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# TECHNICAL REPORT

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Summary

Veritas has been requested by Conoco Norway Inc. to perform a planning study for a research program which will have the objective of improving the understanding of the pile-soil interaction resulting from cyclic tensile loading as produced by a deepwater Tension Leg Platform. The study has been divided in seven tasks.

Veritas has been carrying out the planning of a series of laboratory model pile tests to be integrated in a large scale field test program. The planning of the field test program has been subcontracted to Ertec, Inc. Houston.

This final report presents the results of Veritas' work in the planning study. The work performed by Ertec, Inc. is presented in a separate report dated 28. August 1981.



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PREFACE

The tension leg platform (TLP) concept is presently being considered by Conoco for deep-water locations in the Gulf of Mexico, specifically Green Canyon, Blocks 137 and 184 and Viosca Knoll, Blocks 864 and 908. The soil conditions at these locations will be predominantly soft clay down to a considerable depth and will thus be completely different from the Hutton Field in the North Sea, for which Conoco is presently designing the first prototype TLP, where alternating layers of very dense sand and heavily overconsolidated clays are found.

The foundations of TLP's will have to resist large tensile forces with combined static and cyclic loading. Pile foundations will be the most feasable alternative practically and economically. The behavour of tension piles is, however, poorly understood at present.

In order to improve the understanding of the pile-soil interaction resulting from cyclic loading produced by a deepwater TLP, taking into account any site spesific aspects of the above mentioned Gulf of Mexico Blocks, Conoco authorized Veritas with Ertec as a designated subcontractor to perform a planning study to produce a recommended procedure for conduction such a research program.

The planning stydy was split in 7 different tasks as follows:

Task 1 - Review of Pertinent Literature
Task 2 - Selection of Analytical Methods
Task 3 - Laboratory Test Planning
Task 4 - Feasibility of Field Tests
Task 5 - Recommendations for Field Test System
Task 6 - Schedule and Budget
Task 7 - Final Report

Task 3 was the responsibility of Veritas alone and Task 4 and 5 was covered by Ertec. The remaining tasks were to be covered by Veritas with respect to laboratory model pile tests and by Ertec with respect to field pile tests.

By contract of June 11, 1981 between Conoco Norway, Inc. and Veritas authorization was provided for the tension pile planning study. Ertec, as a designated subcontractor for the planning of field tests, entered into contract with Veritas on June 18, 1981.

The Norwegian Geotechnical Institute and the Division of Soil Mechanics and Foundation Engineering, the Norwegian Institute of Technology in Trondheim (NTH), were assigned as consultants by letter of August 19, 1981, with the objective to give adivse and comments to the planned field and laboratory tests.

Veritas' main responsibilities within the project has been the planning of laboratory model pile tests, the integration of these tests in the overall research program and the administraion of the planning study.

The results of the planning of the laboratory pile test program are presented in this final report. The planning of the field test part of this study was carrried out by Ertec and is presented in their final report dated August 28, 1981.

Comments from NGI and NTH will be enclosed as an addendum to this report as soon as they have completed their review of the final reports.

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# EXECUTIVE SUMMARY

## Introduction

Conoco Norway Inc. is sponsoring a research program with the objective of improving the understanding of pile-soil interaction during cyclic tensile loading. The first part of this program was a three month planning study to produce a recommended procedure to conduct such a program. Aspects of the planned program found to be site specific was decided to be applicable to Gulf of Mexico Blocks 864 and 908 of Viosca Knoll and Blocks 137 and 184 of Green Canyon.

VERITAS has been in charge of the overall execution and administration of the planning study, with Ertec, Inc. working as a designated subcontractor. The main responsibility of Ertec, Inc. has been the development of a field pile test program whereas the planning and development of a laboratory model pile test program has been the main responsibility of VERITAS.

Interim Technical Reports (ITR) were submitted to Conoco, dated 16th, July 1981 and 20th, July 1981 from Ertec, Inc. and VERITAS respectively, which presented the results of the planning study completed as of those dates. These reports included reviews of pertinent literature and a review and discussion of available relevant computer programs. VERITAS presented a laboratory model pile test program for investigation of model pile segments subjected to realistic loading and stress conditions. Ertec presented a feasibility study of candidate field test sites and recommendations for a field test program.

A meeting was held in Houston on 13th August 1981 attended by representatives of Conoco, Ertec and VERITAS. The material presented in the ITR and data compiled by Ertec subsequent to submission of ITR was discussed with emphasis on the choice of test site. It was unanimously agreed by all parties that the planning study should be oriented to developing plans for conducting the field test program at West Delta Block 58 site.

The work completed during the three month planning study is presented in two separate reports both dated August 28th, 1981 by Ertec, Inc. and VERITAS. Included within these reports are portions of the respective ITR's.

## Field Test Program

A comprehensive summary of Ertec's part of the planning study is found in their final report dated 28th, August 1981. It is briefly repeated here.

A two phase field testing program was recommended, consisting of a small diameter segment test and a large diameter deep pile test at West Delta Block 58. The results of the small diameter segment test will be used to develop and calibrate a predictive model to extrapolate results of the experiments to larger prototype piles. The second phase of the field testing program consists of construction, installation and testing of an instrumented pile at same offshore site as the segment tests. A pile 67 m long pile with a 76.2 cm diameter is suggested.

The whole program is planned to be executed over a little more than one year, starting October 1981. A follow-up test is suggested after one year rest period.

