u_o is 145 k.Pa and u_t although only shown as greater than 300 k.Pa was actually measured as 425 k.Pa, therefore u_e is 280 k.Pa.

Mean q_c is 1,0 MPa and σ_{vo} is 0,25 MPa, so ($q_c - \sigma_{vo}$) is about 0,75 MPa and the material plots as soft clay.

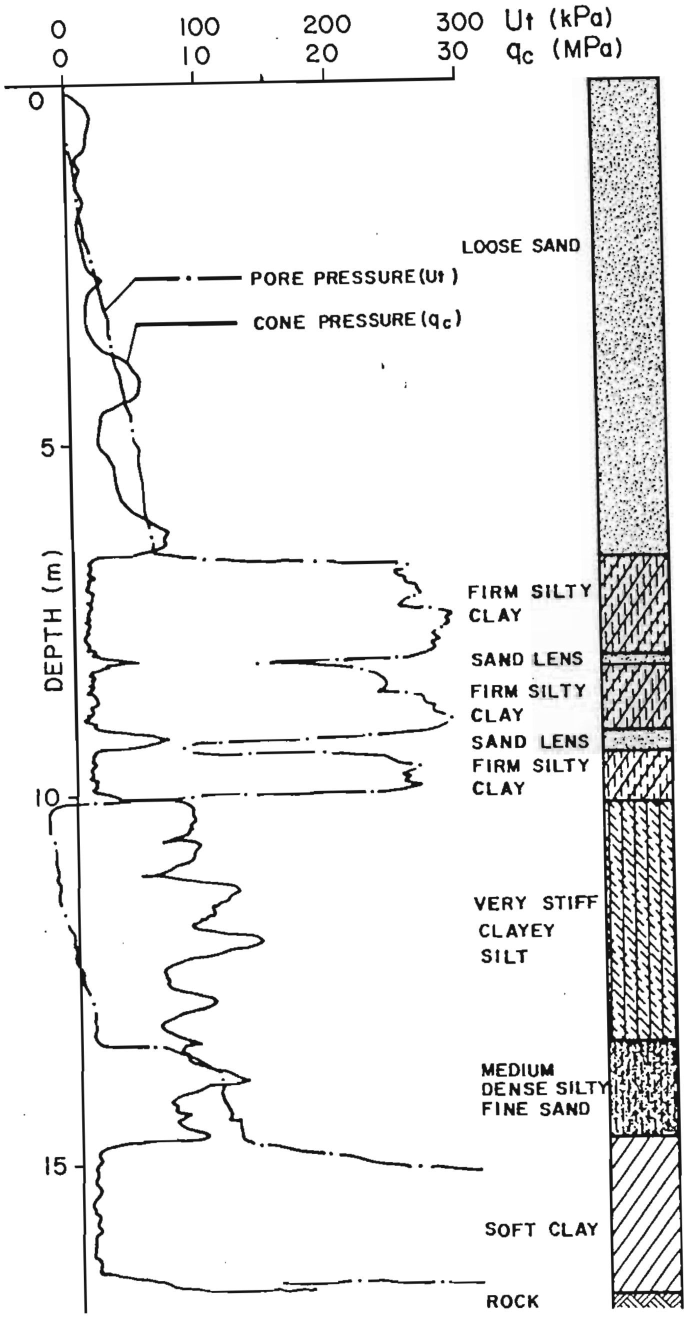

Fig. 3: Cupt N° 5

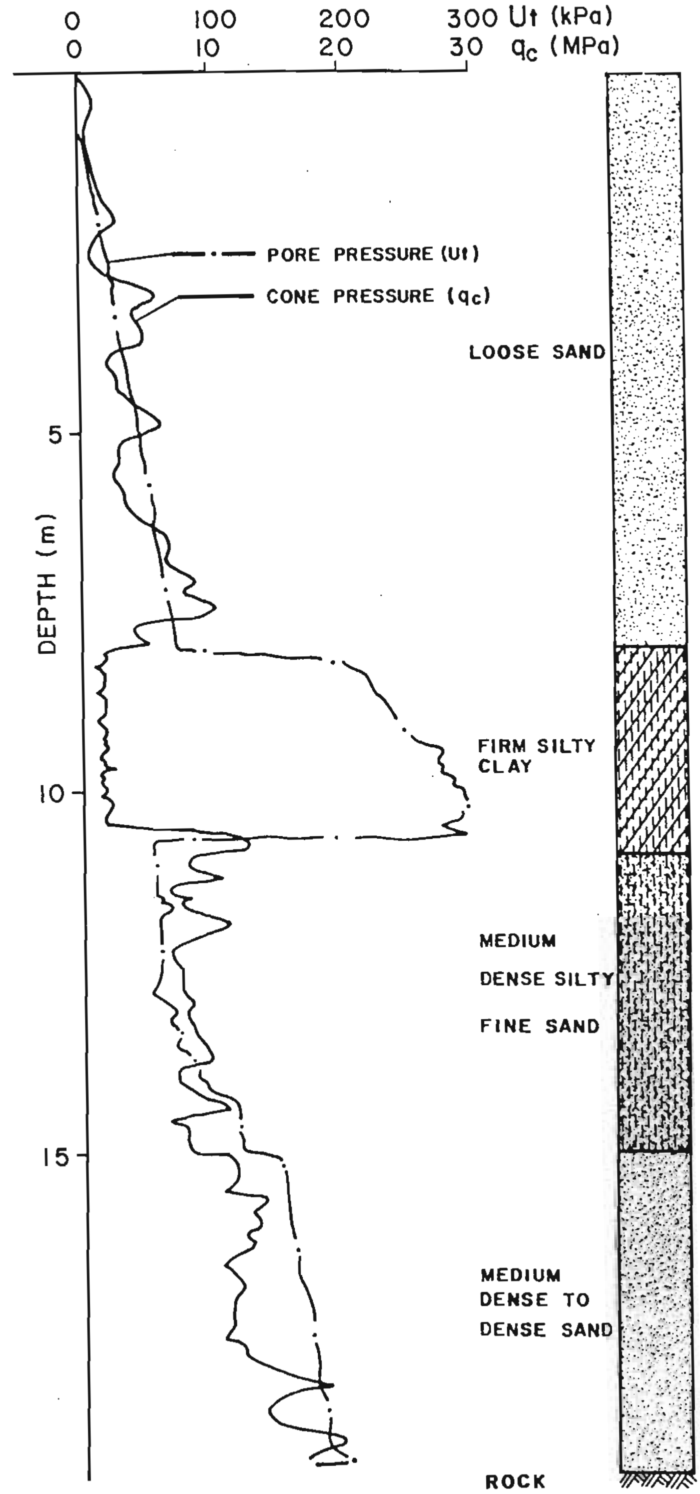

Fig. 4: Cupt N° 6

At the final refusal depths the probe was allowed to remain in position until the pore pressures stabilized at the hydrostatic pressure and these are shown on fig. 3 and 4. This procedure was also adopted at some rod change and dissipation test positions, but for clarity these are not shown on the logs.

All the CUPT's were interpreted in the same manner and the summarized geological section shown in fig. 5 was derived. From the field data the layers were defined with higher resolution resulting in a more detailed geological section which again for the sake of clarity is not shown in the figure.

6. Theoretical considerations

Many theoretical models have been proposed to simulate cone penetration. At this stage no single model would

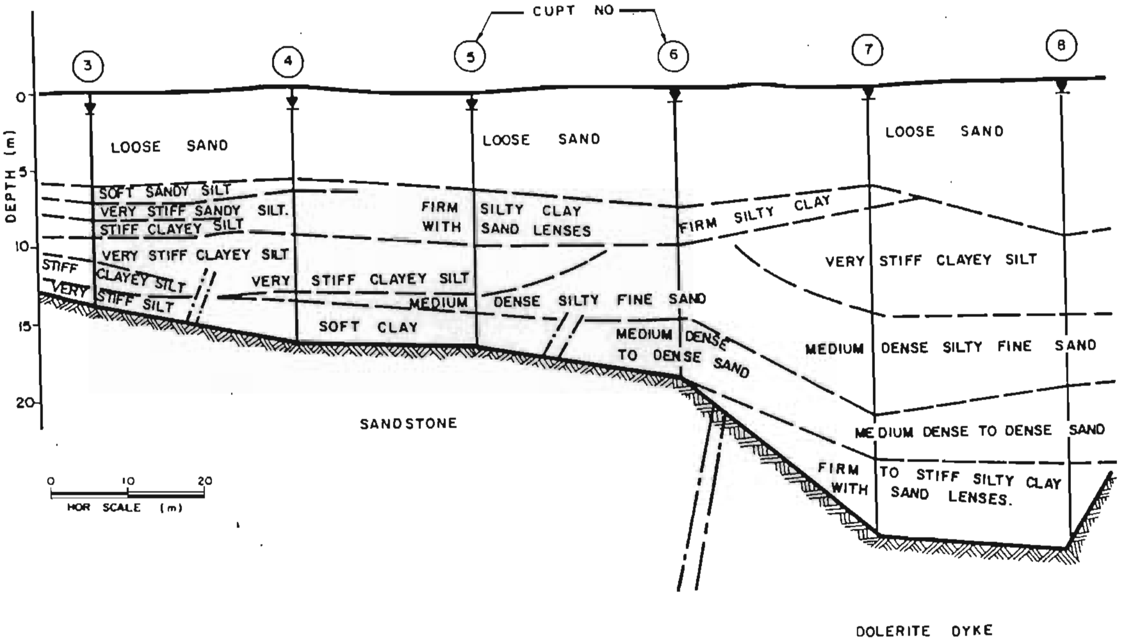

Fig. 5 : Geological section

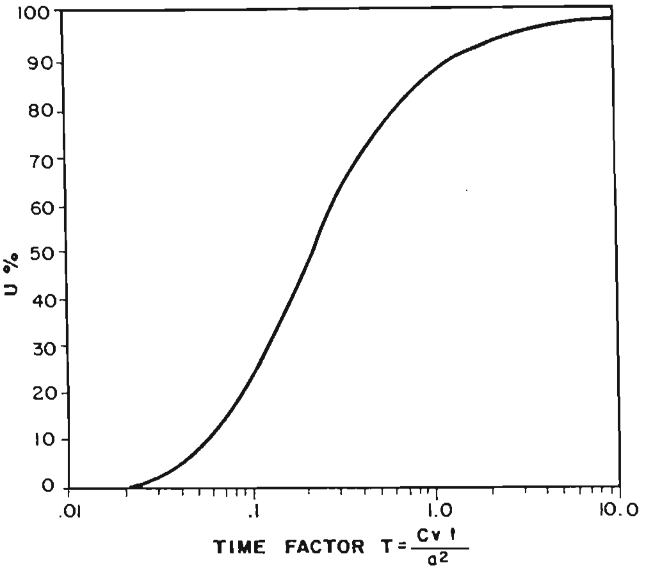

Fig. 6 : Probe time factor

appear to adequately represent all aspects of cone behaviour in a wide range of materials, although some correlate satisfactorily with field results in specific materials. Considerable emphasis has recently been given to cavity expansion models, Rol *et al* (1982), Vesic (1972), although the assumptions made would appear to restrict their applicability to the prediction of pore pressure response.

It is instructive to consider the simple Skempton model of pore pressure response to shear behaviour represented by the equation :

$$\Delta u = B\,[\Delta\sigma_3 + A\,(\Delta\sigma_1 - \Delta\sigma_3)]$$

if B = 1 for fully saturated soils,

$$\Delta u = \Delta\sigma_3 + A\,(\Delta\sigma_1 - \Delta\sigma_3)$$

At any given depth $\Delta\sigma_3 = 0$, since σ_3 is constant.

At failure $A = A_f$, therefore,

$$\Delta u = A_f\,\Delta\sigma_1$$

But for cone penetration $\Delta\sigma_1$ is a function of $(q_c - \sigma_{vo})$ and if this function is linear then the equation may be written,

$$\Delta u = I_u\,A_f\,(q_c - \sigma_{vo}) \qquad (1)$$

where Δu is the initially generated pore pressures on the shear surfaces during penetration, and where I_u may be considered as a constant for the shear regime surrounding a standard cone during penetration.

Typical results suggest that the value of I_u is approximately 0,3.

However, what is actually measured at the probe is the remaining pore pressure generated on the shear surface after a finite fime, t. A useful model to illustrate this is given by Jones, Van Zyl and Rust (1981), since this allows an estimate of the time, t, during which some dissipation will have taken place. The relative amount of dissipation is given by fig. 6 which shows the Probe Time Factor versus Degree of Consolidation.

The Probe Time Factor, T, is given by the following equation :

$$T = \frac{c_v t}{a^2} \qquad (2)$$

where c_v is the coefficient of consolidation,
t is the time which is dependent on the rate of penetration and the dimensions of the model sphere,
a is the radius of the sphere and this can be shown from measured time lag responses to be about equal to the cone diameter.

The above can be shown to be realistic modelling by considering the results in fig. 3 and 4.

For example in fig. 3, at about 7 m depth $(q_c - \sigma_{vo}) =$ 1,10 MPa and u_e is about 150 kPa, in the firm silty clay layer. The coefficient of consolidation c_v may be estimated as 10^{-7} m²/sec. If "a" is equal to the cone diameter and the penetration rate is 20 mm/sec then :

$$t = \frac{2a}{20} = 3{,}5 \text{ sec}$$

Therefore, from equation (2) $T = 2{,}9 \times 10^{-4}$.

From fig. 6 the degree of consolidation, U, in this time is zero.

This means that the maximum pore pressure generated will actually be measured by the probe, about three seconds after generation.

In equation (1) if A_f is assumed to be 0,4 (which is a realistic value for this firm silty clay), then the calculated maximum generated pore pressure is 130 kPa. This is in reasonable agreement with the measured pore pressure of 150 kPa.

A further example can be examined by considering the very stiff clayey silt in CUPT 5 at 10,5 m depth. In this ($q_c - \sigma_{vo}$) is about 8 MPa, u_e is about − 120 kPa and c_v is estimated to be 5×10^{-4} m²/sec. Therefore T is about 1,5 and from fig. 6, U is about 30%. The calculated maximum pore pressure is about − 240 kPa if A_f is about − 0,1, therefore the measured pore pressure should be about − 170 kPa, compared with the actual measured value of − 120 kPa.

Similarly it can be shown that for sands total dissipation would take place in 3,5 seconds or less, so that u_e should be about zero, which agrees with the field results.

It can be inferred from the above that a model predicting maximum generated pore pressures on the basis of simple shear, in conjunction with a simple dissipation rate model allows reasonable predictions of measured excess pore pressures at the probe. This in turn gives justification to the configuration of the proposed materials identification chart.

7. Conclusions

A materials identification chart is proposed which utilizes cone and pore pressures measured during piezometer probing (CUPT).

Theoretical justification for the chart is given on the basis of a simple shear model and the conventional Skempton pore pressure parameters A and B. It is considered that this model expressed in terms of increments of stress invariants, $\Delta\sigma_{oct}$ and $\Delta\tau_{oct}$, and pore pressure parameters, possibly in conjunction with expanding cavity considerations, would result in a better representation of the effects of cone penetration in the surrounding subsoil. A corresponding pore pressure distribution and dissipation model should be developed.

In some respects the chart may be considered as analogous to the Begemann Friction Ratio method used with the Dutch Mechanical Friction Sleeve Tip.

It is however contended that the CUPT method is intrinsically more fundamental and is much more sensitive in detecting thin layers.

It should be noted that the measured values of excess pore pressure are dependent on the particular piezometer probe used. The standardisation of piezometer probe systems is strongly recommended.

8. Acknowledgements

The data used in compiling the Materials Identification Chart has come from too many sites to be individually mentioned. Particular thanks however is expressed for the encouragement and cooperation of the South African Department of Transport since it was primarily on their projects that the method was developed. It is also noted that the example illustrating the use of the chart is taken from a project carried out for the Division of Water Development, Zimbabwe and that their agreement to use the CUPT is greatly appreciated.

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(x) SOFT CLAYS, PROBLEM SOILS IN SOUTH AFRICA

Soft clays

Recent infrastructure developments, primarily along the Natal coast for roads, railways and harbours, have highlighted the significant problems of settlement and stability of embankment and structures on soft clays. Soft clays are not a problem specific to South Africa, and the depositional history, engineering geological and geotechnical properties are similar to those for recent alluvial deposits elsewhere. Nevertheless, there are differences. Probably the most notable is that the alluvial deposits tend to be extremely variable, with sand, silt and clay strata being juxtaposed in a complex pattern both laterally and vertically. This leads to difficulties in ensuring adequate investigation of the strata, which in turn leads to problems of realistically predicting times for settlement. This review discusses the identification, analysis, design and construction on soft clays, giving particular attention to those aspects pertinent to local practice.

■ *G A Jones (Fellow) and P Davies (Member)*

Introduction

Unlike the problem soils described in other papers in this issue, soft clays have no particularly Southern African connotations and have not therefore given rise to any significant localized techniques of investigation, analysis, design or construction.

Nevertheless, the presence of relatively thick deposits of soft clays has frequently necessitated expensive solutions for earthworks and for structural foundations. It is therefore believed that a review of current methods of geotechnical engineering on soft clays is justified on the basis that foreknowledge of the potential problems can lead to economic benefits if the planning, design and construction of projects can be modified to suit the problem.

The emphasis of this review is therefore on the description of well established techniques of investigation and interpretation which are, or should be, in current use in South Africa, rather than on new developments. It is therefore hoped that this review will be of general interest to civil engineers working in areas of soft clays and accepted that it may serve only as a reminder to specialist geotechnical engineers of the limitations of their craft.

Gary Jones, *Pr Eng, graduated at Cambridge in 1957 and is now a student at the University of Natal. Having worked primarily in road construction and design in Zimbabwe and Uganda, he developed an interest in geotechnical engineering which brought him to South Africa in 1965 to work in Natal. Spells with consultants and the Natal Provincial Administration Roads Department brought an appreciation of the problems of civil engineering on soft clays which led to his move to the National Institute of Transportation and Road Research. He subsequently moved back to consulting and is a director of Steffen Robertson and Kirsten at Pretoria. He is the author of papers on embankments on recent alluvium and on piezometer probing.*

Peter Davies, *Pr Eng, is a senior partner of consulting geotechnical engineers Davies, Lynn and Partners. He graduated from the Imperial College of Science and Technology, University of London, with a BSc (Eng) (Civ Eng) in 1968. Specializing in geotechnical engineering, he was later awarded the degrees of MSc in 1969 and PhD in 1975. After spending six years working with consulting engineers in Durban, he formed his own practice in 1981. He has been responsible for the geotechnical aspects of a wide variety of engineering projects throughout South Africa and also in South America, and is the author of various international papers on topics including the stability of rock slopes and the performance of piled foundations in stiff clays and in weathered rocks.*

Location of soft clays

In South Africa soft clays occur predominantly in the coastal areas. The most significant deposits are at Durban, Richards Bay and the Natal North and South Coasts. The Cape East Coast also has areas of soft clays, notably in the coastal strip at Knysna. The southern Cape Coast has soft clays at a number of estuaries and Cape Town itself has some very localized deposits.