At the conclusion of the described research program, preparation of general guideline specifications for the design of tension leg platform foundations in soft clay, based on results of the research program, will begin.

The total cost of the field test program is estimated to be US$ 2,735,351. The estimated cost for fabrication and offshore construction included in this sum is US$ 966,020. In the remaining sum, US$ 1,769,331 is included a 25% risk and ucertainties factor.

Laboratory model pile test program.

The work comprising VERITAS' part of the planning study is contained in this final report.

A survey of existing literature pertaining to pile capacity prediction methods is made. From this survey it was concluded that present knowledge and technology does not adequately address the complex problems inherent in design of tension leg platform pile foundations, where piles are loaded cyclically in tension with a considerable static bias.

Existing computer programs available for this project were reviewed. The need for a more versatile constitutive soil model was noted and it is the hope and intent that results of the proposed research program will lead to information and knowledge which will aid in the development of such a model.

A laboratory model pile test program was recommended to be incorporated with the field tests program. Such a laboratory investigation will utilize specially designed equipment, consisting of a model pile segment 25 mm in diameter embedded in actual soil in a confined testing chamber 200 mm in diameter and 350 mm high. The model pile being equiped with instrumentation capable of measuring shear transfer and total and pore water pressure at soil-pile interphase.

A laboratory test of this kind is capable of closely reproducing the stress conditions around an actual pile and also provides freedom in varying the loading conditions. The flexibility of such tests compared to field tests is superior with respect to loading conditions, stress variation, time for test sequence and costs involved. In addition, valuable information can be obtained to aid in the planning and execution of the large scale field test.

The laboratory test program proposed here is scheduled to comply with the field test program schedule proposed by Ertec. A successful execution of the program is therefore pending on the accurate timing and execution of the site investigation and sampling program to be performed at proposed test site.

The total cost of the laboratory model test program including external consultance work from two Norwegian institutions is estimated to be NOK 3.715.057.- (US$ 599,203). In addition costs involved in participation in follow-up tests is estimated to NOK 175.192.- (US$ 28.257). The costs involved in Task 9 "Design Specifications" have not yet been specified. We feel that further discussions with Conoco and Ertec will be required regarding this point.

The grand total for the combined proposed project will then be NOK 20.849.425.- (US$ 3,36 Million) including the laboratory and field test programs. This estimate is based on a conversion rate of 6.20 for US$ to NOK.

TENSION PILE PLANNING STUDY
SUBPROJECT CNRD 13-1

FINAL REPORT

TASK 1

REVIEW OF PERTINENT LITERATURE

# 1. INTRODUCTION

The concept of Tension Leg Platforms (TLP) involves design and installation of piles to resist the large tensile forces produced by the buoyant platform in the offshore environment. Analytical methods to predict the axial capacity and performance of piles have been developed over the years and applied to design of piles for both offshore and onshore conditions. These methods have in general been highly empirical in nature, and the success of their utilization is to a high degree dependent on experience, engineering judgement and comprehensive knowledge of the behaviour of the specific soil deposit of proposed site.

In order to ensure sufficient capacity for a safe design of the piles, high safety factors must be applied to these methods. Due to their incapability to accurately predict capacity and behavior of offshore piles subjected to long term large cyclic forces, there is reason to believe that the design of these piles is conservative. The mechanism of pile-soil interaction is very complicated and in addition great uncertainties are present to the effect of installation and cyclic variable loading on the capacity and load deformation characteristics of the pile.

This study is focusing on the anchoring of a TLP by piles, which involves capacity prediction of long slender piles in the marine environment. Axial capacity of piles is generally computed by adding the axial resistance due to friction along the pile shaft to the resistanse due to bearing at the point. For piles loaded in tension, however, as is the case for piles supporting a TLP, the tip resistance has no significance although different soils can exhibit some degree of suction. Tensile capacity is therefore based on the determination of the skin friction along the pile shaft.

The most common methods presently used for determination of this skin friction and the different factors determining its magnitude will be presented and discussed in the following.

Attention will be focused on frictional capacity of offshore piles in clay (cohesive soil deposits), with special focus on tensile loading and resulting pile-soil behaviour. The reason for this is that clay soils exhibit great uncertainties to the designer as to the determination of stress conditions and its behaviour during installation of the pile and subsequent loading and clay soils are found in the areas for which this study is to be made applicable.

Esrig and Kirby (1979a) notes, in their review of state of practice for axial pile design, that "significant progress has been made in modelling these portions of the problem amenable to conventional methods of structural analysis but that remarkably little progress has been made towards realistically modelling the soil behaviour". Or in other words, there is little soil mechanics in the engineering of a pile foundation. This might be true, but so far, it turns out an analytical formulation of the soil-pile behaviour during installation and subsequent loading is too complex and contains too many uncertainties for a realistically general accurate capacity prediction to be made.

A short review of the state of practice for axial pile capacity prediction is described. Emphasis will be placed on the different factors affecting the axial tensile capacity of an offshore pile and how these factors have been investigated by load tests and laboratory models.

## 2. THEORETICAL DEVELOPMENTS

The following paragraphs contain a summary of the conventional methods for prediction of the unit skin friction $f_s$ along the pile shaft.

A freebody of an axially loaded tension pile is shown in fig. 1. The ultimate axial load, Q, is kept in equilibrium by the soil resistance along the pile shaft $Q_s$. The weight of the pile is here ommitted.

Distinction is usually made between total and effective stress methods, where the total stress method relates capacity to the undrained shear strength of the clay. Effective stress methods are developed on the basis of the assumption (or fact) that frictional capacity is dependent on the effective horizontal stress against the pile shaft.

## 2.1 Total Stress Method ($\alpha$-method)

A large number of piles in clay have been designed on the basis of a correlation between shaft friction and the undrained shear strength, $S_u$, of the soil. The unit skin friction at a depth along the pile is related to the undrained shear strength of the soil at this depth by an empirical factor $\alpha$ (Tomlinson, 1957) according to:

$$f_s = \alpha\ S_u$$

where $f_s$ = unit skinfriction
$S_u$ = undrained shear strength
$\alpha$ = empirical factor

Given a value for the undrained shear strength based on either in situ methods like the vane shear test or correlations of CPT results or from laboratory measurements on recovered soil samples, the shaft friction is dependent on the choice of the factor $\alpha$. Considerable judgement and experience is required to make a realistic choice of the $\alpha$-values to be applied to the different soil layers along the pile shaft, although there exist guidance in the literature. API (1977) suggests values varying between 0.5 and 1.0, while DnV (1977) mentions values between 0.2 and 1.2. One reason for this difference is that extreemely high values of shear strength has been found for some North Sea clays (~ 800 kPa), resulting in the use of $\alpha$-values as low as 0.2. This extremely low $\alpha$-value is a special case and is not relevant for the typical soft clays found in the Gulf of Mexico. The Veritas values refers to API (1977) for choice of $\alpha$-values. There is thus in practise no difference between the API and the Veritas rules in this respect.

A quasi effective stress method for estimation of the shaft friction is the so called lambda method (Vijayvergiya and Focht, 1972). This empirical method assumes the average pile friction is dependent upon both average undrained shear strength and average effective vertical stress along the pile according to:

$$f_a = \lambda(\sigma'_{vave}+2S_{uave})$$

where
$f_a$ = average unit skin friction
$\sigma'_{vave}$ = average vertical effective stress
$S_{uave}$ = average undrained shear strength
$\lambda$ = dimensionsless coefficient

Vijayvergiya and Focht plotted the results of a large number of land-based pile load tests in very varying soils in terms of $\lambda$ against pile length, and found decreasing values with increase in pile length. Values ranging from 0.12 to about 0.49 were suggested by the investigators. The mehod is recommended for piles longer than 15m and has thus been used favourably for offshore pile design.

## 2.2 Effective stress method ($\beta$-Method)

As the $\alpha$-method related shaft-friction to the shear strength of the clay on a total stress basis, rational design methods have been developed which relates shaft friction to the effective vertical stress in the soil. Several investigators have suggested that side friction of a pile is governed by the effective horizontal stress on the pile surface.

The general expression for this is:

$$f_s = K \tan \delta \; \sigma'_{vo} = \beta \; \sigma'_{vo}$$

where $f_s$ = unit skin friction
$\sigma'_{vo}$ = effective vertical stress at depth prior to pile installation
$K$ = coefficient of lateral earth pressure
$\delta$ = angle of friction at interface
$\beta$ = correlation factor

Determination of the factor $\beta$ has been the subject of extensive research and investigation during the last decade. The essence of many of these expressions is a determination of a coefficient for the lateral earth pressure against the pile to the effective overburden stress. The correlation factor has in general been found to be dependent on the effective angle of internal friction, $\phi'$, the overconsolidation ratio, OCR, and the plasticity index of the clay, $I_p$.

During the past several years a generalized effective stress method has been developing. (Esrig and Kirby (1979b)). This method realizes the frictional resistance is dependent on the normal effective stress on the failure surface, which is believed to be at or close to the pile-soil interface. Determination of the effective stresses on this interface is viewed as a problem of addition of the initial state of stress prior to pile driving, the change in stress due to pile driving, the change in stress due to reconsolidation and the stress change due to pile loading. The driving of a pile is modelled as a cavity expansion on the assumption that soil displacements due to pile installation is in the radial direction. An accurate prediction of the normal effective stress $p'_f$ at failure has so far not been succeeded, and as an approximation the following is assumed:

1. For normally consolidated clays, $p'_f$ is assumed equal to p. (initial in-situ stress).

2. For heavily overconsolidatied soils $p'_f$ is assumed equal to the mean normal stress at the critical state, $p_{cs}$.

A summary of the current practices in predicting skin friction for piles in clay can be found in Table 1 together with references.

## 2.3 Comparison of methods

Comparing the different methods to actual load tests gives an indication of the quality (reliability) of the prediction schemes. Such comparison, however, might be misleading, since load tests are usually performed under different conditions and testing procedures from one test to another. Soil conditions do also vary from site to site, which adds to the effect of nonuniformity in the comparisons.

From reported load tests several investigators have made comparisons of the different methods to measured capacity by backcalculation, using the available soil information and loading records. In fig. 2 are shown results from Kraft, Cox and Verner (1981) in underconsolidated clay of piles varying from about 100 to 300 feet in length. A prediction of capacity was made using the $\lambda$-method, (Vijayvergiya and Focht (1972)), $\alpha$-method (API 1980) and the $\beta$-method suggested by Burland (1973). The piles were load tested in compression and the end bearing capacity was computed using a bearing capacity factor $N_c$, varying from 7 to 18, and subtracted from the measured capacity to get a value for the skin friction. Accuracy in measured load was estimated to be within $\mp 5$ kips and the uncertainties for the different prediction methods are related to the undrained shear strength, the effective stress and the angle of internal friction. The differences between the measured and predicted values are generally within the expected uncertainty for each of the methods, according to the investigators. In this case the λ and β methods tends to overpredict the capacity, whereas the variability about a mean "predicted-to-measured" ratio is quite large for the α-method and small for the effective stress method.