There are occurrences of soft clays in the inland areas, but these tend to be fairly shallow, very recent transported materials at poorly drained areas such as vleis. It is remarkable how many of these, superficially very poor areas, eventually prove to have relatively simple foundation problems, because the depth of soft materials is limited and the materials themselves are frequently sandy silts and silty sands. This does not imply that there are no inland soft clays, but they are certainly uncommon.

Geological origin

The soft clays occur primarily along the eastern seaboard. They are the result of a number of depositional environments which have occurred during periodic changes in sea level. In general the thicker, more clayey deposits have been laid down in lagoon environments of the main rivers, whereas the clayey silts tend to have developed in the tributary systems. However, there are many instances where large thicknesses of soft silty clays occur both in the flood plains of the tributaries and in those of the main rivers. Many of the soft clays contain shells, indicating that these clays were deposited under estuarine environments.

During the changes in sea level many of the rivers changed their courses and the positions of the different types of sediments are not now necessarily related to the present courses. An understanding of the geological history of a particular river system should lead to a better appreciation of the engineering problems; it is, for example, only too easy to make simplifying assumptions about the nature and disposition of the materials within an alluvial deposit and then to devise a solution to suit the simple model. This may either be uneconomically overconservative, if the alluvium is assumed to consist entirely of soft clays, or, on the other hand, totally ignore what could be the major problem of variability of the sediments and the consequential differential rates and magnitudes of settlements.

Brand and Brenner[1] describe the well-known overseas soft clay deposits such as the Scandinavian clays, the Canadian Champlain or Leda clays, the Chicago and Boston Blue clays, the Indian and more recently the Bangkok clays. The literature tends to emphasize the homogeneity of these individual, but relatively large deposits. The South Africa soft clays, whilst similar in many respects and amenable to the same analyses and engineering solutions, are relatively small in lateral extent and are generally highly variable, which introduces further problems for the investigations, analyses and engineering solutions. Although of limited extent, many of the soft clays are of considerable depth, up to and even exceeding 40 m in a few places, which is comparable to the most adverse conditions encountered elsewhere in the world.

Definition of soft clays

There is no standardized definition of soft clay in terms of conventional soil parameters, mineralogy or geological origin. It is however, commonly understood that soft clays give shear strength, compressibility and time related settlement problems.

It is generally understood that soft clays are fully saturated and normally consolidated. In South Africa neither of these conditions may hold, particularly the latter, and such overconsolidation as may exist could be of considerable significance to the solution of a problem.

In the near surface clays, which form a crust, partial saturation and overconsolidation occur together and the overconsolidation is a result of desiccation due to changes in the water table. In below surface clays, overconsolidation may have taken place when the clay was previously at, or close to, the ground surface and above the water table, but due to subsequent deposition, the clay strata may now be below the surface, saturated and overconsolidated. Overconsolidation ratios of two or three are common and higher values have also been noted. Partial saturation does not in itself cause engineering problems, but may lead to laboratory testing difficulties.

Soft clays would generally be expected to have undrained shear strengths from about 10 kPa to about 40 kPa. In other words, soft clays range in strength from exuding between the fingers when squeezed, to being easily moulded in the fingers, or alternatively from being impossible to walk on, to being possible to walk on, albeit with some misgivings.

The particle size classification system approved by the International Society for Soil Mechanics and Foundation Engineering defines clays as those materials with an equivalent particle size diameter less than 0,002 mm. The method of testing to determine the equivalent diameter is hydrometer analysis. In engineering terms, however, materials with as little as about 30 per cent of their constituents being clays, effectively behave as, and are therefore classified as, clays.

The plasticity of the material forms the basis of engineering classification of clays. The plasticity is represented by the Atterberg Limits, ie Liquid Limit (w_L), Plastic Limit (w_p) and Plasticity Index (I_p) where $I_p = (w_L - w_p)$. The Casagrande A line shown in Fig 1 is often used to define the category of the fine grained material.

In common with many soils, it is the in situ state of the material which is important and not only the nature as defined by the soil parameters measured on disturbed samples. The natural moisture content, (w_n), is of particular significance for soft clays and it is convenient to represent the in situ state by its Liquidity Index (I_L), where

$$I_L = \frac{w_n - w_p}{I_p}$$

It may be noted that if the natural moisture content is higher than the Liquid Limit, the Liquidity Index is greater than unity, a situation which may readily exist in soft clays, and may be of considerable significance in the prediction of the behaviour of the clay on loading.

It has been frequently observed that the Atterberg Limits may be significantly affected by the method of sample preparation, viz oven dried, air dried, or not dried at all. Whereas the former is satisfactory for typical road or earthworks materials, structural changes can take place in clays on drying, particularly at oven temperatures, which cause the subsequent Atterberg Limit test results to be inappropriate.

Results from tests on samples of alluvium at Richards Bay showed that at one site the Liquid Limits averaged 88 per cent for air dried, compared to 65 per cent for non dried, and that the Plasticity Indices were 51 per cent and 33 per cent respectively. Since the natural moisture contents averaged 83 per cent, the Liquidity Indices averaged about 0,9 for the air dried samples and 1,5 for the non dried samples. These samples had

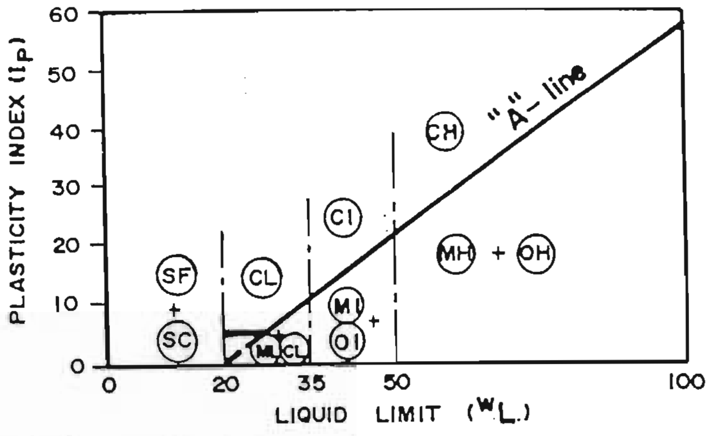

Fig 1: Casagrande Plasticity Chart

relatively high organic contents, averaging 8,1 per cent, and therefore the large differences in Atterberg Limits are not necessarily typical for less organic alluvium. Nevertheless, appropriate sample preparation is essential if comparisons are to be made with published data; air drying is generally recommended, minimum or no drying is preferable for clays with a significant organic content and oven drying is prohibited.

In general, Liquidity Indices exceeding unity indicate that the clay is still undergoing consolidation, whereas preconsolidated clays would be expected to have indices less than unity. High Liquidity Indices may also indicate a definite structure, eg leached Scandinavian clays or cemented Canadian clays, but no such structured clays occur in South Africa. The Liquidity Index may therefore be interpreted as a guide to the geological history of the clay and this in turn has a most important influence on the subsequent prediction of the consolidation behaviour of the clay on loading.

It is convenient to summarize soil descriptions by giving soil profiles together with Atterberg Limits and natural moisture contents such as in Fig 2, which, incidentally, also shows a Cone Penetration Test (CPT) log. Typical indicator test data for South African soft clays are given in Table 1.

Table 1: Typical indicator test data

Area/ Deposit	*Material description*	*Clay %*	*Silt %*	*Liquid limit w_L*	*Plastic limit w_p*	*Water content w_n*
S Coast						
flood plain	light grey sandy	15	35	30	15	35
back swamp	black silty clay	25	55	45	25	50
Durban						
estuarine	mid grey silty clay	50	40	55	35	55
lacustrine (Hippo)	black silty clay	65	30	65	30	90
Richards Bay						
estuarine	light grey sandy silty clay	20	50	40	20	30
estuarine	dark grey silty clay	25	55	50	20	35
lagoon (Black Mud)	black silty clay	35	60	65	30	45

Most soils, and soft clays are no exception, have been subjected to a barrage of semi-empirical or totally empirical correlations of simply measured indices with some less easily measured property, for example the well-known correlation of effective overburden pressure (σ'_{vo}), undrained vane shear strength (Su) and plasticity index (I_p), given by Skempton[2]:

$$Su/\sigma'_{vo} = 0{,}11 + 0{,}0037\ I_p$$

Similar relationships exist for preconsolidation pressures, overburden pressures and plasticity indices. Typical examples are shown in Fig 3, which also indicates the influence of the apparent age of the clay. A young clay is defined as one which has been recently deposited and which has not undergone significant secondary or delayed compression. For these clays the preconsolidation pressure, σ'_p, equals the effective overburden pressure, σ'_{vo}.

Normally consolidated aged clays are defined as those which have undergone compression through self weight and secondary compression for a long time and therefore indicate an apparent preconsolidation pressure. These aged normally consolidated clays exhibit an abrupt change in compressibility at stresses above the apparent preconsolidation pressure, but have a very small increase in compressibility below this pressure.

Overconsolidated clays have been subjected, at some time, to pressures higher than the present overburden pressures and the former have been reduced by surface erosion. During consolidation testing, these clays show a change in compressibility at the preconsolidation pressure, similar in appearance, but less marked, than that shown by aged normally consolidated clays. Below the preconsolidation pressure, however, the effect of the historical stress relief manifests itself in an increase in void ratio and hence in a steeper initial section to the consolidation curve than shown by aged normally consolidated clays.

Generally local soft clays can be designated as young and conform to the correlations of plasticity index with undrained shear strength given by Skempton (Fig 3). Good quality consolidation tests generally show some apparent preconsolidation pressures which are compatible with the geological age of the clays. However laboratory testing to determine apparent preconsolidation pressures is not easy to perform reliably, particularly since sample disturbance may have a marked influence. Therefore overdue significance should not be attached to a few results showing anomalies in the apparent preconsolidation pressure and hence in the age of the sample.

There are many expressions developed for estimating consolidation parameters. These generally relate the Compression Index Cc, (defined as $\delta e/\delta \log \sigma'$ the slope of the e-log σ' curve in the appropriate pressure range), or the Compression Ratio, CR, (defined as $C_c/(1+e_o)$), to either the natural moisture, w_n, or the initial void ratio, e_o. Empirical equations given by Assouz et al[3] are as follows:

$C_c = 0{,}40\ (e_o - 0{,}25)$
$C_c = 0{,}01\ (w_n - 5)$
$C_c = 0{,}006\ (w_L - 9)$
$CR = 0{,}14\ (e_o + 0{,}007)$
$CR = 0{,}003\ (w_n + 7)$
$CR = 0{,}002\ (w_L + 9)$

It has been found that local soft clays can be described by the above empirical equations. These form either a basis for assessing compressibility, or for assessing the reliability of compressibility parameters obtained from consolidation tests. Once again, however, caution must be exercised if the organic contents are high, since this can distort the behaviour of the clay. Typical shear strength and consolidation data for soft clays are given in Table 2 and Table 3 respectively.

Table 2: Typical shear strength data

Area Deposit	*Material description*	*Undrained C_u kPa*	*Drained c' kPa*	*Ø°*
S Coast	black silty clay	15-25	5	25
Durban	black silty clay	10-25	0	25
Richards Bay	sandy silty clay black silty clay	15-35 10-20	10-20 5-15	25-30 20-25

Table 3: Typical consolidation data

Area	*Material description*	*Compressibility m_v m²/MN*	*Consolidation c_v m²/year*
S Coast	light grey sandy silty clay black silty clay	0,3 - 1.0 0,5 - 1,5	5 - 10 1 - 5
Durban	black silty clay	0,5 - 2,0	0,5 - 2,0
Richards Bay	light grey sandy silty clay black silty clay	0,2 - 1,0 0,5 - 2,0	5 - 10 1 - 5

Engineering problems

Soft clays give rise to a very large range of engineering problems. Much of the fairly recent experience of soft clays has been gained along the Natal coastal strip, including Richards Bay, through the infrastructure development for roads, railways and harbours.

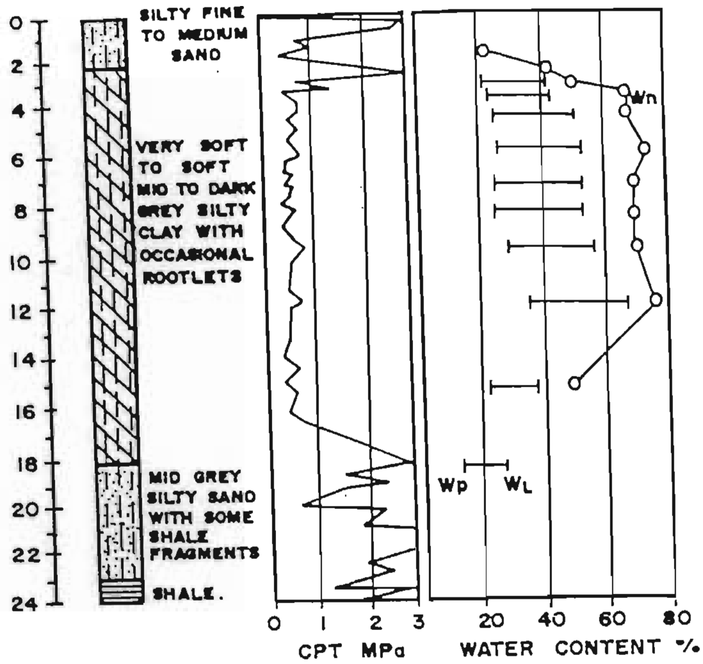

Fig 2: Typical borehole log, CPT and Atterberg Limits.

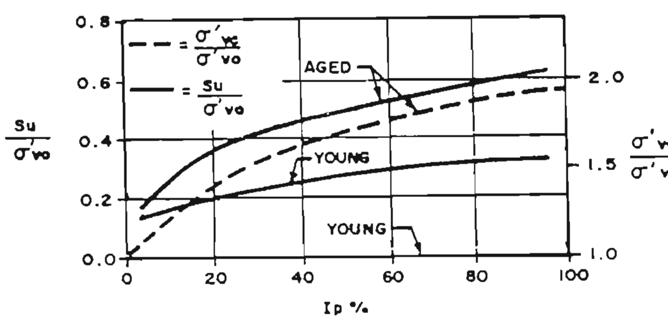

Fig 3: Plasticity index versus strength ratio $\frac{Su}{\sigma_{vo}}$ and preconsolidation ratio $\frac{\sigma_{vc}}{\sigma_{vo}}$ for young and aged clays

Embankments

A number of large embankments have been constructed for roads and railways. Some of these have had stability failures, considerable construction difficulties or large long-term settlements, often in excess of what were thought to be conservative predictions.

Practically all the reported stability problems have taken place during construction. These have resulted either in the classical rotation failure of a section of an embankment or even, in a few instances, to a complete displacement of the subsoil.

It is often found that the clays have a desiccated crust of relatively high shear strength. This may allow access during construction and it may be advisable to preserve this advantage by minimizing the removal of vegetation and its root structure. The higher strength, however, should not be relied upon in stability analyses since, in the short term, the crust is often brittle and is discontinuous, particularly after some settlement. In the long term, it will almost certainly become saturated and lose its dry strength characteristics.

Settlements, and specifically differential settlements, are a major problem associated with embankments on soft clays. In areas of fairly deep clays embankments have settled up to 30 per cent of their height, ie over 2 m for a 7 m high fill, and the settlements have continued for about 10 years, before reducing to a negligible rate. In extreme instances settlements of up to 95 per cent of embankment height have occurred.

In urban and harbour areas reclamation over soft clays is often carried out. Generally the thickness of fill is insufficient to cause stability problems, but medium to long term settlement may create difficult problems for the structures and services to be built after reclamation.

Structures

The low shear strength of soft clays precludes the use of shallow footings. In some cases it may be economical to place footings in a compacted fill over the clay, but it is likely that this solution would only be acceptable for relatively light structures which can tolerate differential settlements.

Under some circumstances structural rafts, probably on a pioneer fill layer, may be feasible, but then settlements will be large, and in most cases probably unacceptable.

Piled foundations are the norm and fortunately in many of the soft clay situations in South Africa, it is possible to pile through the clay into rock. In some instances, notably at Richards Bay, this may not be economical and foundations may be relatively expensive if they have to be piled through a thick stratum of soft clay, into an underlying dense sandy stratum. The problems of negative skin friction and lateral loading on piles may then be very severe and particularly difficult to overcome at, for instance, bridges where the approach embankments will settle subsequent to the bridge construction.

Engineering solutions

In general the problems associated with soft clay engineering do not readily lend themselves to ingenious solutions. For embankments it is necessary either to live with long term settlement problems, or to adopt fairly expensive solutions to accelerate the settlements so that they occur during construction. For structures, which are usually less tolerant of settlements, the problem is even less tractable and essentially is solved through by-passing the clay and many, but not all, of the problems.

Embankments

Recent experience of soft clays in South Africa has included the construction of numerous large embankments, primarily for roads and railways. At a significant number of these severe problems of both stability and settlement have resulted, often expected, but sometimes unexpected despite the site investigations which were conducted.

Stability: The solution of stability, or bearing capacity, problems of embankments on soft clays involves two approaches. The first is to decrease the shear stresses so that they are compatible with the shear strengths, and the second is to increase the shear strength of the soft clay so that it can sustain the shear stresses.

Decreasing the shear stress is achieved by flattening the embankment slopes. This may be by either reducing the slope from the usual 1 : 1,5 to say 1 : 2,5, or by introducing a berm. For various reasons, such as providing the extra mass in the most efficient position to act as a counterweight, and also because berms do not require to be compacted to the same degree as the main fill and may utilize poorer materials, it is usually preferable to use berms rather than slope flattening. Generally thick clay deposits favour berms and shallow deposits render slope flattening more appropriate. It is often found that berm heights should be about one-third of, and berm widths twice the embankment height to provide the most economical configuration.

It is widely reported in the international literature that embankments have been constructed with lightweight materials, such as PFA, and thus the shear stresses are reduced. This is a technique which is unlikely to be of frequent use in South Africa, since it relies on the local availability of lightweight materials to be economical. It may be noted that generally the purpose of using lightweight materials would be to reduce settlement, rather than to avoid stability problems, although of course the two problems often occur together. A recent extension of this principle is the use of expanded polystyrene foam as the embankment material.

The alternative approach of increasing the shear strength of the subsoil may utilize a number of techniques. Probably the foremost of these is stage construction, in which increases in shear strength are achieved through consolidation of the subsoil, induced by the earlier stages of embankment construction. This, however, takes time and careful monitoring to ascertain that the predicted strength gains have in fact been achieved.

A variation of the technique, where sufficient time is not available, is the acceleration of consolidation by the installation of drainage systems. These may be simple surface sand blankets, possibly with a network of sand filled trenches, or a grid of vertical sand drains or similar manufactured geofabric and plastic drains.

In general, it is likely that the primary purpose of these techniques will be to minimize post construction settlements, rather than promote stability. If, however, the latter is the objective, then it must be emphasized that improvement of shear strengths is necessary only in the highly stressed zones and not to great depth under the full width of the embankment.