From investigations by Flaate and Selnes (1977) the same observation is made, as can be seen in figures 3 and 4. Here, observed side friction from load tests is plotted against undrained shear strength and average vertical effective stress. The side friction calculated from the undrained shear strength has quite a scatter with the resulting data points mainly inbetween the lines expressed by 1.5 $S_u$ to 0.4 $S_u$, whereas the β-method suggested by Flaate and Selnes (1977) shows more consistency. One should here remark that most of the data are from tests on relatively short timber and concrete piles onshore. Another comparison between different methods is also given as a graphical presentation of the quotient of calculated to observed side friction in the form of frequency curves, as shown in figure 5. Here again, it can be seen that effective stress methods may seem to overpredict the side friction, but that the scatter in results of observed to computed side friction is larger for the total stress method than for the effective stress method.

A comparison between predicted to measured capacity when using the generalized effective stress method for calculating the side friction has been made, (Esrig and Kirby (1979b)). They conclude that the methodology provided "reasonable" predictions of capacity for piles in normally and lightly overconsolidated clays. There was, however, a variable difference between the ratio of predicted to measured capacity for pipe piles and timber piles, where the capacity for pipe piles tended to be overpredicted by about 30%. For overconsolidated clays the method always seemed to overpredict the frictional capacity. In order to make the method compare to actual load tests an empirical procedure has to be utilized correlating the overconsolidation ratio to the undrained shear strength of the clay.

## 3. FACTORS AFFECTING FRICTIONAL CAPACITY IN CLAY

As noted earlier, the tensile axial capacity of a pile is related to numerous factors. For design purposes the effects believed to be of most importance are tried accounted for in the design process, but there are factors and situations in the life of a pile which might affect the capacity which are highly unpredictable. As a result, such factors are difficult to incorporate into the models.

Of the most important factors affecting single axial pile capacity, are the following:

- The engineering properties of the soil mass prior to pile installation.
- Length, stiffness and displacements characteristics of the pile.
- The method of pile installation.
- The changes in stress and engineering properties produced by pile installation.
- The changes in stress and engineering properties occuring with time after installation.
- The loading conditions: longterm loading, transient loading, repeated loading, tension and compression loading.

### 3.1 Engineering Properties of soil

It is obvious that the performance of a pile subjected to load is dependent on the engineering properties of the surrounding soil mass. With the term "engineering properties" is meant the results of well known standard tests on actual soil samples. These include placticity index, overconsolidation ratio , particle size distribution, unit weight, consolidation parameters and shear strength characteristics from triaxial and unconfined tests.

Determination of such parameters will define the soil mass for the designer, and they are important in order to select values for parameters in the prediction schemes. Ideally, a complete information of the engineering properties of the soil should enable an accurate capacity prediction for a pile, even with the methods available today. One problem, however, is that such a complete information for an offshore site is usually not available and the soil samples taken from offshore are often associated with great sample disturbance which make the interpretation of their properties for in situ conditions quite unreliable.

The mode of deformation of the soil affects the behaviour of the pile. Initially the response of the pile depends on the deformation properties of a large volume of soil at very low shear strains. As the load increases and failure is approached, the soil in a limited critical zone near the pile becomes highly stressed and a shear plane is developed on a cylinder concentric to the pile wall (Parry and Swain, 1977). At failure the behaviour of the pile is covered by the strength and deformation properties of a small volume of soil close to the pile wall.

## 3.2 Pile Geometry

Apart from the behaviour of the soil subjected to loading, the pile itself will also deform when loaded. A short stubby pile will behave more like a rigid body with negligible relative deformations, whereas a long slender pile will exhibit substantial relative deformations upon loading.

The effect of having a flexible pile to support axial loads will tend to produce relative deformations between pile and soil along the upper part and therefore reach a maximum shearing resistance in these layers. This is concluded from the fact that mobilized shear stress is dependent upon soil shear strain up to a maximum pile displacement. A more flexible pile is likely to mobilize ultimate shaft friction in the upper portions of the pile earlier than a less flexible pile. The effects of strain softening and cyclic degradation may therefore be more susceptible for a flexible pile, with the result of loosing capacity in its upper portion before full strength is mobilized further down the pile.

### 3.3 Pile Installation and subsequent effect

The method of installing the pile has a significant effect on the behaviour of the-pile soil system during loading. Experiments with different installation procedures show a clear distinction between the behaviour of grouted, driven and jacked piles, with the ultimate strength of similar size piles increasing from the first to the last. (Gallagher and St John (1980)).

For grouted piles the soil close to the pile is subjected to a stress relief and the horizontal stress against the pile wall will presumably not be greater than the in situ pressure in the soil before installation. As a pile is driven or jacked the soil is compressed and an increase in horizontal (radial) stress against pile wall results. This pressure will be greater for a closed end pile than for an open ended since a larger volume has to be expanded.