It should be noted that the position of the highly stressed zones cannot be determined from limit state circular, or non-circular, stability analyses. Theoretically a finite element stress analysis method is necessary, if a simple empirical approach is considered to be inadequate for the particular circumstances. In practice such sophistication is rarely justified and improved drainage under the slopes and their shoulders, for a depth about equal to the embankment height, will be sufficient.

A further approach to increasing the shear strength of the subsoil is the introduction of strengthening systems. The possible variety of these is limited only by the imagination of the designer; Roman style bundles of sticks, old tyres, steel cables, geotextiles, stone columns, lime piles and conventional piles are a few of the choices. It may be argued that many of these systems should more correctly be described as changing the modulus of the soil reinforcing system, rather than increasing the shear strength, so the techniques should be described as soil improvement methods.

Settlement

Settlements on soft clays are due to both primary consolidation and secondary compression. Both are time dependent but the causes are different. The former is due to pore pressure dissipation and can be predicted with some confidence from consolidation tests. The latter is a more complex phenomenon and more difficult to predict, although long term consolidation tests do provide some indication of the likely magnitude. The relative amounts of primary and secondary settlements are also difficult to predict, but in general high plasticities or organic contents tend to indicate that secondary compressions may also be high.

Relatively high shear stresses caused by low factors of safety, and short drainage path lengths would also appear to induce high secondary compressions. Most of these factors cannot be controlled, but where they exist, viz high organic contents etc, careful consideration should be given to increasing the factors of safety of embankment slopes, by using berms or flatter slopes, to reduce the potential secondary compression.

Much effort has been expended internationally to establish simple relationships between the secondary compression index, C_α and readily determined soil parameters. Two of these, which are useful for a first estimate if long term consolidation tests are not available, are given by the following two equations due to Simons[6] and Mesri and Godlewski[7] respectively:

$$C_\alpha = 0{,}00018\ w_n\ (\%)$$

where w_n is the natural moisture content

$$C_\alpha = (0{,}025 - 0{,}10)\ C_c$$

where C_c is the compression index

Often it is not so much the amount of settlement that causes problems, but the rate at which it occurs. Techniques for dealing with potential settlement problems on soft clays therefore concentrate on either planning and organizing to minimize the effect of the problem, or on accelerating the rate of settlement. In fact there is practically no economical method of reducing the total amount of settlement in soft clays.

The techniques of preloading, together with surcharging if necessary, offer a very attractive solution to the problem.

Unfortunately sufficient time frequently seems to be unavailable to allow the techniques to be effectively utilized, with the result that either large amounts of surcharge have to be used, and expensive double handling of materials involved, or a partial solution with significant remaining post construction settlement has to be accepted. Often the amount of surcharge load is limited by stability considerations.

The time scale for settlements, and hence for fully effective preloading, may be practically an order of magnitude larger for soft clay than for more permeable alluvium, say five years compared with half a year. This does not imply that preloading will have no benefits, if less than the ideal time is available; fortunately the non-linear nature of typical time-settlement characteristics of soft clays shows that about half the final consolidation settlement occurs in about one-fifth of the consolidation time. This means that significant benefits may be obtained on soft clays by preloading, possibly with the addition of surcharging, even if only a relatively short time is available.

Instances where even a quite limited time for preloading may be beneficial are at embankment-structure interfaces. For example, where an embankment includes a bridge, possibly over a river or road, then preloading the abutment areas will reduce, if not eliminate, the problems of downdrag and lateral loading on the bridge piles and also reduce the perennial problem of a bump at the bridge approaches.

A further example is at structures through embankments, such as culverts or underpasses, where the normal settlement may result in a pronounced longitudinal sag along the structure. This could well be sufficient to seriously impede the proper functioning of the structure and there are cases of box culverts of about 3 m by 3 m cross section, settling about 2 m at the centre, rendering them incapable of carrying the flow for which they were designed.

Possibly the most significant engineering advance in construction on soft clays in recent years in South Africa has been the development of engineered preloading and surcharging. A prerequisite for successful preloading and surcharging is a thorough site investigation so that the preloading scheme can be designed and the performance predicted. Since such techniques would normally be considered only where the subsoil conditions are difficult, it is practically axiomatic that accurate predictions of performance are correspondingly difficult. It is therefore necessary to monitor performance so that modifications to the loading plan can be made if required.

Performance monitoring generally consists of piezometers, settlement and slope indicators. Originally the emphasis was on piezometers, but this has shifted to strain measurements. Amongst the reasons for this are that piezometers reflect only the pore pressure changes in their immediate vicinity, and in a multilayered subsoil this could be misleading, also that some difficulties have been experienced with the more complex rapid response instruments.

It may be noted, although more relevant to monitoring for stability considerations, that the relative magnitude of lateral strains to vertical settlements provides a useful guide to performance. Such measurements can be taken fairly simply and shown in the chart form described by Matsuo and Kawamura[8]. Strain measurements alone do not provide the complete picture and generally a judicious combination of the various types of measurement is the optimum.

It should also be emphasized that any monitoring scheme is only as effective as the organizers allow it to be; in other words, the overall planning must take cognizance of the impossibility of providing accurate predictions and must therefore allow some flexibility in the construction programme. If this is not possible, and in some cases this will be so, then the preloading and surcharging may not be fully effective although they may nevertheless be worthwhile.

Many case studies have been reported of preloading schemes which have been successfully carried out; perhaps if these also analyzed the economic advantages an even more convincing demonstration of the benefits would be obtained and this could provide an impetus to more advanced forward planning for further schemes.

An excellent example of a well planned preloading and surcharging scheme is at the Bayhead in Durban Harbour. The works were carried out for South African Transport Services and consisted of the reclamation of about 80 ha of tidal swamps. The subsoil was about 5 m to 10 m of loose sand underlain by an average thickness of 10 m of soft clay, varying from zero to 20 m thick across the area. Dense sand underlies the clay. Large settlements were expected due to the reclamation fill and it was therefore intended that all heavy structures would be piled.

To minimize differential settlements and downdrag loads on piles at these structures, the fill was preloaded. Because of the advance planning, it was normally possible to avoid the use of sand drains to accelerate consolidation; at one structure, however, the Wheel Shop, a preloading scheme had to be effective in a six month period.

In this area the reclamation fill was about 2 m thick. Laboratory tests showed the coefficient of consolidation, c_v, to average about 1,6 m^2/year, with a range of about 0,4 to 2,8 m^3/year; these are fairly typical results, as shown in Table 3. Assuming one dimensional consolidation, the time for 90 per cent settlement would therefore be about 30 years,

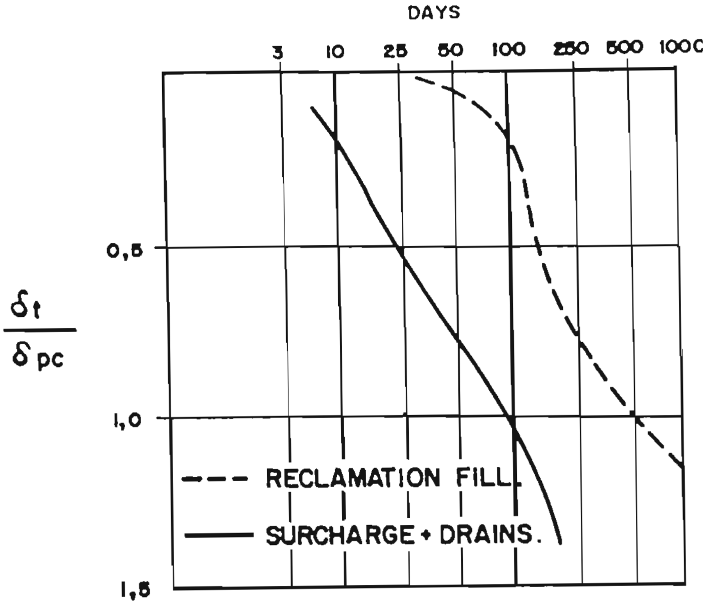

Fig 4: Effect of surcharge and drains on settlement ratio $\frac{\text{Settlement } \delta t}{\text{Primary settlement } \delta pc}$ against log time

which was unacceptable. If a surcharge fill about 2,4 m high was applied, the time necessary for the required settlement would have been about six years, which in this particular case was also unacceptable. Sand drains were therefore installed at 2,3 m centres to a depth of about 24 m.

It should be noted that sand drains rely largely on the horizontal permeability of the subsoil, whereas conventional settlement analysis utilizes the vertical permeability. In the layered estuarine deposits found in South Africa, the coefficients of consolidation in the horizontal direction are sometimes considerably larger than those in the vertical direction, and often at least slightly larger. This factor may be of significant influence in the design of a sand drain system.

Fig 4 illustrates the rates of settlement measured in the Wheel Shop area after surcharging and sand drain installation, and also the settlement measured at an adjacent area which was subjected only to the reclamation fill. Back calculations indicate that the coefficient of consolidation in the undrained area was in fact significantly higher than that determined in the laboratory, there being about one order of magnitude difference, and also that the sand drains decreased the consolidation time by about one order of magnitude. The results also illustrate the development of secondary compressions.

There has generally been a tendency either to ignore secondary compressions, or to make some nominal allowance for them. Accumulated evidence of embankment settlement records along the Natal coast demonstrates that secondary settlements can be significant, even within a relatively short period, and cannot be ignored.

Prediction of secondary settlement depends both on conventional, but long term consolidation testing — Rowe Cell tests in which pore pressures are measured are particularly useful — and on experience in the area. In some cases fairly large secondary settlements can be tolerated, but if they cannot, then the only feasible treatment is the suppression of secondary settlement development by surcharge loading.

Structures

Structures over soft clays are practically inevitably piled. This is largely because the piling industry has developed considerable expertise in construction in the coastal alluvial deposits, so that economically competitive foundation solutions are readily available. To a certain extent, whilst this is undoubtedly a tribute to the piling industry, it has mitigated against the adoption of alternative solutions, with the result that experience gained through performance monitoring of alternatives has not been developed, other than in a few isolated cases.

A possible exception is the use of vibroflotation, but it must be emphasized that the primary use of this, and similar techniques, has been in the more sandy deposits. Evidence from overseas and from local experience suggests that in soft clays a vibro replacement system can undoubtedly be effective, but the close spacing and large quantities of materials required for the stone columns may render the system less cost effective than well proven piling. The effectiveness of these techniques is limited by their depth limitations in the thicker clay deposits.

Similarly, the now well established technique of diaphragm walling has been used comparatively infrequently, although superficially at least, there would appear to be many situations where a technically superior solution may have resulted. Undoubtedly again, if the foundation problems can be solved by competitively priced conventional piling, there is little incentive to adopt other techniques.

There has been a growing use of slip coatings on piles to reduce the down drag forces caused by consolidation of the surrounding alluvium. The technique is well established and there can be little doubt concerning its efficiency. However, it is relatively expensive and at this stage can probably be justified only on medium to large projects; on smaller projects it is likely that designing the piles to take the downdrag forces will be more economical. Each project should be individually costed and as a very approximate rule of thumb it may be taken that the down drag forces can be reduced to about 10 - 20 per cent of the calculated values over the coated length of pile.

Lateral loads on piles due to subsoil consolidation are more difficult to predict and hence pose a much more complex design problem than that of vertical loads. In most cases a fairly conservative semi empirical approach should be adopted, such as that advocated by De Beer and Wallays[9].

For some structures which are to be built in areas underlain by soft clays, it is possible that a raft foundation may obviate the necessity for piling. A typical example would be a light industrial building for which the settlement criteria may not be stringent. The raft could then be simply a fill of appropriate thickness. The construction of the fill well in advance of the building helps to reduce the settlement which the structure will experience. Unfortunately, forward planning for most projects rarely allows advantage to be taken of such a preloading technique.

However, this method, coupled if necessary with surcharge loading, and possibly with systems to accelerate the subsoil drainage and hence consolidation settlements, may well offer a viable approach to difficult foundation problems far more frequently than has hitherto been attempted. Landfill methods using various waste materials can be used for reclamation projects in soft clay areas and these techniques may be of particular benefit where the soft clay zones are, as is so often the case, subject to flooding.

Field and laboratory testing

As already mentioned, classical soil mechanics to a large extent developed around the problems of soft clays. As a result, the field and laboratory techniques are well established and extensively described in the literature.

In many, if not most, of the South African soft clay situations, the clay is only one of the relevant alluvial strata, the others being silts and sands. An essential first step is to determine the relative extent both laterally and vertically of the different strata: this may appear to be an unnecessary statement of the obvious, but there can be no doubt that unduly pessimistic modelling of the strata can lead to gross exaggeration of the real problem, with correspondingly excessive construction measures being taken. An engineering geological contribution to the depositional sequence of the particular alluvium is essential and this can often be materially assisted by air photographs and input from other sciences.

Fig 2 illustrates the relevance of careful strata delineation. In this case the shear strength of the soft clay determined the stability of the proposed embankment. Due to the proximity of the sand, dissipation of excess pore pressures was expected to be fairly rapid in the highly stressed upper part of the clay and hence acceptable shear strengths would be attained in a reasonable time. Similarly, the overall predicted settlements, although large, were expected to occur in an acceptable time due to the vertical drainage both upwards and downwards into the sand strata.

On this basis no methods of accelerating drainage, or providing stability support, were incorporated. Fig 5, which shows pore pressures during construction, indicates that the strata modelling was appropriate and that the design and construction method decisions appeared to be

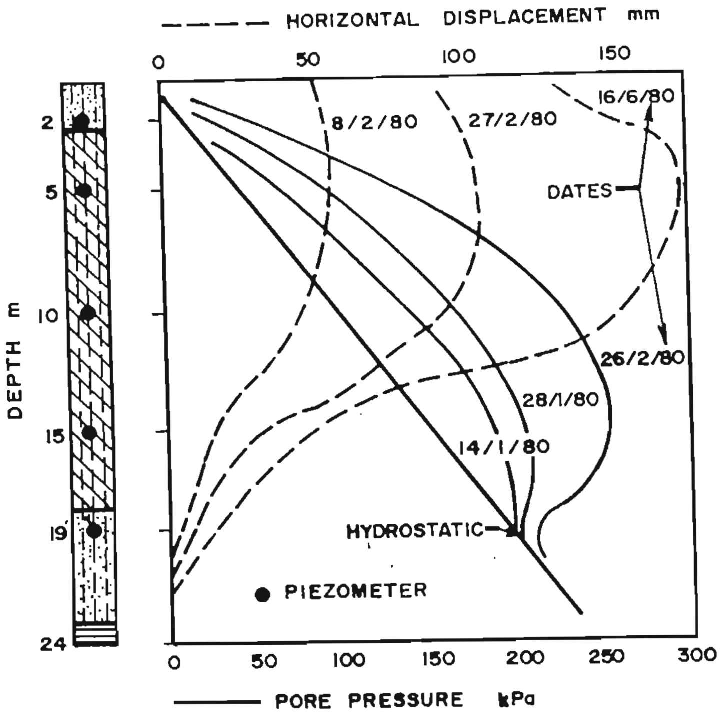

Fig 5: Piezometer and inclinometer data

justified and of considerable economic benefit.

Ironically however, although the predictions and early monitoring of performance led to what were thought to be reasonable decisions, the embankment has not in fact behaved as predicted, having settled a greater amount over a longer period. It must be presumed that the assessment of the coefficients of consolidation or the drainage paths were inappropriate. It is probable that the most significant problem was the modelling of the drainage paths. It should be noted that relatively thin clay inclusions within sand strata may radically alter the overall drainage pattern.

It may also be seen from the example, that since the settlement time is inversely proportional to the coefficient of consolidation, c_v, as opposed to the square of the drainage path length, little benefit is achieved through highly sophisticated testing to refine the accuracy of measurement of c_v, if the disposition of the strata is poorly established. This does not however imply that reliable representative values of c_v are not necessary.

Similar arguments can be advanced to demonstrate that for stability problems, little is gained by highly accurate shear strength measurements, if the strata delineation is not known. It may be noted that conventional stability analyses on soft clay will generally require both the undrained shear strength, c_u, and drained shear strength parameters, c' and $\varnothing'$. The former is best established from in situ testing, because sample disturbance of soft clays can be very significant unless thin walled piston sampling is used.

It is now generally agreed that Vane Shear Testing should be viewed as an index test, rather than as a measurement of a fundamental property of the clay. For this reason it is necessary to correct the measured vane shear strengths by a factor dependent on the plasticity as described by Bjerrum[10] and illustrated in Fig 6. The vane measured strength is multiplied by the appropriate factor to give a design field strength.

The drained parameters would almost invariably be taken as c' of zero, if there is no overconsolidation, and $\varnothing$ in the range of about 20° to 25°, for soft clays, but somewhat higher if there are significant silt and sand contents. Since the shear strength in terms of effective stresses is $(\sigma - u) \tan \varnothing'$, where σ is the normal stress and u is the pore pressure, it can be seen that a proper assessment of the pore pressure will be as significant as measurement of $\varnothing'$, and that any analysis will not be very sensitive to the relatively small range of $\varnothing'$.

The foregoing emphasizes the primary need for representative modelling of the soil strata, but it should not be read that there is no place for accurate soil testing; reliable testing is essential, but strata delineation is equally, if not more, important.

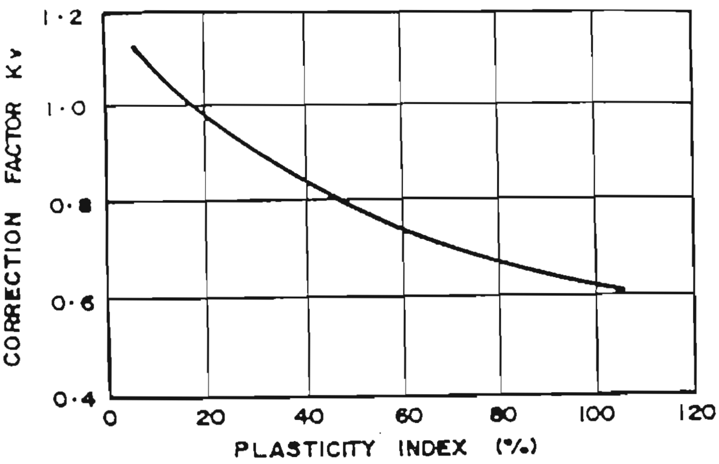

Fig 6: Vane strength correction factor for soft clays

Field Tests

Unquestionably however, the recent emphasis both in South Africa and internationally has been on in situ testing.

The well established techniques are Standard Penetration Testing (SPT) in boreholes, Vane Shear Testing either in boreholes or using a shielded rod system, and Cone Penetration Testing (CPT — formerly called Dutch Probing). A less frequently used in situ technique is permeability testing carried out by installing a piezometer either in a borehole, or occasionally by direct driving of the piezometer into the soil.

The recently developed techniques are the Piezometer Probe and the Self Boring Pressuremeter.