The distinction between driven and jacked piles has recently been acknowledged and is thought to be due to the destructive influence of the cyclic loading (rebound and impact) implicit with driving. As the pile is driven or jacked into the ground a region close to the pile wall will be completely remolded. (Vijayvergiya, 1977). The thickness of this remolded zone depends on the shape and dimensions of the pile tip for a closed ended pile and on the thickness of the pile wall and soil plugging for an open ended pile. This remolding and compression of soil is accompanied by generation of excess pore pressure and therefore changes in stress conditions around the pile. Maximum soil-pile friction that can be mobilized upon loading is therefore determined on the shear strength of the remolded, reconsolidated soil mass.

Residual stresses are induced in the pile-soil system due to both method of installation and subsequent loading (Cooke, 1979). Ignoring this residual stress might lead to an overestimation of shaft capacity (Holloway, Clough and Vesic (1978)).

As time passes after pile installation, the excess pore water pressure produced by the installation will dissipate and the remolded zone will consolidate in radial direction. This will result in an increase in effective stress and thus increase in frictional capacity. In figure 6 is shown a plot of bearing capacity as a function of time after driving (Flaate and Selnes (1977)). For this case, full capacity is seen to be developed first after about three months after driving.

## 3.4 Type of Loading

Axial loading imposes a shear stress on the soil at the pile surface. The response of a pile depends on the nature of the loading and on the nature of the soil to respond to this resulting shear stress.

Piles are generally tested to failure on land by a constant rate of displacement method. Design methods which relate pile capacity to soil properties are compared to results of load tests in which the rate of displacement is standardized.

Piles used as anchorages offshore will be subjected to a sustained load and a dynamic component depending on the environment. During rough weather and storms, the dynamic component will increase substantially so that maximum service load will be close to double the sustained load. (Gallagher and St John (1980)). In the case of a tension pile supporting a TLP the load excerted on the pile will consist of a sustained load from the buoyancy of the platform being pulled down in the water, and the dynamic component resulting from the action of waves, wind and current. The basis for the TLP-concept is to have uplift force on the piles at all times in order to avoid the effect of snapping of the anchoring cables after a slack.

Cyclic loading may affect the capacity of a pile. The effect of cyclic loading on capacity of piles has been focus for intensive investigation during the last decade. Cyclic loading of clay (Sangrey (1977) and Andersen (1976)) produces excess pore pressures, and shows decrease in shear strength with increased number of cycles. The potential for a similar behaviour by an offshore pile is obviously of concern.

Sustained static or cyclic loading may give rise to large irrecoverable displacements over the long term. Excess pore pressures may develop in the immediate soil layer and as a result reduce the effective normal stress and hence the frictional capacity. The rate of generation of pore pressure is linked to the overconsolidation ratio of the clay but there exist uncertainty among investigators as to how the eventual generation of pore pressure affect the pile capacity. Kraft et al (1981) found in their investigation that one way cyclic loading at a certain load rate actually increased the capacity. This is also indicated by Sangrey (1977) for contractive soils, on the basis of laboratory tests showing that some number of cycles of loading with drainage are necessary to bring the soil to a completely stable condition. Bea (1975), however, found that cyclic load not influenced the capacity.

An investigation by Puech and Jezequel (1980) of cyclic tensile capacity of test piles shows that no stabilization of pile top displacements occured after 1500 cycles. The cyclic load level at maximum was here 0.52. During the cyclic tests a progressive drop in the lateral earth pressure coefficient, K, was also recorded in the upper layer. Upon subsequent cyclic tests no significant changes in pore pressure or total radial stresses were observed.

Holmquist and Matlock (1976) and Bogard and Matlock (1979) performed model studies of axial pile behaviour in remoulded clay (described later). They found that repeated one-way loading did not significantly affect the capacity of the test pile and that the failure resistance corresponded approximately to the peak vane shear strength of the remolded soil. By increasing the cyclic load level so that yielding occured in each cycle the deformations increased whereas no significant reduction in frictional resistance was experienced. However, by introducing two-way cyclic loading (loading alternating tension and compression) on the pile, however, a severe degradation in resistance was encountered. The resistance was reduced to approximately 1/3 of the initial static loading peak resistance. Local consolidation did not affect this behaviour upon subsequent two-way loading. The variations in pore pressure as measured during the cyclic loading tests were very inconsistent. No consistent relationship was found to relate the value of pore pressure, or change in this to frictional resistance. The mechanism of cyclic degradation in two-way cyclic loading was believed to be destruction of physio-chemical bonds between particles of clay accompanying realignment of the clay particles paralell to the direction of shear strain. The same observations and conclusions were found by Grosch and Reese (1980) when using the same model pile tested in the field by being pushed beyond the end of a shallow borehole.

One inportant point in the design and prediction of tension pile behaviour is to what degree experience from tests on piles loaded in compression is valid for loading in tension. Cox and Kraft (1979) find in their investigation that tensile capacity equals the frictional capacity in compression. Janbu (1976), however, suggests a reduction in frictional capacity for negative skin friction and piles loaded in tension.

Parry and Swain (1977) remarks that the vertical effective stress during downward loading may exceed the in situ initial effective stress, while it is unlikely to do so under upward loading. This suggests that the lateral coefficient of earth pressure, K, may decrease during uplift of a pile. This effect is also discussed by Tejchman (1976). However, due to the effect of the overburden, there is reason to believe that a reduction in lateral pressure due to uplift might have validity for short piles, whereas for longer flexible piles, this effect would be small.

From the previous finding that two-way cyclic loading causes drastic reductions in the frictional resistance it is obvious that pile flexibility is a critical factor in the interaction mechanism between pile and soil when tensile loading and especially cyclic tensile loading is considered. Even moderate static tensile forces may produce relative movements between pile shaft and soil which are sufficiently large to cause plastic slip on the interface. When a cyclic load is superimposed the amount of irrecoverable pile movement will increase in the previously strained parts of the pile. Due to the detrimental effects of cyclic loading on the shaft frictional resistance, a minimum shaft resistance is approached along the upper part of the pile which requires the mobilization of deeper and deeper soil layers to balance the external forces. This gradual deepening of the degradation effects on the shaft resistance may be looked upon as a progressive failure behaviour which accelerates as the detrimental effects approaches the pile tip.

# 4. EXPERIMENTAL MODELLING OF PILE BEHAVIOUR

Load tests on instrumented piles have generally been utilized as a verification of design methods and to study the effect of loading conditions and different soil parameters. Such tests, however, are very expensive and the ability to perform large parametric studies and long term loading effects is limited. Controlled laboratory modelling of pile behaviour has shown to be a powerful tool to study different aspects of the pile soil interaction behaviour. The following is a description of some of these tests reported in the literature. Care must be taken to the interpretation of results from such laboratory tests to actual piles in the field. The scaling factor can easily lead to misinterpretation. A review of large scale pile load tests applicable to tension piles will be reported elsewhere in this report.

An investigation to study the different parameters affecting axial capacity of piles in clay was made by Bea (1975). A triaxial cell containing a soil sample 2 1/8-inches in diameter by 5 inches long was used to simulate soil and confining pressure. A mild steel rod 1/2-inch in diameter was used as the model pile.
The soil sample was predrilled before insertion of the model pile. Only measurements of interface strength and displacement of pile were made, no attempts were made to measure pore water pressure or total pressure on the pile-soil interface. The study revealed that interface strength increased with effective normal stress in the soil mass. Also correlations were made with regard to the clay's plasticity index. The less plastic clays were found to exhibit larger frictional resistance. Bea also notes that when imposing one-way cyclic loading on the pile the apparent interface strength remained unchanged. A difference was found in the interface strength from this triaxial rod shear test and the direct shear test between steel and soil. This was concluded to be due to the differences in stress states induced by the two testing procedures and apparatus. This information led the authors to recommend application of the rod shear test results in making pile interface strength predictions from laboratory investigations.

Poulos (1980) describes an analyses procedure development for cyclic axial response of a single pile where results are compared to result of model pile tests. Solid aluminum piles 20 mm diameter and 250 mm long were jacked through remoulded clay placed in a cylindrical container 152 mm in diameter and 230 mm deep. The tip of the pile protruded from the bottom of the soil. Ultimate capacity was determined and cyclic load was imposed on the pile upon one hour of consolidation.

Poulos reports experimental results are reasonably consistent and indicate little reduction in ultimate load capacity unless half the amplitude of cyclic load exceeds 60-70% of ultimate static capacity. Total stress analysis (developed by the investigator) showed better agreement with the test results than the effective stress solutions presented.

An investigation by Holmquist and Matlock (1976) to study the resistance - displacement relationships for axially loaded piles in soft clay utilized laboratory model pile test equipment. The model consisted of an instrumented aluminium tube one inch in diameter and 40 inches long with a closed end. Instrumentation consisted of two strain gauge locations to measure difference in axial load. This model pile was inserted into a testing drum 30 inches in diameter and 30 inches deep. Confining pressure was applied to the soil by controlled water pressure in heavy rubber tubes around the perimeter of the drum. The entire drum was filled with a soft remolded marine clay. Controlled loading and displacement measurements of the pile were performed. The shear strength of the clay was obtained using a motorized miniature cyclic vane shear device.

To determine effects of installation procedure on static resistance two tests were run. One test by driving the model pile into the clay under confining pressure, and the second by carefully boring a one inch diameter hole in the clay, installing the pile and then supply the confining pressure. Upon equal consolidation time for the two tests it was found that the resistance displacement curves were quite similar, however the resistance values appeared almost double for the driven pile. The effect of cyclic loading was described earlier in this report and will not be repeated. It was concluded from the test that the laboratory modelling was capable of simulating field conditions.

The same laboratory set up was improved (Bogard and Matlock (1979)) to include vertical confining pressure and measurements of pore water pressure at the pile-soil interface. The intent was to study the pore pressure development and dissipation during loading and subsequent consolidation and also an investigation of the eventual shear-induced pore pressure generation resulting from cyclic loading. It was hoped that such an investigation could aid in the understanding of the mechanism of cyclic degradation in light of observed pore pressure changes and current effective stress concepts of axial pile-soil interaction. An attempt was also made to measure the soil pressure on the pile interface by measuring the circumferential and longitudinal wall strains in the pile. This, however, did not produce reliable results and could not be used for interpretation. The pore pressure measurements proved to be satisfactory during the experiments, but no relationship was found to relate pore pressure changes to frictional resistance, nor could it help to rationally explain the mechanism of cyclic degradation. A mechanistic interpretation of cyclic degradation was suggested, rather than making an attempt of developing a rigorous mathematical explanation.

It can be concluded from this that laboratory model testing represents a powerful tool to the investigation of the pile-soil interaction mechanism and that such tests represent an economical quidance and interpretation tool for full scale field tests. As a result field instrumentation could be simplified and tests more rationally performed when laboratory model tests preceede full scale tests. A laboratory model test, however, should not be a substitute for a large scale field test.