The Piezometer Probe (CUPT) described by Jones and Van Zyl[11] is a development, largely carried out in South Africa, of the familiar CPT. The CUPT system measures cone pressures and, simultaneously, the induced pore pressures during penetration. The nature of the soil can be established from the relationship between the cone and pore pressure (Jones and Rust[12]). The option may also be exercised of stopping penetration at any depth and measuring the rate of dissipation of the excess generated pore pressure. This is an in situ permeability test and coefficients of consolidation may be estimated from the results[11].

As earlier emphasized, strata delineation is an essential aim of any investigation; since one of the primary objectives of the strata definition is to differentiate between zones of different soil behaviour, for which the consolidation characteristics are vital, then it will be appreciated that the piezometer probe is a powerful tool.

Sampling of soft clays without causing significant disturbance is extremely difficult, unless thin walled piston sampling is utilized. If it is not, then for normally consolidated soils, errors caused by sample disturbance, which affect the compressibility parameters more than the shear strength parameters, result in over estimation of the compressibility. This may lead to over estimation of settlements and hence to unnecessarily conservative designs.

Conventional sampling also restricts sample size, so that triaxial tests are usually confined to 38 mm diameter samples and consolidation tests to 75 mm diameter samples. Although this is not a problem for homogeneous isotropic soft clays, many of the deposits do have significant layering, resulting in considerable anisotropy, affecting both the shear strength and consolidation characteristics.

The ideal situation is therefore to test soils in situ where disturbance can be minimized and sample size increased. The Self Boring Pressuremeter (SBP) was developed for this purpose (Wroth[13]). The test is performed in conjunction with normal borehole work and consists of advancing a tube into the soil to the test position and then inflating a membrane. The pressure require to inflate the membrane is measured at strain intervals.

The pressuremeter operates in a similar manner to a soft ground tunnelling machine in that the soil is excavated at the face and the machine is advanced simultaneously, preventing the yield of the soil into the excavation. The design and operation of the equipment are arranged to minimize soil disturbance. Pore pressures can be measured, and drained or undrained conditions tested. Elastic moduli and shear strengths can be derived.

Both the CUPT and the SBP are fairly sophisticated instruments in concept, but are well adapted for field use. In order to obtain the maximum benefits from them it is necessary to closely control their operation to decide what is the most appropriate testing in any situation. This has advantages, since it forces the geotechnical engineer into the field, and mitigates against the tendency for too much emphasis to be placed on complex analyses, rather than on thorough modelling of the real soil conditions.

Although the foregoing emphasizes the importance of in situ testing, particularly as recently developed, it must at the same time be emphasized that interpretation of the results may depend to a large extent on semi empirical correlations specific to local conditions, whereas the classical laboratory tests do not have this disadvantage.

Laboratory tests

There have been few advances in laboratory testing of soft clays. Laboratory equipment has become more sophisticated in that manual control of triaxial or consolidation testing can now be replaced by automatic computer controls; data recording and processing of results can also be automated, but essentially the tests themselves have remained the same.

In triaxial testing greater emphasis is now placed on the test procedure representing the loading conditions which will apply in the field, since it is now recognized that the stress path significantly influences the soil behaviour. A prerequisite is therefore to return the sample to the same state in the laboratory as existed in the ground before sampling. Complete stress path testing is complex and expensive, and can only be justified if minor refinements in design will produce large cost benefits.

A further modification of testing is based on the recognition that few soils are homogeneous and isotropic. This may be particularly so for sedimentary deposits where the shear strength, compressibility and permeability may be markedly different in the vertical and horizontal directions. Testing is therefore arranged to model the loading conditions which will apply in practice.

Consolidation testing has also changed little for a considerable time. The test has always been time-consuming, but essentially simple. Recent modifications have included automated data acquisition and processing systems. A significant advance has been the introduction of large diameter test equipment (150 mm) and the facility of permitting either horizontal or vertical drainage, and the measurement of pore pressures.

Fig 7 shows typical results of determinations of coefficients of consolidations from in situ permeability tests and small and large diameter laboratory tests for the soft clay described in Fig 2. The results show a marked apparent dependency of c_v, on effective stress and also indicate the large range of c_v obtained for very similar samples; it therefore also illustrates the dilemma facing the designer in selecting the appropriate coefficient of consolidation.

In the particular case illustrated there is a range of approximately an order of magnitude in c_v measured for samples obtained by piston sampling and very carefully conducted tests. It is in these circumstances that the strategy of using piezometer probing becomes attractive, since a large number of dissipation tests can be conducted. Clearly, however, the results of these tests can be interpreted only on the basis of calibration against laboratory tests and experience of real embankment settlements and back analysis.

It may be noted that there is often considerable difficulty in interpreting consolidation tests at low pressures because the test data do not always conform to the square root or logarithm time models. This may result in under-assessment of primary consolidation times and hence in an unconservative exaggeration of the influence of effective stress on the consolidation characteristics.

Consolidation testing provides both compressibility and consolidation data. The assessment of the former, expressed as either coefficients of compressibility, m_v, or compression indices, C_c, is generally accepted to be much more reliable than assessment of coefficients of consolidation. Moderate conservatism in estimating compressibility, and hence settlement, does not usually significantly change the evaluation of the potential problem, or the solution. Estimation of compressibility from conventional laboratory consolidation tests is therefore considered to be adequate, provided that some care is taken to minimize sample disturbance and to correct for any which may occur by the usual methods of interpretation of laboratory data (Schmertman[14] and Estimation of Consolidation of Settlement[15]).

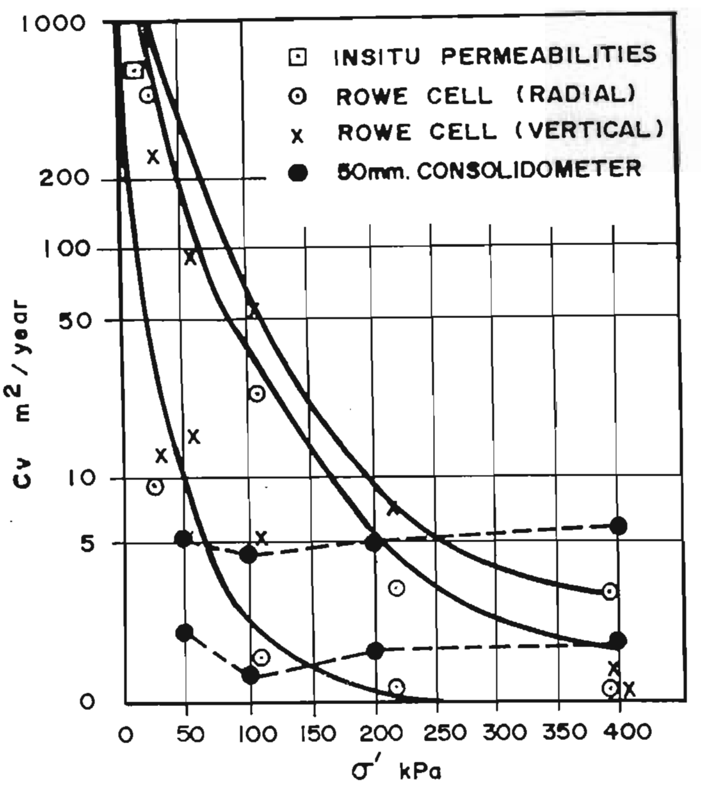

Fig 7: Coefficient of consolidation versus effective stress for various test methods

In many cases initial assessments of primary consolidation settlements can be made from the results of cone penetration tests (CPT). These give moderately realistic predictions for sandy alluvium, but the confidence level in clays is very much lower. Such assessments of compressibility should be viewed as qualitative rather than quantitative, unless correlations of CPT results with laboratory results, or with local experience, are available.

There is little published data in South Africa on the measurement of secondary compression characteristics of soft clays. There is of course no difficulty in carrying out the tests; they are simply longer term but otherwise standard consolidation tests. The problem is solely the testing time and hence expense and delay in obtaining data so that the convenience of using semi-empirical relationships dependent on more easily obtained data is attractive. This is unfortunate since the development of a local data base is prevented and no progress is possible.

Conclusions

Significant deposits of soft clays occur in South Africa, predominantly along the eastern coast. The clays are generally normally consolidated, or slightly overconsolidated, slightly to moderately sensitive and may be very soft. Stratification is often complex and extremely variable, resulting in realistic modelling of the strata requiring considerable investigation.

Stability and settlement problems for embankments are common and many structures have to be piled to considerable depths to provide adequate foundations. The costs of civil engineering works in areas underlain by the soft clays are therefore high and investment in proper investigation and analyses is more than justified.

It would appear that the South African soft clays do not exhibit any peculiar properties when compared to those described in the international literature. Investigation and analysis therefore follow normal international practice for these materials.

It is worthy of note that over the past two decades many embankments in South Africa have been monitored and a wealth of experience has been acquired. Whilst this has shown that many embankments have performed as predicted, there are numerous instances where this has not been the case. Unexpected secondary compression is one explanation for embankments continuing to settle significantly after the predicted end of consolidation.

Undoubtedly laboratory testing programmes should take cognizance of this problem and incorporate long-term consolidation testing. Measurements of pore pressures, such as can be carried out using Rowe Cell consolidometers, add extremely useful information. High quality undisturbed sampling is also essential and it is noted that thin wall piston samplers are available for this purpose.

The other, and possibly more likely, explanation for non-conformity with expectations, is that the overall consolidation characteristics as defined by the drainage paths are incorrectly assessed, a problem which is difficult to overcome in highly variable strata.

There has been a considerable emphasis both internationally and locally on in situ testing. The piezometer probe is an important addition to the range of techniques which are available. It is relatively low cost and provides a large amount of data on the stratification of the often multilayered deposits and on many of the soil properties. Much of this information is of a semi-empirical nature and it would be generally unwise to use in situ testing to the exclusion of conventional sampling and testing. It would appear, however, that the ideal combination consists of a basic programme of in situ testing, in conjunction with carefully selected high quality sampling and laboratory testing.

Techniques to overcome the problems of construction over soft clays include coating of piles to reduce down drag, sand or geofabric drains to increase consolidation rates and preloading and surcharging under structures and embankments. These methods have become more common and are likely to increase in usage. They are not inexpensive and the warrant for using them must always depend on the prediction of performance, with or without their use, and the relative costs. At present the prediction methods, although soundly based in theory, rely to a large extent on local correlations and experience. In South Africa this is fairly fragmented amongst the different practitioners and therefore the data base is weakened.

It is therefore asserted that the most significant advance in the art of engineering on soft clays in South Africa would be the synthesis of the knowledge and experience which now exists.

Acknowledgements

This review is based on information and advice freely given by colleagues Drs Loudon and Webb, Messrs Clark, Schwartz, Wrench and Yeats. Most of the data has been obtained from schemes carried out for the Department of Transport, the Roads Department of the Natal Provincial Administration and SATS. The support of these and other organizations has been invaluable in our understanding of the soft clay problems and in developing appropriate construction techniques.

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(xi) PIEZOCONE TESTING TO PREDICT SOFT SOIL SETTLEMENT

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Piezocone testing to predict soft soil settlement

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ABSTRACT: Piezocone testing is a versatile and comprehensive geotechnical investigation technique which is particularly suitable for recent alluvial deposits. In common with most in situ methods it is essentially a semi empiricial system for which the results require to be correlated with conventional geotechnical parameters measured by other means. Such correlations are described here for three major road embankments on the Natal coast which were constructed in the late 1970's and have both extensive initial laboratory test results and subsequent monitoring data. The correlations allow constrained modulus coefficients, α_m, and a cone time factor to be obtained from piezocone data so that coefficients of compressibility, m_v, and coefficients of consolidation, c_v, can be assessed for these alluvial deposits. The resultant α_m, 1,24, for these subsoils is shown to be significantly different from that quoted in the literature. This may be because the subsoils were very highly stressed and further research on this aspect is necessary.

1 INTRODUCTION

Over the past two decades the development of the national road system has led to a large number of high embankments being constructed over soft deposits. This is particularly so along the Natal coast where there are approximately forty freeway crossings of rivers, of which thirty involve a significant extent of alluvium. For reasons of road geometric alignment and provision of adequate flood capacity, the river crossings generally result in embankment heights in excess of 7m which often gives rise to settlement and stability problems.

These problems are never unforeseen and considerable efforts have been made to predict the behaviour of embankments, since this has a major impact on the satisfactory performance of the road as a whole.

In practically all cases the stability has been adequately ensured using the conventional geotechnical approach of undisturbed sampling, laboratory testing and limit state equilibrium analyses. Where stability has been shown to be a potential problem it has been overcome by any one of, or a combination of the techniques of flattening side slopes, adding berms, installing subsoil drainage and controlling the rate of construction.

Compared with stability assessment, success with the prediction of the amount and rate of settlement has been less impressive and, it may be added, this is not a problem confined to Southern Africa. In order to address the problem the authors, amongst others, have developed over the last 15 years the technique of piezocone testing (CPTU - cone penetration testing (CPT), with pore pressure (U) measurement). This is now a well established procedure internationally, albeit the theoretical modelling and interpretation are not as well developed as the sophisticated measuring, recording and data processing techniques. To a large extent interpretation remains semi empirical in that shear strength and consolidation parameters are obtained by comparison of CPTU results with other testing techniques or with performance measurements.

Three road embankments along the Natal coast, viz Umgababa, Umzimbazi and Umhlangane, Figure 1, have been extensively monitored since their construction in the late 1970's. This has provided a unique opportunity to back-analyse their performances to obtain settlement and consolidation parameters.

A research project (Rust and Jones, 1990) was sponsored by the Research and Development Advisory Committee (RDAC) of the South African Roads Board, which enabled the back-analysis and piezometer cone testing to be carried out. From the latter, settlement and consolidation parameters were derived and compared with the measured performance data so that appropriate correlation factors for these alluvial materials were obtained. This paper summarizes the findings of the research project and discusses the implications.

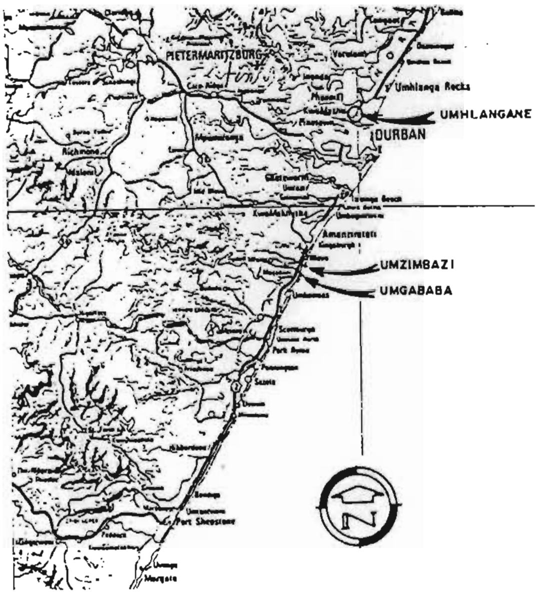

Fig 1 Site Plan

Table 1. Laboratory test results - indicators

AREA/DEPOSIT	MATERIAL DESCRIPTION	CLAY <2 μm (%)	SILT 2-60 μm (%)	LIQUID LIMIT w_L (%)	PLASTIC LIMIT w_P (%)	MOISTURE CONTENT w (%)
South Coast						
Floodplain	Light grey sandy clay	15	35	30	15	35
Backswamp	Black silty clay	25	55	45	25	50
Durban						
Estuarine	Mid-grey silty clay	50	40	55	35	55
Lacustrine	Black silty clay	65	30	65	30	90
Richards Bay						
Estuarine	Light grey sand silty clay	20	50	40	20	30
Estuarine	Dark grey silty clay	25	55	50	20	35
Lagoonal	Black silty clay	35	60	65	30	45

Table 2. Laboratory test results - shear strength

AREA	MATERIAL DESCRIPTION	SHEAR STRENGTH PARAMETERS		
		c_u kPa	c' kPa	
South Coast	Black silty clay	15-25	5	25
Durban	Black silty clay	10-25	0	25
Richards Bay	Sandy silty clay	15-35	10-20	25-30
	Black silty clay	10-20	5-15	20-25

Table 3. Laboratory test results - consolidation

AREA	MATERIAL DESCRIPTION	COEFFICIENTS OF COMPRESSIBILITY AND CONSOLIDATION	
		m_v (m^2/MN)	c_v (m^2/year)
South Coast	Light grey sandy silty clay	0,3-1,0	5-10
	Black silty clay	0,5-1,5	1-5
Durban	Black silty clay	0,5-2,0	0,5-2,0
Richards Bay	Light grey sand silty clay	0,2-1,0	5-10
	Black silty clay	0,5-2,0	1-5

1.1 Geology

The geology of the Natal coast has been extensively described by Brink (1985), King (1942), Orme (1974), Maud (1968), Moon and Dardis (1988) and at the particular sites of Umhlangane (Sea Cow Lake), Umgababa and Umzimbazi by Jones, Levoy and McQueen (1975), Jones, Rust and Tluczek (1980) and Jones and Rust (1981).

The coastal development of Natal has been predominantly along a narrow, relatively flat strip fringed by dunes of Berea red sands. Immediately inland, hills of shales of the Ecca Group and tillites of the Dwyka group of the Karoo Sequence are encountered. The hinterland topography becomes much steeper and more deeply incised. Further inland sandstone of the Natal Group (locally referred to as Table Mountain Sandstone) of the Cape Supergroup and granite occur.

The climatic N-value in the area where the rivers rise is generally 2 or less and rocks are deeply weathered and readily eroded. The N value as defined by Weinert (1964) is calculated from the evaporation during January, Ej, and the annual precipitation, Pa, viz

$$N = 12\,Ej/Pa$$

Hence where N is 2 or less the climate is hot and humid and rock weathering is mainly due to chemical decomposition. Due to changes in sea level the valleys have become drowned in the lower reaches and are characterized by lagoons with deep alluvial deposits. The sources of these deposits include granite, sandstone and shales and this results in multi-layered deposits of sands, silts and clays. The larger rivers viz Tugela, Umgeni, Umkomaas are deeply incised, have depths to bedrock of 50m or so and are predominantly sandy with basal boulder layers. The shorter rivers are much shallower, and generally have alluvial deposits of about 20m of sandy silts and clays.

The recent alluvial deposits are normally consolidated and laboratory testing does not show unusual characteristics. On the flood plains the surface deposits are often lightly overconsolidated due to desiccation. The soft clays which are the source of the major embankment problems comprise very soft to soft, slightly organic, medium to highly plastic, mid to dark grey, silty clays often containing shell fragments. The liquidity indices are often very high and the clays may be regarded as typical young clays. At two of the sites, Umgababa and Umzimbazi the subsoils are heterogeneous, having layers of intercalating silty sands and clays. At Umhlangane, however, the subsoil is a practically homogeneous very dark grey silty clay for the full depth of about 18m.

Typical laboratory test data from the original site investigations and Brink (1985) is given in Tables 1, 2 and 3.

1.2 Piezocone Testing (CPTU)

Piezocone testing has been described in many publications and perhaps the most useful recent compendium of information is that by Meigh (1987). The piezocone is distinguished from conventional cone penetration testing (CPT) by having a system for measuring the pore pressures generated by penetration of the cone. This data can be interpreted to considerably enhance the identification of the subsoil material type, the shear strength and the consolidation characteristics. A brief review of the relevant interpretation methods is given below.