## 5. ANALYSIS OF PILE BEHAVIOR

Several authors have attempted to produce models which combine some of the phenomena observed in element behaviour to principles of soil mechanics. Such models can only be calibrated by comparison of their predictions with experimental data.

Two main different types of analysis is usually employed. One is where the pile soil system is simulated by a discrete element model where the pile is divided into concentrated masses and springs and the soil modelled as springs at concentrated points. (See fig.7). The pile flexibility is simulated by the linear pile springs, and the soil springs are assigned non-linear load deformation characteristics to reflect actual soil behaviour. Another frequently used model for static load-displacement analyses is the finite element formulation simulating pile-soil behaviour. Here an axisymmetric representation of the soil is utilized and the soil continuum is divided into elements with stress-strain properties similar to observed soil behaviour. At the pile-soil interface where relative movements can occur, special "non-dimensional" slip elements are utilized with success.

Below is a short description of two published analysis methods which include effects of cyclic degradation using the one-dimensional discrete element model. Following this is a short description of a discrete element program for simulation of static pile load tests and a finite element program developed for the analysis of uplift behaviour of drilled shaft foundations. The two latter analysis programs are contained in the VERITAS program library and ready for use.

Poulos (1980) described a model where the pile and soil are considered as an elastic continuum with the allowance for interface slippage included. The pile is discretized and an element is considered failed when the maximum shear stress is exceeded. The soil is characterized by an elastic secant modulus which may vary with depth and may be adjusted according to the type and magitude of loading.

For static loading a constant base value of elastic modulus is chosen and is used to calculate the static load deflection characteristics up to the onset of failure. An upper limit of element shear stress is defined and is used for allowence of pile-soil slip. The solution is recycled until, for all elements the total pile-soil stress is less than or equal to the limiting values. The maximum elemental shear stress may be caluclated by taking an effective or total stress approach.

To account for the effect of cyclic loading a static analysis is performed for maximum and minimum cyclic load. The cyclic stress and displacement is determined by subtracting the minimum value from the maximum value obtained from this analysis. Degradation factors are applied to the skin friction and soil modulus depending on the appropriate level of cyclic shear stress and strain at each element. The new values of skin friction and soil modulus is compared to the ones previously estimated and the pile-soil system is reanalysed if the difference is greater than a specified tolerance. Poulos suggests both a total and effective stress approach for the estimation of degradation factors. This model is dependent on calibration to data from quite extensive soil and pile-soil investigations to determine soil properties and their degradation characteristics.

Matlock and Foo (1979) describes an analysis method for both driving of foundation piles by impact or vibration plus static or dynamic axial loading of piles. A discrete element mechanical analogue represents the pile member and a hysteretic, degrading support model is used to describe the nonlinear inelastic behaviour of the soil. Strength degradation is provided as a function of deflection and the number of reversals of deflections of a pile element in the range beyond an initially elastic condition.

An analysis program for axially loaded piles using the one dimensional discrete element model to represent the pile-soil system was presented by Holloway (1975). (Also described by Haugsøen (1979) for the VERITAS-version). This program was developed to analyse a series of hammer blows, statically equilibrating the forces at the end of each blow. After a pile driving phase the appropriate initial condition may be applied to analyze pile load test performance. Non-linear or bilinear load deformation response at the pile soil interface and at the pile tip is modelled using an incremental (piece-wice linear) formulation. The program allows for combined treatment of compression and tension load behaviour such that sequential simulation of the entire load test can be performed.

Withiam (1978) presented a finite element model for theoretical investigation on the uplift behaviour of drilled shaft foundations. The program uses axisymmetric two-dimensional finite elements with one-dimensional slip elements along the shaft-soil interface. The soil is modelled as nonlinear and stress dependent according to the model proposed by Duncan and Chang (1970).
Strength parameters for this hyperbolic model are obtained from triaxial tests and applicable to the soil type in question. Also the initial stress condition on the shaft interface must be supplied which involves estimates of the lateral pressure coefficient. The finite element analysis were limited to monotonic load applications (hereby eliminating any possibility of modelling cyclic compression-uplift loading conditions). Behaviour of drilled shafts during load tests compared reasonably well with the predictions from the program.

## 6. SUMMARY

Each of the analysis methods described has its own advantage to the probelm of describing the behaviour of a tension pile. The discrete element models exhibits a more simple solution to the rigorous problem by viewing it as a one-dimensional problem, with the two former methods superior to the latter because of the inclusion of cyclic degradation factors. A finite element concept of the problem has the advantage of including the soil and pile as an elastic continuum and therefore exhibits the intent of looking at the problem on a total view. This involves that pile butt displacements will include deformations of the soil mass and not only along the soil pile interface, but limitations exist in the modelling to make the problem solving complete for tension pile behaviour.

Common for all the methods described is that they all are dependent on the parameters defining the soil characterictics when subjected to the complex mode of loading resulting from the application of variable axial load on a pile in soil medium.

Given a set of soil parameters to perform an analysis, the methods successfully performs distribution of forces along the pile and computes resulting displacements, but the results can change drastically upon small variation of these parameters. Due to the relatively low stress level with respect to the pile, the pile modulus is justified to be assumed linearly elastic throughout the analysis. The soil, however, performs highly non-linear when loaded beyond a certain point. The level for when the respons becomes non-linear is dependent upon many factors and there exist diverging opinions as to what tests to perform to determine an appropriate load-deformation characteristic applicable to pile loading. These mentioned factors may consist of the following:

- stress level
- the way loading is applied, i.e. simple shear, rod shear, triaxial shear, etc.