1.2.1 Subsoil identification

A soils identification chart, Figure 2, has been drawn up by Jones and Rust (1982, 1983) based primarily on data from South African alluvial deposits. In this chart the dynamic pore pressure, u_d, viz the total pore pressure u_T measured during penetration, minus the hydrostatic pressure, u_o, ie u_T - u_o, is related to the normalized cone pressure (q_c - σ_{vo}), viz cone pressure, q_c minus the effective overburden pressure, σ_{vo}.

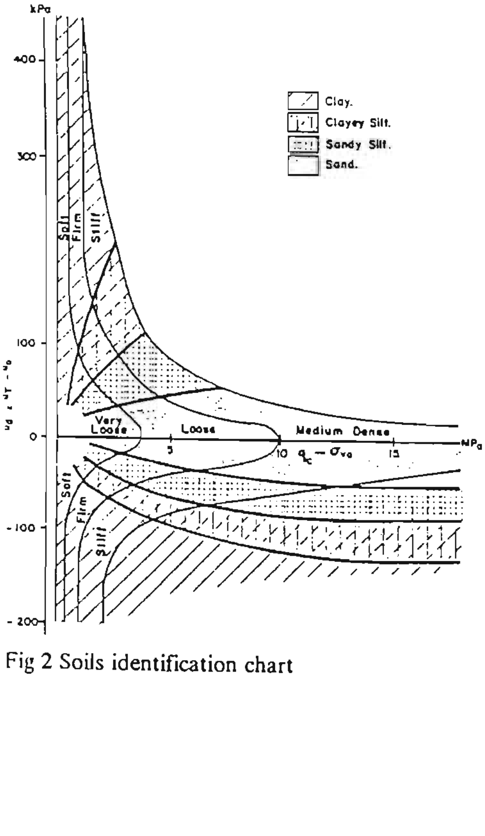

Fig 2 Soils identification chart

1.2.2 Undrained shear strength

The undrained shear strength is derived directly from the cone pressure, q_c, by using the following equation:

$$q_c = c_u N_k + \sigma_{vo}$$

where
- q_c = cone pressure
- c_u = undrained shear strength
- N_k = cone factor
- σ_{vo} = effective overburden pressure.

1.2.3 Effective shear strength

Various methods of assessing effective shear strength parameters from CPTU data have been presented by a number of authors and are summarised by Meigh (1987). The application of these techniques are fairly complex and fall outside the scope of this paper.

1.2.4 Compressibility

The conventional coefficient of compressibility, m_v, may be assessed directly from cone resistance by the following equation:

$$m_v = 1/\alpha_m q_c$$

where
- α_m = constrained modulus coefficient
- q_c = cone resistance

Values of α_m for a variety of soil types are available in the published literature and summarized by Meigh (1987).

1.2.5 Consolidation

Coefficients of consolidation, c_v, can also be obtained from CPTU results by relating them to the time taken for excess pore pressures to dissipate after ceasing penetration. Although theoretical solutions have been suggested for this relationship, Jones and van Zyl (1981) put forward a semi-empirical relationship for the Natal recent alluvial deposits. In this the time for 50% piezocone dissipation, t_{50}, was directly related to c_v on the basis of comparing these t_{50} with laboratory consolidation test data for samples taken from the same positions as the piezocone dissipation tests. Thus a cone dissipation factor was determined with a numerical value of 50 in the following equation:

$$c_v = 50/t_{50}$$

where
- c_v = coefficient of consolidation, m^2/year.
- t_{50} = half piezocone dissipation time, minutes.

2 PROJECT METHOD AND INVESTIGATION

2.1 Method

As stated previously comprehensive investigation and monitoring data was available for a period of longer than 10 years for three major embankments. Three information sets were available viz the original site investigation data, the subsequent monitoring information and the recent piezocone tests.

Using the monitoring data from the Umgababa embankment, the field parameters derived from the back-analysis were compared with the new CPTU information to obtain the appropriate correlation factors, ie α_m and the cone dissipation factor. These factors derived from Umgababa were then used with the CPTU data to predict the performances of the Umhlangane and Umzimbazi embankments. The CPTU predicted performances were then compared both to the measured performance and to the original investigation data in order to check the efficacy of the methods and factors.

2.2 Investigation

A total of 21 CPTU's were carried out in June 1989: 11 at Umgababa, 6 at Umzimbazi and 4 at Umhlangane. Of these, 6 were put down through the medians of the embankments. These therefore included the effect of the load on the subsoil. The remainder were either close to the embankment toes and hence also included the embankment influence, or deliberately away from the toes to exclude this effect.

The CPTU's were carried out using recommended international procedures and equipment. Dissipation tests were carried out at frequent intervals. Overnight dissipation tests were included to obtain the full dissipation records and also to definitively measure the ambient pore pressures under the embankments. These pressures were known to be in excess of hydrostatic since consolidation was not yet complete even after more than 10 years.

Using the soils identification chart and the borehole logs of the original investigations, geological sections were drawn up so that strata thicknesses could be accurately defined for settlement analyses.

3 ANALYSIS AND DISCUSSION

3.1 Umgababa Parameters

The subsoil compressibility and consolidation parameters for the Umgababa embankment were evaluated from the monitoring records. Information on the ambient pore pressures was obtained from the CPTU. Figure 3 shows the ambient pore pressure against depth and very clearly indicates that the clay layer had excess pressures.

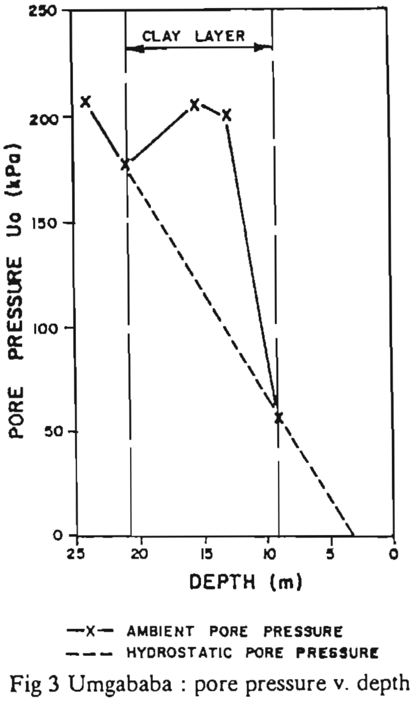

Fig 3 Umgababa : pore pressure v. depth

The detailed geological sections were obtained from the CPTU's and original investigation data. Figures 4 and 5 show a typical CPTU log and section.

From this data and from the construction and monitoring history it is possible to represent the embankment performance using a classical two-dimensional consolidation model. The steps for this process are summarized as follows.

At the cross section, Figure 5, the total thickness of clay is 14,2 m and the drainage path length of the major clay layer is 6,55 m. The embankment design height was 5,6 m, but because of ongoing settlement during construction (1,1 m) the final thickness of fill was 6,7 m, hence the increase in total stress was 147 kPa (6,7 x 22). The large embankment width to subsoil depth ratio results in there being practically no decrease of stress with depth.

The ambient pore pressure at 15,5 m depth was measured as 205,8 kPa and the water table was at 3,3m. Therefore the excess pore pressure, u_e, at this depth of 7,0 m into the clay, ie just below the half depth, was

$$u_e = 205{,}8 - (15{,}5 - 3{,}3)\ 10$$
$$= 83{,}8 \text{ kPa}$$

Hence the degree of consolidation, U_z, at that depth was:

$$U_z = 147 - 83{,}8/147$$

$$= 0{,}43$$

Using the Terzaghi consolidation theory and Taylor's chart relating degree of consolidation, depth factor and time factor T_v, the latter has a value of 0,33. From the Terzaghi relationship between T_v and percentage consolidation $\bar{U}$, it can be shown that $\bar{U}$ was 64,2% at the time of the CPTU investigation, at which stage the settlement was 2,69 m.

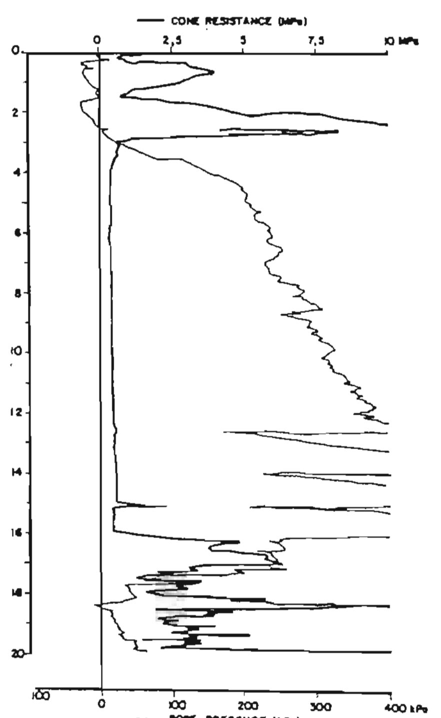

Fig 4 Umgababa : typical piezocone log

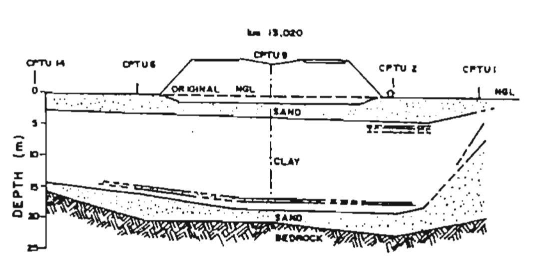

Fig 5 Umgababa : geological section

The present rate of settlement was also known, 7 to 8 mm/month, and the problem therefore becomes one of trying various combinations of c_v and m_v so that the best fit for both present settlement and present rate of settlement are matched. A spreadsheet approach was adopted to enable multiple iterations of combinations of c_v and m_v to be performed and Figure 6 shows a realistic fit of one of these combinations to the settlement record.

This gives the following compressibility and consolidation parameters:

$$c_v = 1{,}5 \text{ m}^2/\text{year}$$

$$m_v = 1{,}8 \text{ m}^2/\text{MN}.$$

The fit is not perfect in that the model predicts a 1989 settlement rate of 9,5 mm/month, whereas the actual is 7 to 8 mm/month, and also the measured settlements in the early stages are more rapid than predicted. However, no attempt was made to account for stress dependency of c_v on effective stress, or for changes of drainage path length with time, since it is believed that these complications would not add anything useful to the analysis.

These back-analysed c_v and m_v values are in agreement with the laboratory test values given in Table 3.

The model predicts that the 90% consolidation will take a total of 24 years and that the total settlement will then be 3,86 m ie a further 1 m of settlement has yet to occur.

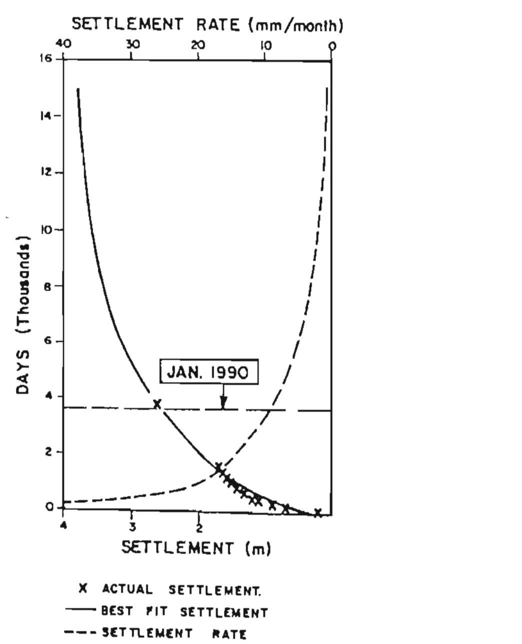

Fig 6 Umgababa : modelled and measured settlements

3.1.1 Constrained modulus coefficient, α_m, at Umgababa

It was then necessary to correlate the m_v deduced from the Umgababa performance records, viz 1,8 m^2/MN, with the piezocone pressures, q_c, at Umgababa. The average cone pressure in the clay layer for the relevant four CPTU's outside the influence of the embankment were very consistent, so it is considered reasonable to infer from these that the subsoil under the embankment had the same initial average q_c value of 0,45 MPa.

$$\text{From} \quad \alpha_m = 1/m_v q_c$$

$$= 1/1{,}8 \times 0{,}45$$

$$\alpha_m = 1{,}24$$

3.1.2 Cone dissipation factor at Umgababa

A similar correlation process to that given above was undertaken to compare the model consolidation c_v with the CPTU predicted c_v in order to obtain the appropriate cone dissipation factor. It was necessary to use the c_v obtained from the CPTU dissipation tests conducted outside the embankment. These results were, as may be expected, significantly different from those performed at higher stresses under the embankment. The average time for half dissipation, t_{50}, was 37,0. Therefore using the same form of relationship between t_{50} and c_v as that given by Jones and van Zyl for the measured t_{50} and the back-analysis c_v (1,5 m^2/year)

$$c_v = \text{cone dissipation factor}/t_{50}$$

$$\text{Cone dissipation factor} = 37{,}0 \times 1{,}5$$

$$= 55$$

This is in very close agreement with the value of 50 which was originally given by Jones and van Zyl.

3.2 Application of Umgababa CPTU correlation factors to Umzimbazi and Umhlangane

Using the correlation factors (α_m = 1,24; cone dissipation factor = 55) deduced in the foregoing from Umgababa it was then possible to predict settlements and consolidation times for Umzimbazi and Umhlangane from their CPTU data. These predictions could then be compared with the measured performances to check the reliability of the proposed factors and method.

3.2.1 Settlement

Umzimbazi

At Umzimbazi the settlement records, Figure 7, and measurements of excess pore pressure show that consolidation had ended and that the total settlement was 1,75 m. The average original thickness of the clay layer was 6,95 m and the height of fill at the end of construction was 4,6 m. The average CPTU cone pressure outside the embankment was 0,316 MPa, hence from α_m = 1,24, m_v = 2,55 m^2/MN and the clay layer settlement could be calculated as follows:

$$\delta = 4{,}6 \times 22 \times 6{,}95/1000 \times 1{,}24 \times 0{,}316$$

$$= 1{,}79 \text{ m}$$

and this compared with the measured settlement of 1,75 m of which about 1,66 m was due to the clay consolidation and 0,09 m due to sand compression.

It is interesting to note that an independent check on the amount of clay consolidation could be made by comparing the clay thickness outside the embankment with that under the centre of the embankment. The former, as already stated, was about 6,95 m, whereas that in the centre was 5,20 m. This suggested, assuming the clay layer was of constant thickness, that 1,75 m of settlement had taken place in the clay.

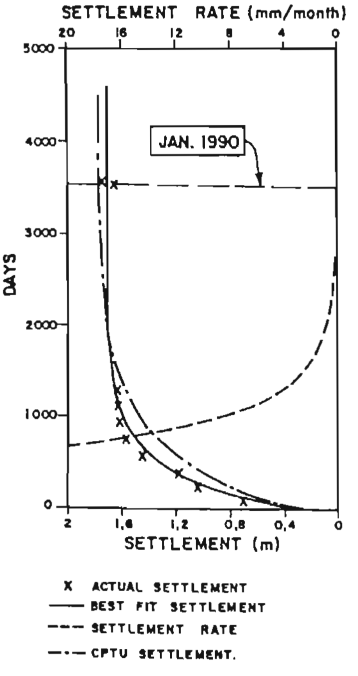

Fig 7 Umzimbazi : modelled and measured settlements

Umhlangane

The end of construction fill height, including compensation for settlement during construction, was 7,80 m. The average q_c value was 0,56 MPa. The thickness of the clay layer was 12,0 m and hence the calculated settlement was :-

$$\delta = 7{,}8 \times 22 \times 12{,}0/1000 \times 1{,}24 \times 0{,}56$$

$$= 2{,}94 \text{ m}$$

The 1989 settlement was 2,79m and the rate of settlement was about 1,0 to 1,5mm/ month which may be due to secondary compression, although the CPTU indicated that small excess pore pressures still existed. If a similar calculation was made as previously, using the Terzaghi consolidation charts or equations, then it could be shown that 90% consolidation had been reached at Umhlangane. Since the 1989 settlement was 2,79m, the 100% consolidation would be 3,10m. This suggests that a further 17 years would be required to achieve this settlement at 1,5mm/month, or that the total consolidation time will be over 30 years.

This is in conflict with the c_v assessment given in the following section which leads to a time of 20 years. These factors taken together suggest that the measured settlement should be that measured up to 1989 viz, 2,79m, plus say a further 1mm/month for 5 years ie 60 mm giving a total of 2,85m. This should be compared to the predicted value from the CPTU of 2,94m.

3.2.2 Consolidation time

Umzimbazi

The average half dissipation time, t_{50}, from the piezometer cone tests is 22,8 min. The average c_v could be predicted using the deduced cone dissipation factor (55,5):

$$c_v = 55{,}5/22{,}8$$

$$= 2{,}44 \text{ m}^2/\text{year.}$$

The time settlement data for Umzimbazi - Figure 7, shows a modelled best fit curve which gave a c_v of 4 m^2/year and an m_v of 2,45 m^2/MN. Figure 6 also shows the plot using the CPTU predicted c_v (2,44 m^2/year) and the CPTU predicted m_v (2,55 m^2/MN). It can be seen that although the fit is satisfactory in the later stages, it is poorer than the best fit model in the early stages. This lack of fit in the early stages is similar to, but more marked than that for Umgababa.

Umhlangane

Similar calculations were made for the Umhlangane data (t_{50} = 30 min) and these resulted in a CPTU derived c_v of 1,85 m^2/year, compared with back-analysis of the measured settlement which gave c_v in the range of 2 to 6 m^2/year.

4 CONCLUSIONS

Extensive laboratory testing was carried out at the sites of three embankments over recent alluvium. Summarized m_v and c_v data from both conventionally tested 50 mm diameter samples and from 150 mm diameter Rowe Cell tests are given in Table 4 together with the CPTU derived and back-analysis derived parameters.

The CPTU predicted settlements and the measured settlements (including assessed future settlements) are also given in Table 4.

4.1 Settlement

It can be seen that the settlement predictions based on the CPTU are very close, about 5% over-prediction. Therefore it is concluded that the constrained modulus coefficient for recent alluvial clay deposits is valid, ie:

$$\alpha_m = 1{,}24$$

It is noted that this value is much lower than previously used in Southern Africa or than is suggested in the literature (Meigh 1987). There is, however, no question that these embankments have settled much more than originally predicted using laboratory consolidation test data and this justifies the low α_m for these materials. What cannot as yet be deduced from the results is whether the value of α_m is valid for all similar alluvial deposits or whether there is some factor which makes these three embankments and their subsoils unusual. There is

Table 4. Measured and predicted settlement data

PARAMETER	UMGABABA	UMZIMBAZI	UMHLANGANE
Coefficient of compressibility m_v m^2/MN			
Lab 50 mm	1,67	0,82	0,53
Lab Rowe 112-224 kPa	0.93	1,06	0,47
Lab Rowe 224-392 kPa	1,53	0,92	0,39
CPTU predicted	1,80	2,55	1,43
Performance measured	1,80	2,36	1,38
Settlement, m			
CPTU predicted	3,86	1,79	2,94
Performance measured	3.86	1,66	2,85
Coefficient of consolidation, c_v m^2/year			
Lab 50 mm	0,71	2,1	0,68
Lab Rowe 112-224 kPa	2,60	3,8	0,47
Lab Rowe 224-392 kPa	1,40	3,7	0,52
CPTU predicted	1,50	2,44	1,85
Performance measured	1,50	4	4 - 6

no doubt that the three embankments were extensively monitored because it was believed at the design and construction stage that there were serious potential stability and settlement problems. The subsoils were relatively highly stressed, and this may well have led to high strains not due to consolidation alone.

The research project envisages extending the work to include further embankments where monitoring records are available and where the stress levels are lower. In this way the proposed constrained modulus coefficient, α_m, value can be confirmed or modified.

4.2 Consolidation Time

In Table 4 The comparison for coefficients of consolidation, c_v, is not as close as that for the compressibility parameters. It must be stressed, however that even with good laboratory testing and strata delineation, the reliability of the prediction of consolidation times is unlikely to be better than the range of say 50% below to 100% above the nominal predicted time. The project shows that reasonable values are obtained for coefficients of consolidation using the CPTU- derived expression, and this means that embankment performance predictions for recent alluvial deposits can be made from piezocone dissipation tests:

$$c_v \ m^2/\text{year} \quad = \quad 50/t_{50} \ (\text{mins})$$

The investigation described demonstrates the considerable benefits derived from using the CPTU for subsoil investigations. The emphasis has been on the numerical values of the CPTU-derived compressibility and consolidation parameters. The advantages of obtaining high quality subsoil identification and strata delineation are also most important since these too are essential for adequate predictions of embankment performance.

Whilst enthusiastically advocating the use of the CPTU, it is not suggested that the technique eliminates the need for conventional boreholes with sampling and laboratory testing. They should be seen as complimentary methods, although for reconnaissance level investigations, piezocone testing may well be sufficient.

ACKNOWLEDGEMENT

The work reported was carried out as Project No 89/14 for the Research and Development Advisory Committee of the South African Roads Board. The original investigations and designs for the embankments were carried out by Consulting Engineers van Niekerk Kleyn and Edwards, and A A Loudon and Partners. The subsequent monitoring has been primarily the responsibility of the former together with the Natal Roads Department. It is due to the foresight and cooperation of all these that the invaluable data was collected which enabled this project to be conducted. This emphasizes the vital contribution of performance monitoring to geotechnical engineering.

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(xii) A COMPARISON BETWEEN IN-SITU AND SEISMIC METHODS D SITE INVESTIGATION IN THE ALLUVIAL BEDS OF THE SABI RIVER, SOUTH EASTERN ZIMBABWE

Proceedings of the Eighth Regional Conference for Africa on Soil Mechanics and Foundation Engineering / Harare / 1984

A comparison between in situ and seismic methods of site investigation in the alluvial beds of the Sabi river, south eastern Zimbabwe

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SYNOPSIS : Site investigations for subsoil conditions can be done by means of diamond drilling, in situ probing and geophysics. A site investigation in the alluvial beds of the Sabi river required piezometer cone penetration testing and seismic regraction surveying along two section lines to determine the nature and depth of alluvium as well as the profile and consistency of the bedrock. The two methods are summarised and a comparison is made of their advantages and limitations, using examples from the investigation. It is concluded that the two methods complement each other and that a combination of both techniques was necessary for this investigation.

INTRODUCTION

The exploration of subsoils for a structure can be done directly by diamond drilling, by in situ penetration methods or indirectly by means of geophysical methods. The choice of method or combination of methods depends upon soil conditions, information required and economics. An investigation of the Chitowe Dam site in south eastern Zimbabwe required the combination of piezometer cone penetration tests and seismic refraction techniques to evaluate the subsoil and bedrock profile on two sections across the alluvium comprising the river bed. This paper summarises the piezometer cone penetration test (CUPT) and the seismic refraction method, illustrating the merits and limitations of each in relation to the investigation of the section lines at Chitowe Dam site.

PIEZOMETER CONE SYSTEM

One of the most widely used in situ testing techniques is the cone penetration test (CPT). In recent years the CPT equipment has been developed to monitor continuously pore pressures and penetration resistance. (Jones and Rust, 1982). The analysis of the CUPT data enables the identification of subsoil in addition to the parameters obtained from the conventional test. The method was developed semi-empirically by correlating CUPT data with independent soils classification tests.

The piezometer cone is basically the standard 35,7 mm diameter, 60° cone (Figure 1).

Strain gauge load cells measure the cone pressure and the total cone plus sleeve friction enabling conventional friction ratios to be deduced. A filter ring behind the cone enables the pore pressure to be monitored by means of a miniature transducer. The two load cell and pore pressure transducer outputs are amplified in the penetrometer tip and transmitted by cable through the rods to the surface. The surface control, with variable gains,

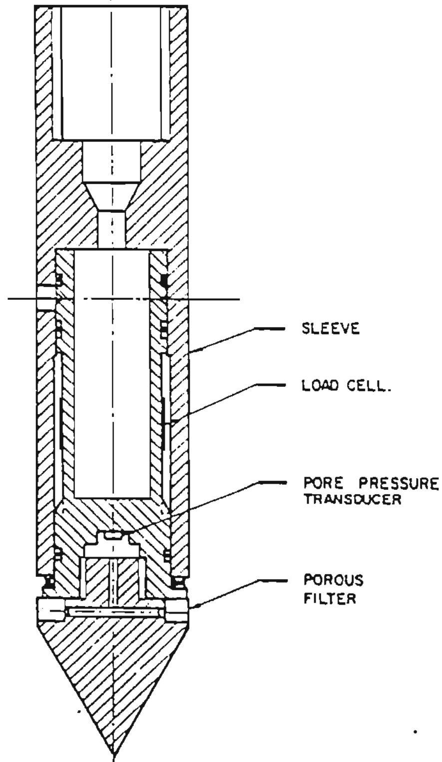

Fig 1 Piezometer Probe

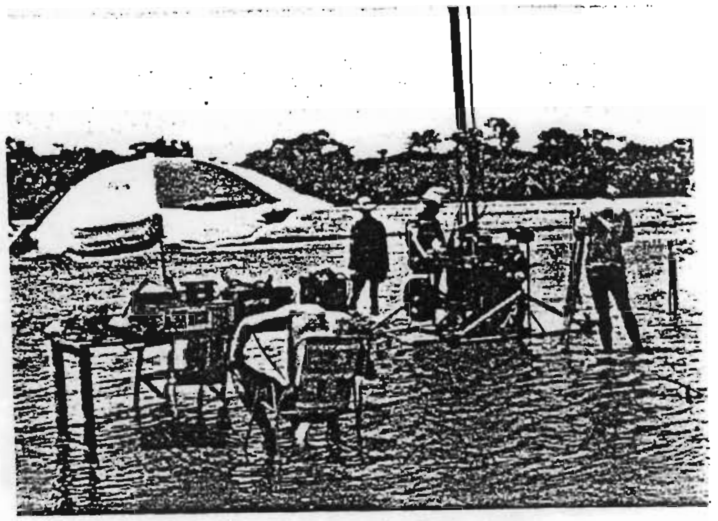

Fig 2 CUPT Test in Sabi river

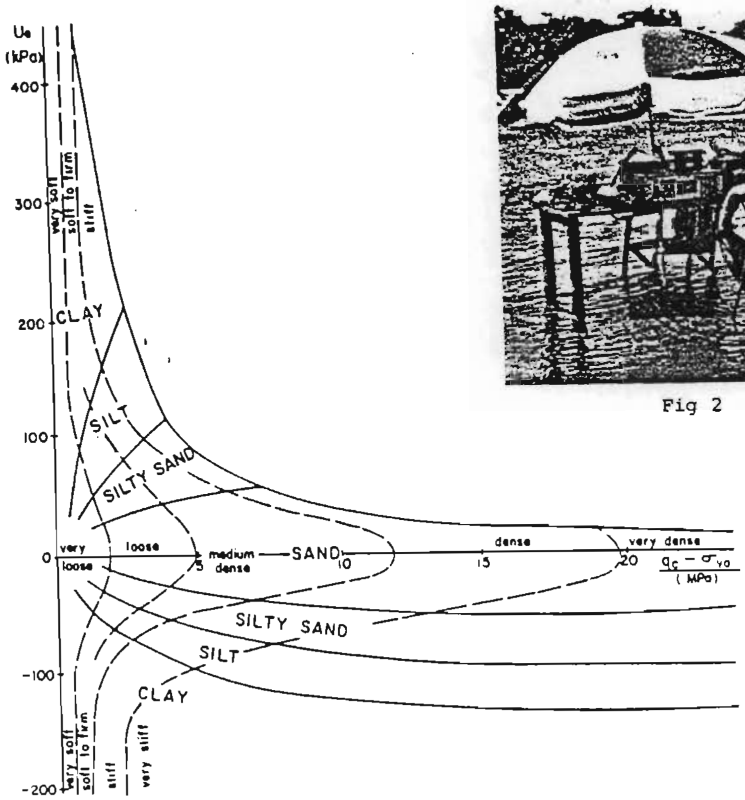

Fig 3 Soils identification chart

conditions the analogue signals and feeds them into a chart recorder and a data acquisition unit. An optical shaft encoder senses the depth of penetration (Figure 2).

A chart has been developed correlating the excess pore pressure (U_e), and the cone pressure (Q_c) minus the total vertical stress (σ_{vo}), against the soil type and consistency obtained from laboratory tests of undisturbed samples and examples cited in the literature (Figure 3).

The interpretation of the results of a CUPT test by means of the soils identification chart is shown on Figure 4. A detailed soil profile can be obtained in this manner to considerable depth in a matter of several hours for each test, The depth to bedrock is obtained directly by refusal of the probe.

SEISMIC REFRACTION SURVEYING

This method of geophysical exploration is well documented in the literature (Hawkins, 1961; Mooney 1977; Redpath, 1973) and has been used frequently in shallow geotechnical investigations. The refraction method consists of measuring the travel times of compressional waves generated by an impulsive energy source. The energy is detected by means of geophones and recorded on a seismograph. The instant of the shot is also recorded on the raw data which consists of travel time and distances. The time-distance information is then manipulated to convert it into the format of velocity variations with depth. In field conditions where there are discontinuities and variations in the soil profile it is necessary to use the continuous seismic refraction method to determine reversed seismic profiles. The traverses are arranged such that overlapping first arrivals from opposite directions are recorded for each refractor at each geophone location. This usually necessitates a minimum number of seven shots per array of twelve geophones, but depends upon the depth to the hardest refractor the number of intermediate refractors, and the spacing of the geophones.

The total length of each traverse is selected to suit site conditions and is normally a multiple of about four times the depth to the hardest refractor.

Various methods of interpretation are used depending on the amount of data available and the particular requirements of the

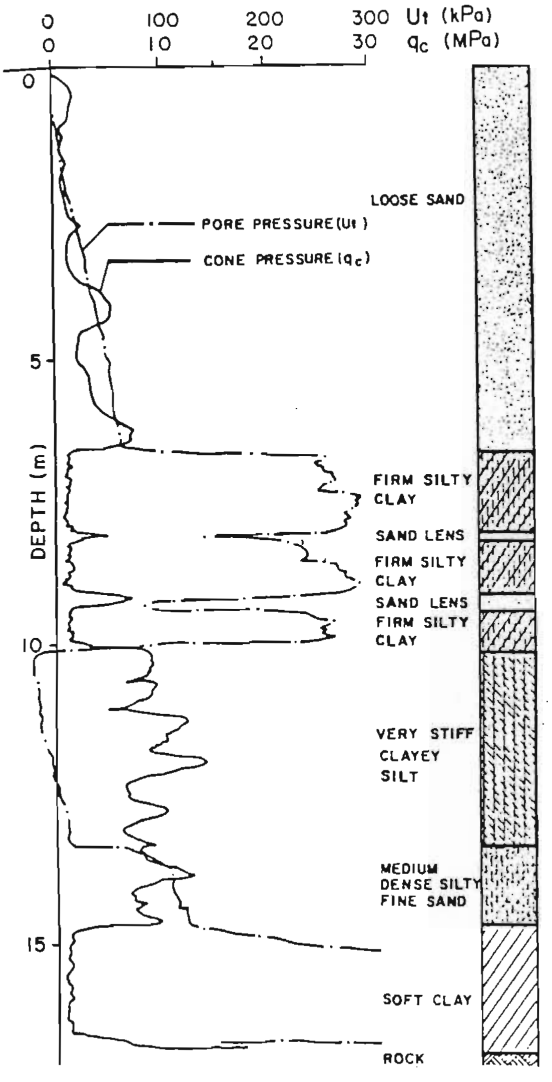

Fig 4 CUPT test interpretation

investigation. The commonly used "delay time" or "time depth" methods are described in the literature and prove to be relatively accurate if there are few abrupt changes in bedrock topography. The "delay time" is analagous to the time taken by a pulse to travel up or downward through a layer from one interface to another. If the delay time for each layer below a particular detector can be determined, then the depth beneath the detector to each layer can be computed. Thus by means of a continuous reversed time-distance plot over the full length of the traverse, depth information can be computed at each detector. The true velocity of the longitudinal waves for each layer can also be obtained from the continuous reversed time-distance plot. This is related to the consistency of the material and with prior knowledge of the geology, an assessment of rock quality can be obtained. The range of velocities for various materials has been documented in the literature (Jakosky 1957).

COMPARISON OF THE IN SITU CUPT TESTS AND THE INDIRECT SEISMIC REFRACTION SURVEY

Two section lines across the alluvial deposits forming the bed of the Sabi river were investigated by both methods. The CUPT tests were carried out at 25 metre intervals and geophone locations were at 5 metre intervals on sand banks, and 10 metre intervals over stretches of flowing water. Figure 5 shows the sections with the results of the seismic refraction survey superimposed on the detailed results obtained from the CUPT tests.

The sections show the nature of the bedrock profile below the alluvium that now forms the river bed. A deep paleochannel is evident on both sections and is associated with a doleitic intrusion into the sandstone country rock as confirmed by diamond drilling on a section line. The longitudinal wave velocity of the bedrock in the paleochannel zone was found to be low, which indicates a higher degree of weathering or fracturing.

The CUPT tests show that the alluvium consists predominantly of free-draining loose to medium dense sands with some layers and lenses of soft clays and stiff silts.

The detail from each CUPT test is correlated to construct an idealised section of the soil profile which has value in assessing the general nature and properties of the alluvium with a good degree of confidence. It is evident that on Line A the loose sands extended only to about 10 metres depth across the full width underlain by soft to firm silty clays, stiff to hard sandy silt and in the central channel, medium dense silty sand and firm to stiff silty clay. Line B, however, is shown to have clays and silts near to the banks only, and one isolated silty lens in the central channel, the remainder being sands.

The seismic refraction results are more generalised and identify only two refractors which are the saturated alluvium and the bedrock. The alluvium is only evident as a homogenous layer of constant velocity which has a slight variation across the sections. No identification of intermediate refractors is possible due to the limited lateral extent of the thin silty layers and lenses except on the left bank side of Line A. On this portion of the section it is interesting to note that the main refractor was identified at a relatively shallow depth, having a fairly low longitudinal velocity. When correlation with the CUPT tests was made it became evident that the refractor was coincident with a continuous layer of stiff to hard clayey and sandy silt about 5 metres above the true bedrock level as shown by refusal of the CUPT tests. The true velocity of the hard sandy silt was computed as 2950 m/sec which falls within the range of velocities for sandstone material. The underlying materials were shown by the CUPT tests to be medium dense sand and soft clay. This condition is termed "velocity inversion" and

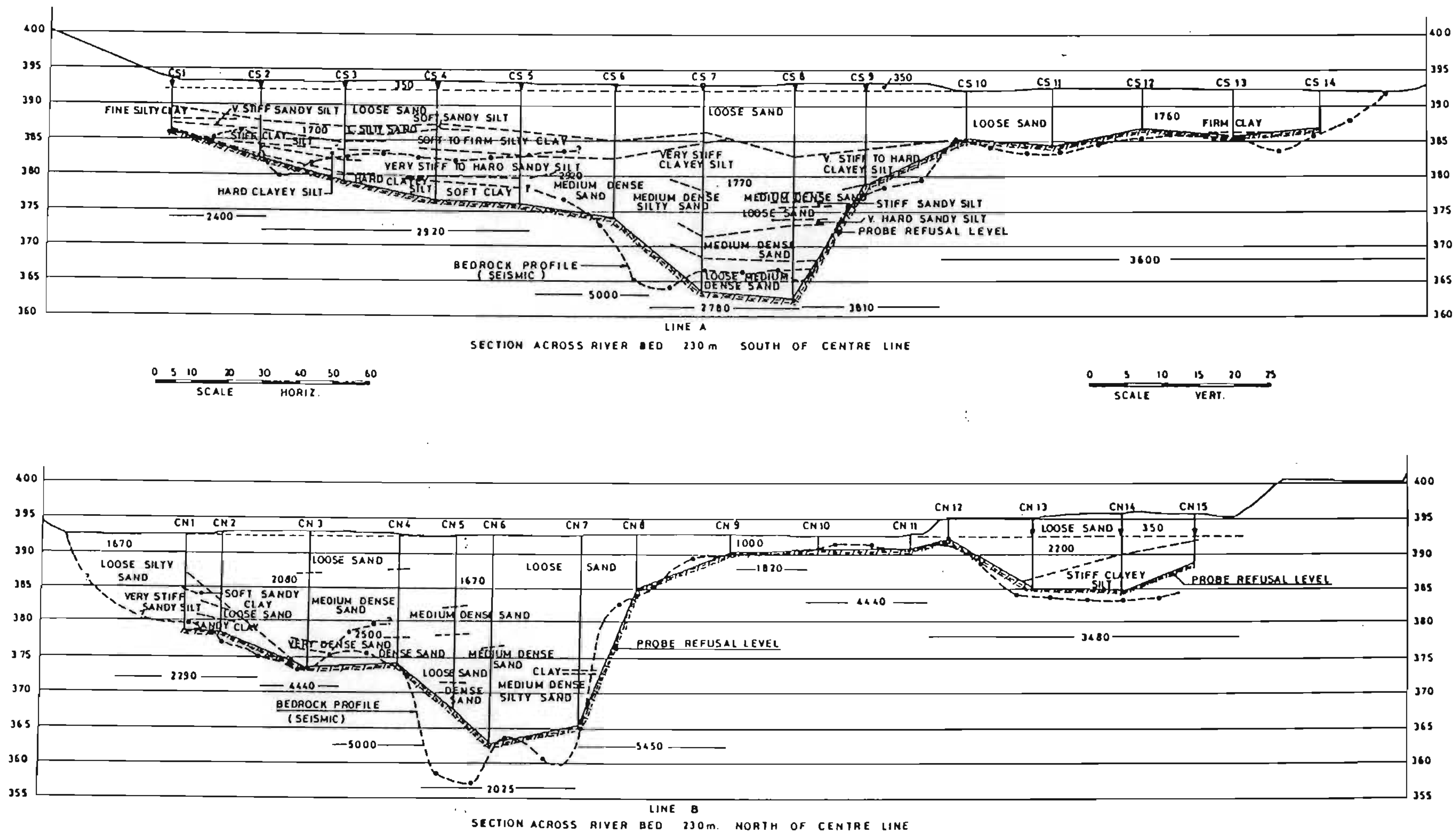

Figure 5 Sections across Sabi river comparing results from CUPT tests and seismic refraction survey.

the refraction method is unable to detect the presence of the lower velocity layers.

The second limitation is that the refraction method cannot detect thin layers or differentiate a refractor accurately if there is poor velocity contrast. The many thin lenses in section line A amply illustrate this limitation whereby the refraction method indicates a homogenous medium. The lack of velocity contrast between the alluvium and the sandstone bedrock is clearly evident in the paleochannel on both sections and hence the inaccuracy of the seismic refraction profile in these portions. The velocity contrast should be a factor of two for reasonably accurate depth predictictions or even to enable detection of an underlaying refractor. This offers an explanation for the inability of the refraction method to detect the underlying sandstone on Line A below the hard sandy silt layer of velocity 2950 m/sec because the velocity of the sandstone would have to be greater than about 6000 m/sec which is far above the normally accepted limits cited by the literature (Jakosky, 1957). This limitation is referred to as the "blind-zone", and is only eliminated in conditions where layers have thicknesses of the same order as the geophone spacing and where refractors have a velocity that contrasts sharply with the overlying material.

A discrepancy in the seismic results indicated a step in bedrock topography between CUPT tests CS5 and 6 which correlates with the point at which the hard sandy silt layer merges with a zone of medium dense sand, thereby accounting for the sudden loss of refractor continuity. A similar discrepancy was also evident on section Line B between CUPT tests CN3 and 4 and is explained by the presence of a very dense to dense sand layer of limited extent. These examples serve to illustrate certain limitations in the seismic refraction method.

The seismic refraction sections illustrate that the true velocity of the sandstone varies considerably. The true velocity is directly related to the consistency of the material and hence the rock quality can be estimated throughout the section. This information is of value to the engineer and geologist in assessing the probable areas of incompetent bedrock. The low velocity zones in the refractor are usually associated with major joints, faults, fracture zones or intrusions in the bedrock which can be explored more intensively by means of diamond drilling. It is obvious that the CUPT test indicates nothing about bedrock quality and depth information is only accurate in boulder free deposits. The linear continuity of bedrock topography is assumed between each CUPT test which implies that close spacing of tests is a necessary requirement for obtaining accurate profiles.

CONCLUSIONS

The piezometer cone penetration test is an in-situ test for soils which can be used to determine the consistency and type of soil by correlating cone pressures and pore water pressures using a materials identification chart. The method gives detailed soil profiles at each test position which can be used to obtain idealised geological sections. The method is limited to saturated field conditions which somewhat curtails its use in drier regions where partially saturated conditions are more common. The test is limited also in the inability to differentiate refusal due to obstructions such as boulders from bedrock level. The tests are conducted quickly in a matter of a few hours to depths of up to 35 metres, but the cost is relatively high due to the specialised equipment and personnel required.

The seismic refraction method is an indirect technique of obtaining subsoil conditions in terms of the depth and true velocity of the refractors in the vertical section. The true velocity is correlated to the consistency of the materials to give a generalised section showing the profiles of the refractors and the consistency. The method can be used in most field conditions and is not influenced by small boulders or obstructions. The method has the advantage of enabling an assessment of the bedrock quality to be made continuously across a section. The method is limited in the inability to detect low velocity layers underlying higher velocity layers and in the requirement for good velocity contrast between layers. The surveys can be conducted quickly covering up to ten traverses per day, making it cost effective in most site investigations.

The two methods of investigation complement each other in several ways. The depth of bedrock is measured accurately at piezometer cone penetration test positions but the seismic refraction method provides more continuous information with accuracies of about 10% except where it is limited by velocity inversion and poor velocity contrast. The penetration method gives detailed information on subsoils to bedrock level and the refraction method gives fairly detailed information on bedrock quality and continuity. Thus it is illustrated that both site investigation methods provide subsoil information, but that the combination of the two methods is more effective in presenting a more complete understanding of the subsoil conditions. It must be stated, however, that both methods are indirect and that they should be confirmed by diamond drilling.

ACKNOWLEDGEMENTS

The data has been extracted from a comprehensive report on the geotechnical investigation for Chitowe Dam site for the Ministry of Water Resources and Development, Zimbabwe. Particular thanks are expressed to Mr T C Kabell of the Designs Branch (M W R D) for encouraging the use of seismic exploration techniques and to S R K and Partners for providing the data.

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APPENDIX II

CONSOLIDOMETER-CONE RESULTS

APPENDIX II

CONTENTS

t	δH	$\delta H/H$
0,25	55	24
0,5	79	35
0,75	105	47
1,0	117	52
2,0	146	65
3,0	159	71
4,0	169	75
5,0	180	80
6,0	188	86
8,0	205	91
10,0	212	94
12,0	217	96
16,0	222	97
17,0	223	99
18,0	224	99
19,0	224	99
full scale 225		

SOIL	:	TSPC. Series 1
DATE	:	Jan 1975
LOAD	:	4 lb
DEPTH	:	-
SENSITIVITY	:	-
CHART SPEED	:	30 mm/min
RUN	:	

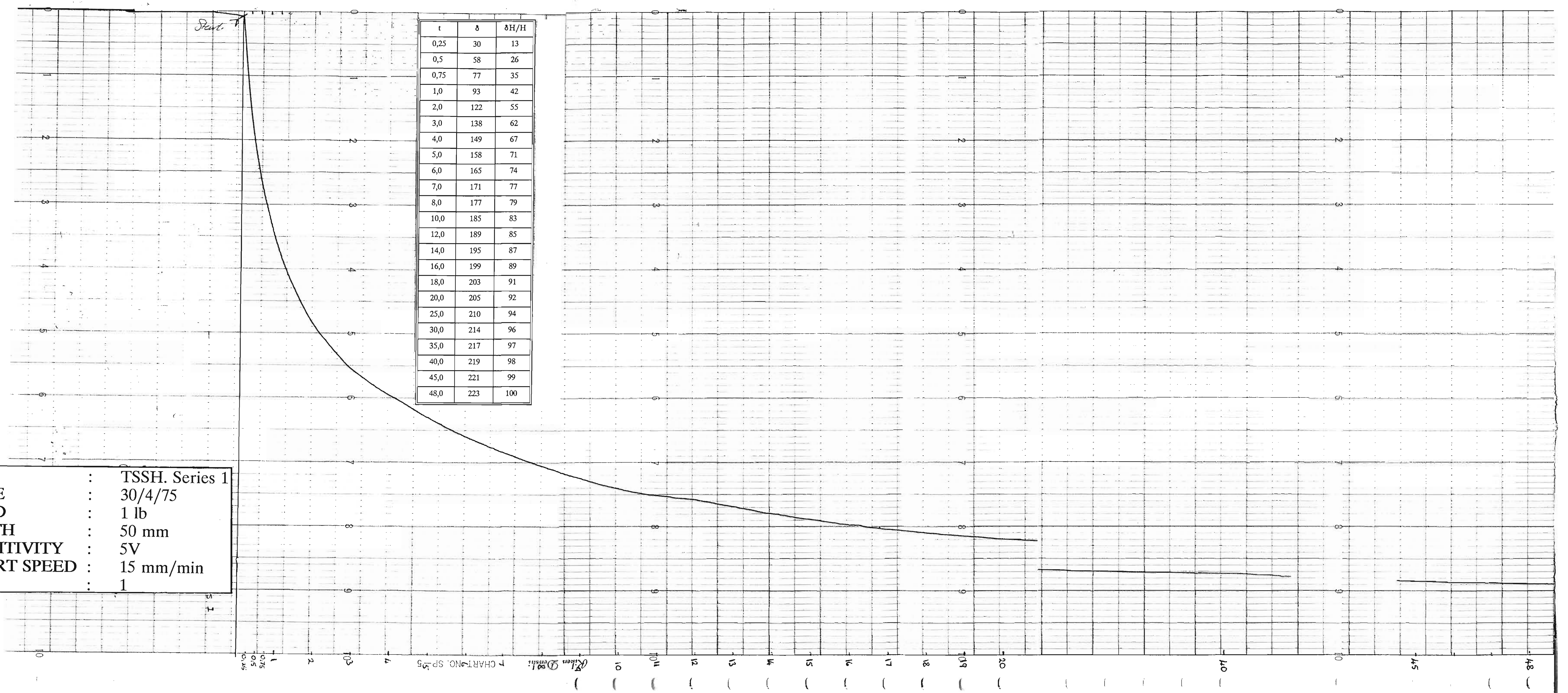
SOIL : TSPC. Series 2
DATE : 8/1/75
LOAD : 5 lb
DEPTH : 205 mm
SENSITIVITY : -
RUN : 3

t mins	δH mm	δH/H %
0,25	4	3
0,50	9	7
0,75	14	11
1,0	20	16
2	37	30
5	62	50
7	72	58
10	82	66
20	98	78
40	111	89
60	116	93
100	121	97
200	125	100

SOIL : TSPC. Series 2
DATE : 9/1/75
LOAD : 11 lb
DEPTH : 245 mm
SENSITIVITY : 5V
RUN : 5

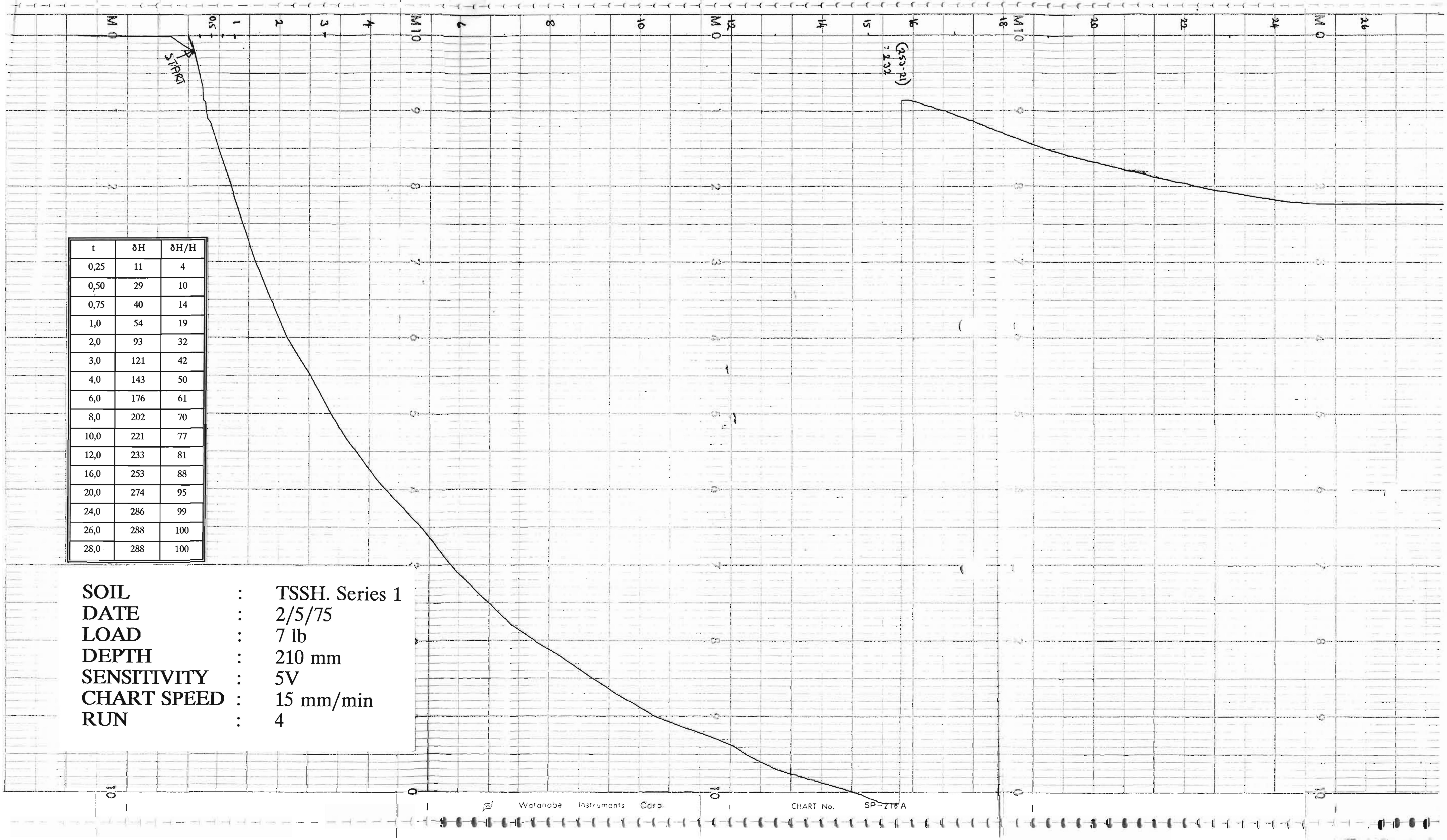
t mins	δH mm	δH/H %
0,25	7	9
8,50	9	11
0,75	18	23
1,0	24	30
2	38	48
3	45	57
4	50	63
5	53	67
6	56	71
8	60	76
10	63	80
15	66	84
20	68	86
30	73	92
40	74	94
50	76	96
60	78	98
80	79	100
97	79	100

SOIL	:	LPC. Series 2
DATE	:	29/1/75
LOAD	:	11 lb
DEPTH	:	255 mm
SENSITIVITY	:	5V
CHART SPEED	:	10 mm/min
RUN	:	6



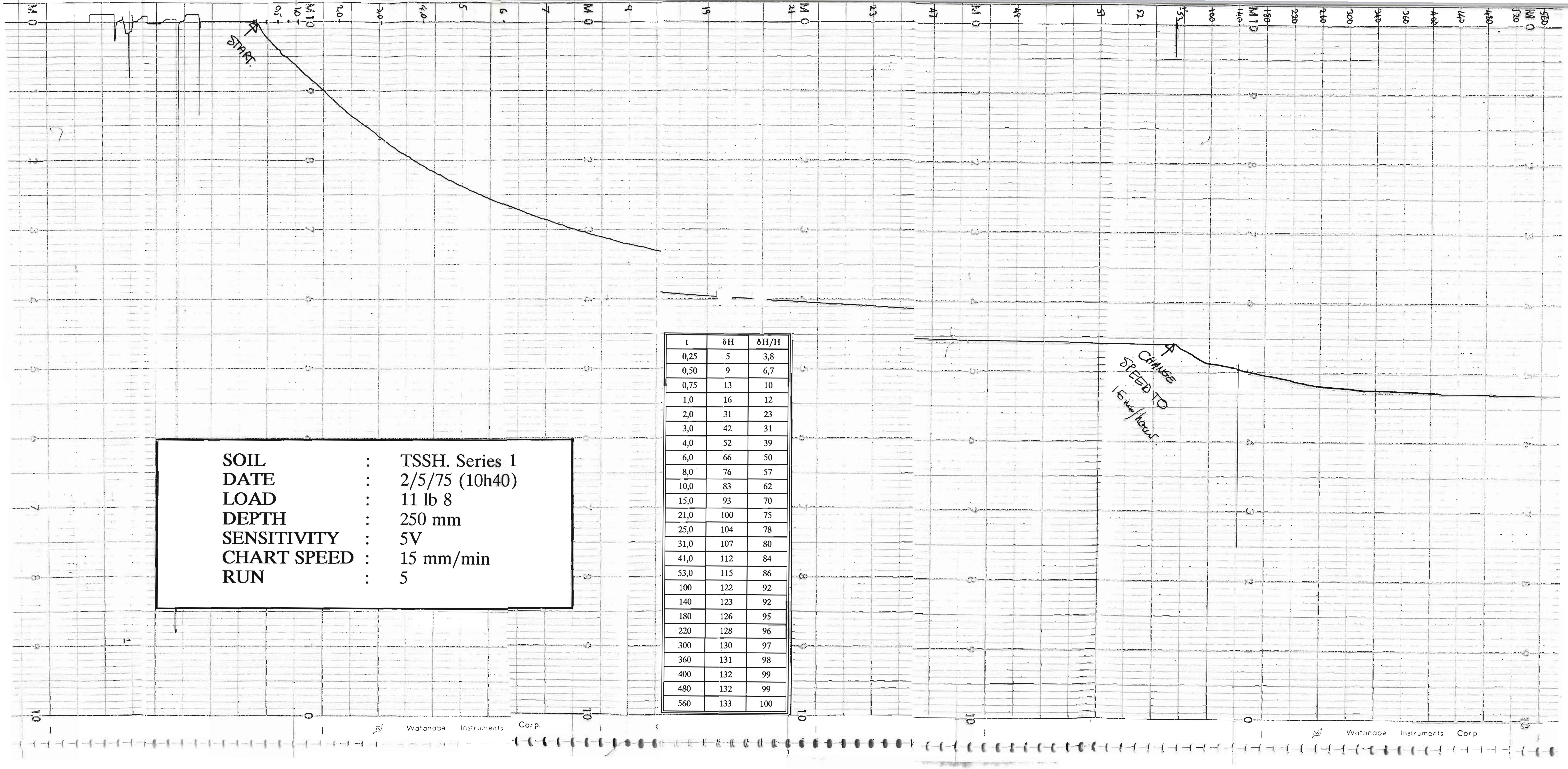
t	δ	δH/H
0,25	30	13
0,5	58	26
0,75	77	35
1,0	93	42
2,0	122	55
3,0	138	62
4,0	149	67
5,0	158	71
6,0	165	74
7,0	171	77
8,0	177	79
10,0	185	83
12,0	189	85
14,0	195	87
16,0	199	89
18,0	203	91
20,0	205	92
25,0	210	94
30,0	214	96
35,0	217	97
40,0	219	98
45,0	221	99
48,0	223	100

SOIL	:	TSSH. Series 1
DATE	:	30/4/75
LOAD	:	1 lb
DEPTH	:	50 mm
SENSITIVITY	:	5V
CHART SPEED	:	15 mm/min
RUN	:	1



t	δH	δH/H
0,25	11	4
0,50	29	10
0,75	40	14
1,0	54	19
2,0	93	32
3,0	121	42
4,0	143	50
6,0	176	61
8,0	202	70
10,0	221	77
12,0	233	81
16,0	253	88
20,0	274	95
24,0	286	99
26,0	288	100
28,0	288	100

SOIL : TSSH. Series 1
DATE : 2/5/75
LOAD : 7 lb
DEPTH : 210 mm
SENSITIVITY : 5V
CHART SPEED : 15 mm/min
RUN : 4



SOIL : TSSH. Series 1
DATE : 2/5/75 (10h40)
LOAD : 11 lb 8
DEPTH : 250 mm
SENSITIVITY : 5V
CHART SPEED : 15 mm/min
RUN : 5

t	δH	δH/H
0,25	5	3,8
0,50	9	6,7
0,75	13	10
1,0	16	12
2,0	31	23
3,0	42	31
4,0	52	39
6,0	66	50
8,0	76	57
10,0	83	62
15,0	93	70
21,0	100	75
25,0	104	78
31,0	107	80
41,0	112	84
53,0	115	86
100	122	92
140	123	92
180	126	95
220	128	96
300	130	97
360	131	98
400	132	99
480	132	99
560	133	100

APPENDIX III

GRAPHICAL EVALUATION OF SETTLEMENT RECORDS

GRAPHICAL EVALUATION OF SETTLEMENT OF RECORDS

Extensive use has been made of Asaoaka's (1978) method to determine c_v from the settlement records. This is briefly described below and illustrated in Figures III.1.a; III.1.b; III.2; III.3 and III.4

1. The observed time-settlement curve plotted to an arithmetic scale is divided into equal time interval, Δt (usually Δt is between 30 and 100 days). The settlement ρ_1, ρ_2 etc corresponding to Δt_1, Δt_2 etc are read off and tabulated. (Fig III.1.a)

2. The settlement values ρ_1, ρ_2 etc are plotted against ρ_{i-1}, ie as points (ρ_1, ρ_2) etc in a coordinate system of ρ_i, and ρ_{i-1}. The 45° line ρ_i - ρ_{i-1}, is drawn (Fig III.1.b)

3. The points plotted in (2) are fitted by a straight line. Where the line intersects the 45° line gives the end of primary consolidation settlement. The slope of the line B_1, is given by:

$$c_v = \frac{-5}{12} d^2 \frac{\ln B_1}{\Delta t}$$

4. i) The plotted line will become straight only after the end of construction and represent only consolidation.

   ii) Sometimes two lines can be fitted with the second representing secondary compression.

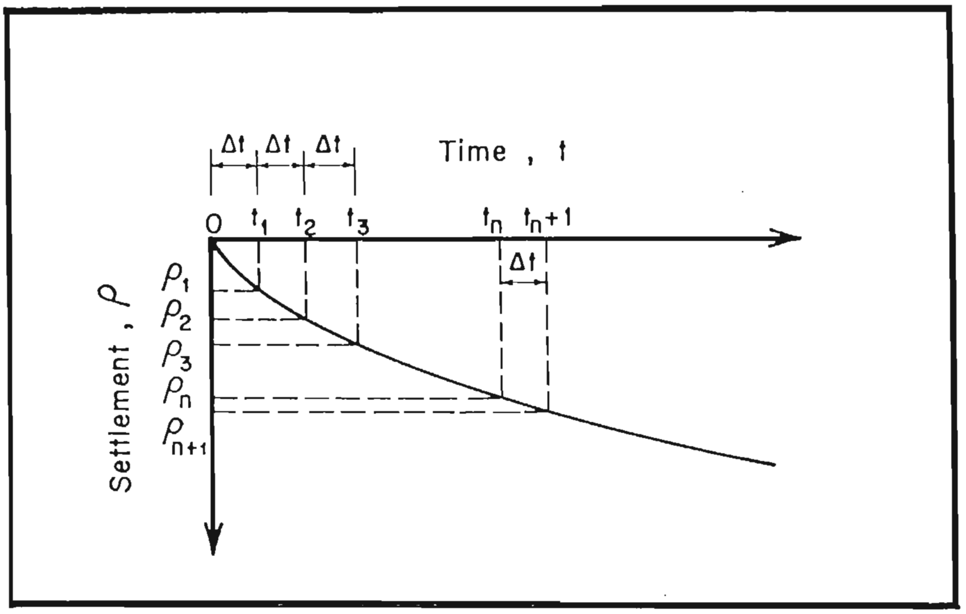
   iii) The method is also applicable for embankments where vertical drains have been used, in which case the dominant drainage length will be the effective drain spacing.

The significant advantages of this method are that it is theoretically correct and it does not require any knowledge of the final settlement.

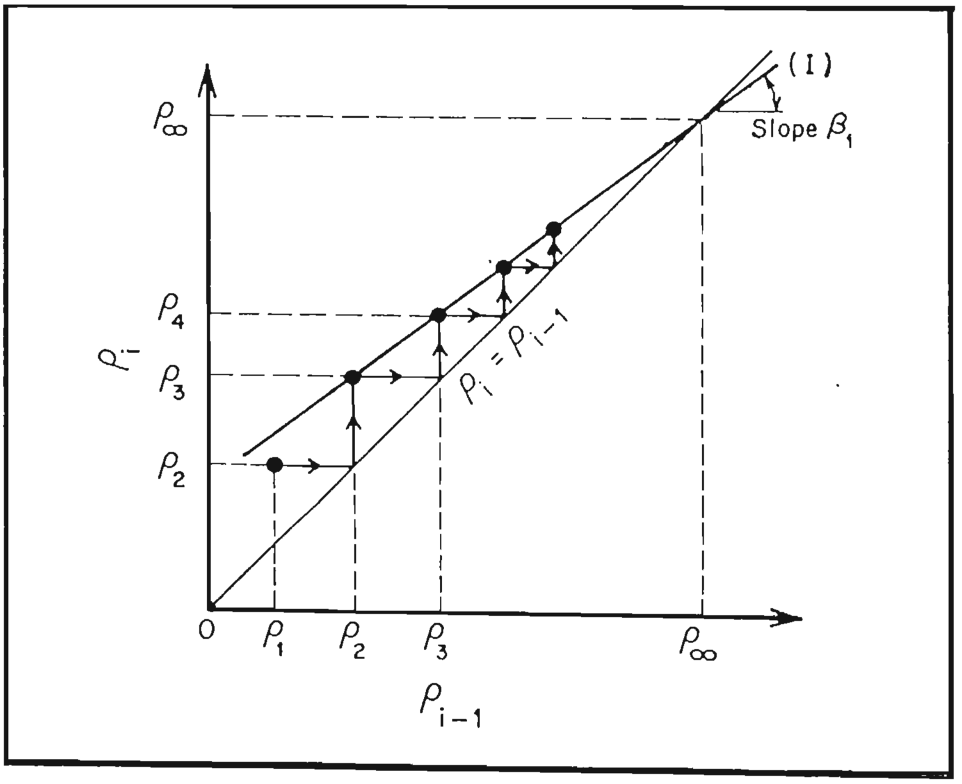
As with any consolidation analysis the evaluation of c_v is dependent of the square of the drainage path length hence this length must be assessed with considerable care. The method clearly separates any primary consolidation and secondary compression and where the latter is significant, it can be deducted from the total settlement for evaluation of α_m.

Asaoka graphical analyses were performed for every embankment and two of these for the Bot River data at different stages of construction are shown as examples in Figures III.2, III.3 and III.4. It may be noted that a linear regression line is fitted through the data points. The slope of the line and the point of intersection with the base line (45°) can be calculated, or obtained graphically, to give B_1, for use in the equation in (3) to give c_v and also to directly give the settlement, hence time, at the end of consolidation.

In the examples the horizontal and vertical scales are not equal hence the so called 45° line is not at 45°; this is taken into account in the calculation of the slope B_1.



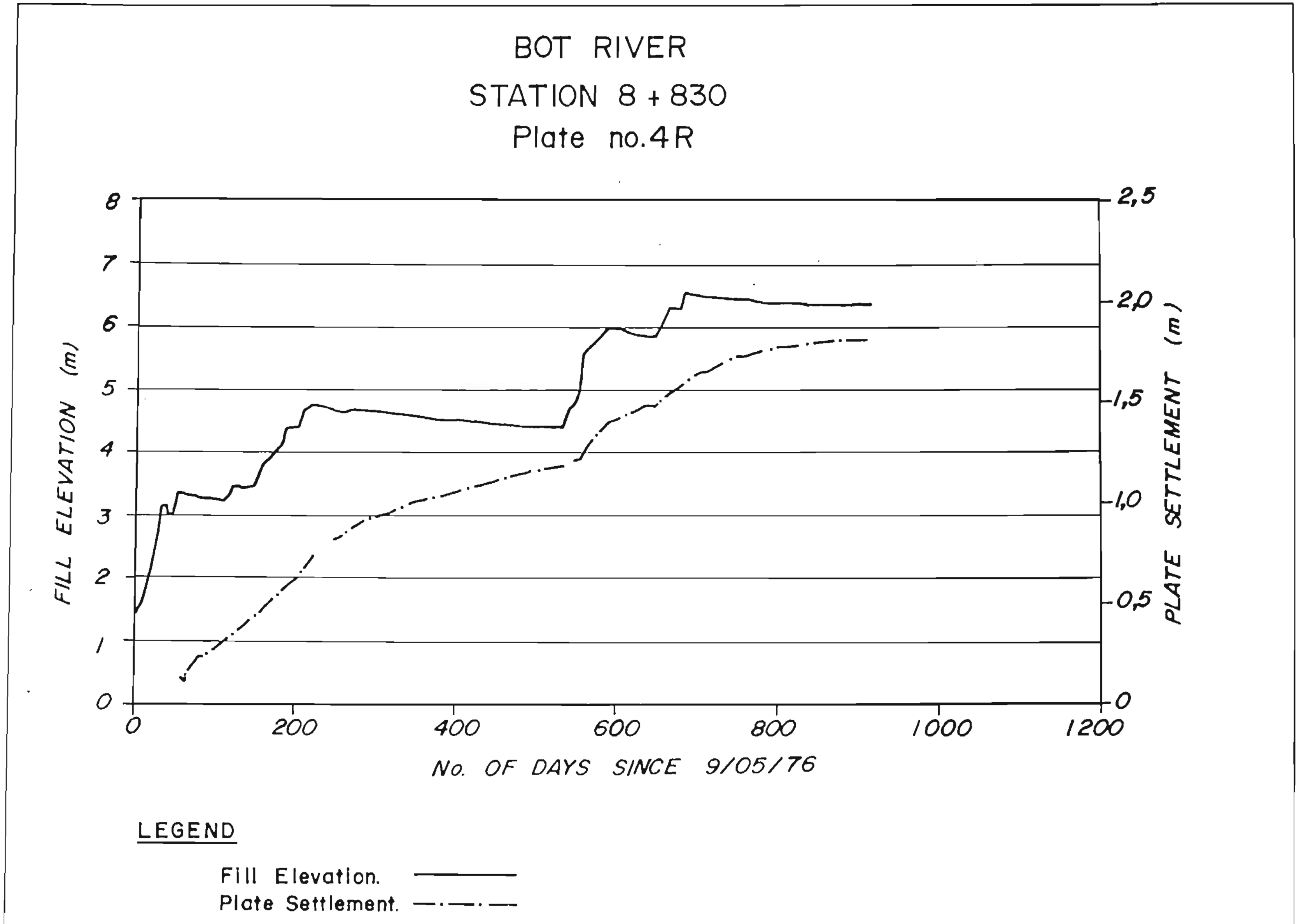
a) PARTITION OF SETTLEMENT RECORD INTO EQUAL TIME INTERVALS



b) PLOT OF SETTLEMENT VALUES AND FITTING OF STRAIGHT LINE

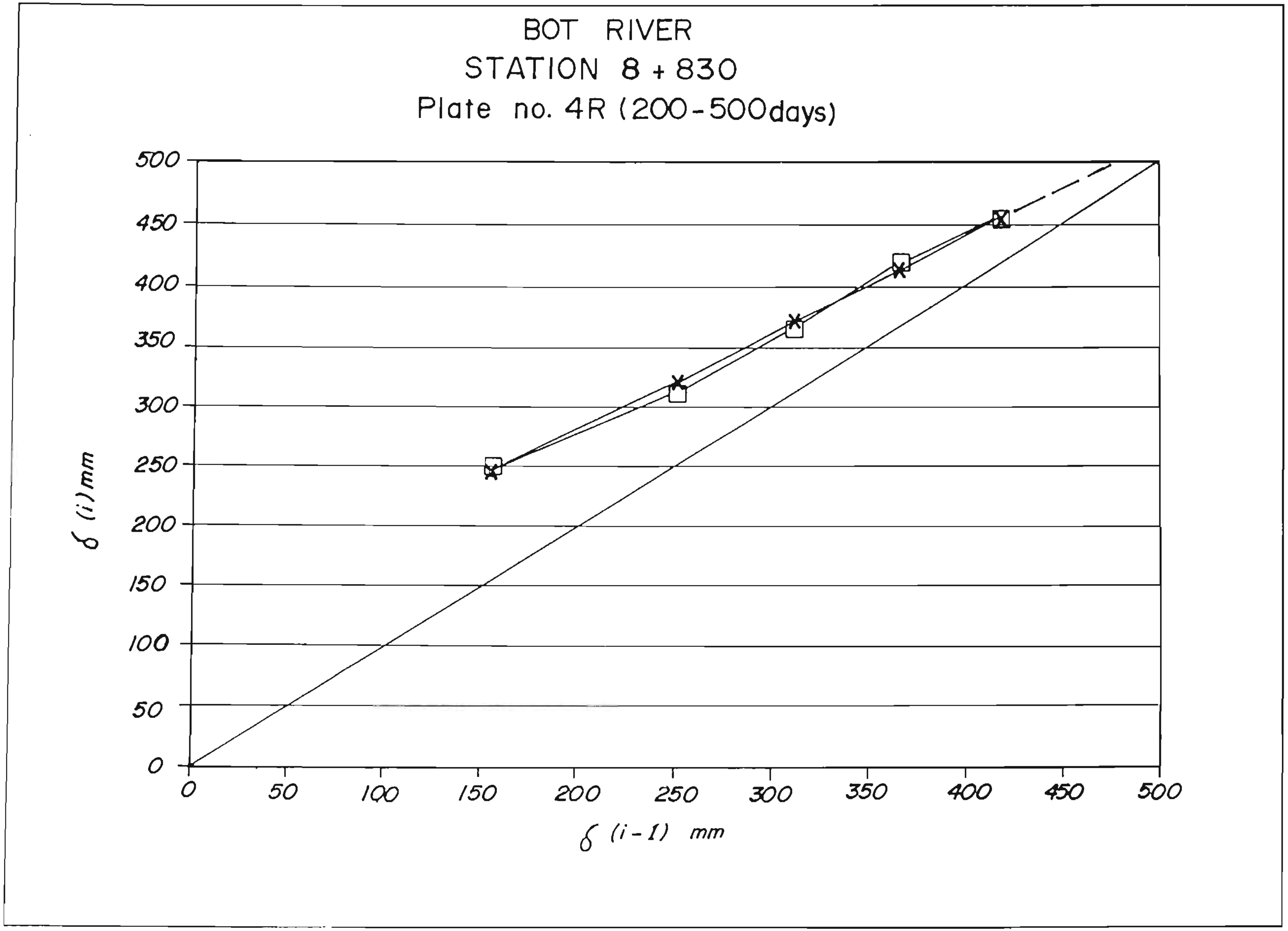
STEPS FOR THE USE OF ASAOKA'S METHOD

FIGURE No. III . 1



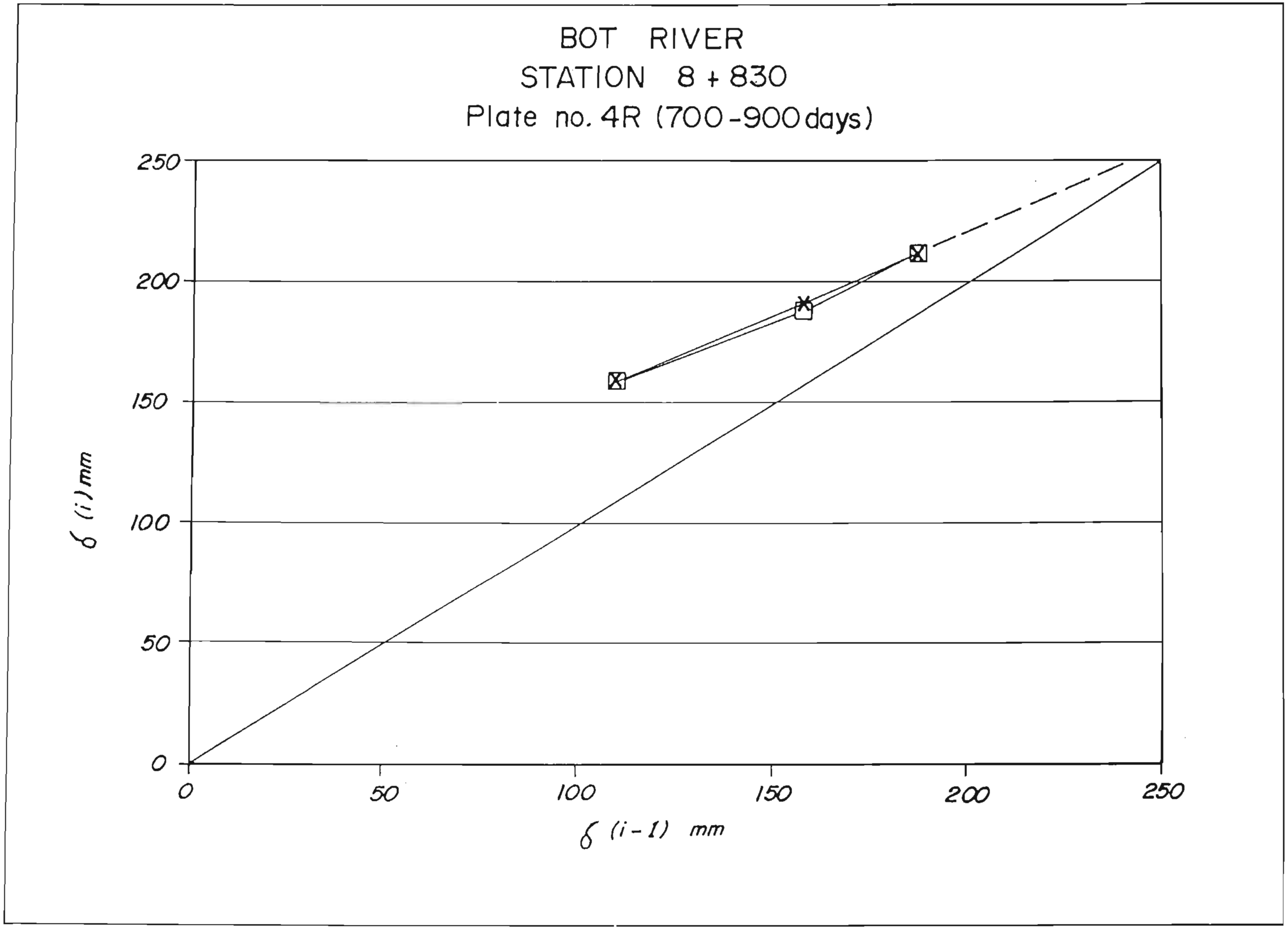
BOT RIVER SETTLEMENT RECORDS

FIGURE No
III . 2



ASAOKA PLOT

FIGURE No
III .3



ASAOKA PLOT

FIGURE No III . 4