- overconsolidation ratio
- plasticity index
- consolidation characteristics, i.e. generation and dissipation of pore pressure.

From laboratory modelling of pile behaviour it has been found that the frictional capacity will decrease

- with increasing placticity
- when two-way cyclic loading occur (half the amplitude amounts to ~ 60% of static capacity)
- with decreasing lateral pressure.

It has not succeeded, however, to relate the reduction in frictional capacity to porepressure generation due to loading, nor has successful measurements of the total pressure against pile shaft been reported. The decrease in capacity has been explained as destruction of physiochemical bonds due to particle realignment, but no quantification of this behaviour related to measurable soil parameters has been made.

It was concluded that realistic laboratory modelling of the pile-soil system can give valuable information about the effects of installation, consolidation (set-up) and behaviour of frictional resistance during different loading conditions. Effective stress methods are believed to be superior to total stress methods when analyzing pile-soil interaction and therefore predictions of the pore water pressure generation along the pile wall must be made. In order to get an insight into the effective stress on the pile-soil interface, measurements of the total pressure against this interface is mandatory in addition to measurements of porepressure.

TENSION PILE PLANNING STUDY

SUBPROJECT CNRD 13-1

FINAL REPORT

TASK 2

SELECTION OF ANALYTICAL METHODS

# 1. INTRODUCTION

Foundation design considerations changes according to the type of foundation considered. Development in the offshore activity requires larger facilities in deeper waters and therefore changes in loading condition results. Accordingly, analytical methods used during design and analysis of piled offshore foundations have experienced continous advancements and refinements in order to predict the pile behavior and capacity. Recent advancments pertaining to the analysis of axial pile behaviour has mainly concentrated on the material models used in the pile-soil interaction and the study of factors affecting side friction capacity of the pile.

The main purpose of the laboratory test program described in Task 3 of this report is to study pile-soil behaviour simulated at different depths along a long flexible pile subjected to various combinations of static and cyclic loading. Data from these test will be the basis for a rheological soil model which can describe the axial strength and stiffness of a tension pile.

## 2. SOIL MODELS

Rheological soil models presently under evaluation are the following:

- the multiple spring - friction block model used in the program DRIVE 7 described by Matlock and Foo (1979)

- hyperbolic load-deformation model described by Duncan and Chang (1979)

- the model presently being evaluated and implemented in the DYNPEL program module of Veritas program system SESAM 80.

The models shall preferrably incorporate:

- non-linear load-deformation characteristics

- stiffness degradation caused by cyclic loading

- strength degradation caused by cyclic loading

- "cyclic creep" effects caused by one-way cyclic loading

The physical understanding of stiffness and strength degradation may be improved by applying soil element models as described by Andersen (1976), Kvalstad and Dahlberg (1980) which were based on simple shear and triaxial cyclic tests respectively.

A close coordination with Ertec and Veritas to incorporate the results from laboratory model tests and segment and large scale field tests is expected to result in an improved pile-soil model which resembles the observed behavior and can be used in predictions for actual tension pile design.

Conoco Norway has recently entered into contract with VERITAS for purchase of the finite element program system SESAM 80. Implemented in this system will be a program module for soil-structure interaction. In this program a dynamic pile-soil element is under development and additional input into this development, based on the results from the proposed tension pile study, will hopefully give further refinement and result in an improvement of the system.

## 3. PILE-SOIL INTERACTION ANALYSIS

Analysis of pile behaviour with finite element programs or discrete element-spring models has been described in Section 5 on TASK 1 of this report. The programs relevant for this study which are presently available at VERITAS are:

- APSI, a one-dimensional discrete element-spring program for analysis of pile driving with subsequent load testing, using hyperbolic or bi-linear interface material models.

- AXIPLN, a two-dimensional finite element program for plain strain and axisymmetric conditions with a non-linear material model based on the Duncan-Chang model.

- SPLICE, a three-dimensional pile and pile-group analysis program based on discrete element-spring models or an elastic Mindlin solution for pile-soil-pile interaction.

None of these programs are presently equipped with a degradation model, and the DRIVE program available at Ertec, Inc. may presently be the best alternative for the overall pile-soil interaction analysis with degradation effects.

Analysis of installation effects on the stress-strain-strength behaviour at the pile-soil interface in the adjacent soil mass could be based on cavity expansion theory. Results from the proposed test program can be used to improve and aid in the present development of such a cavity expansion model performed by Ertec.

Analysis methods for radial consolidation will be required for evaluation of pore pressure dissipation after installation and during cyclic loading. Finite element programs exist for use in this project where pore pressure generation models can be combined with pore pressure dissipation due to consolidation.

As an example of a small parametric study utilizing the program APSI, figures 9 and 10 show the shear stress variation along a pile loaded cyclically (one way). The case modelled in this example was 230 feet long, 30 inches in diameter, wallthickness of 0.5 inches and a soil profile resembling the West Delta block 58 site. A hyperbolic load-deformation model was used for the pile-soil interface as shown in figure 8. For the two cases shown the soil stiffness $K_i$, is the only parameter changed, where this is ten times larger for the case shown in figure 10. The pile is loaded in steps from 200 Kips to 800 Kips and cycled between 500 Kips and 800 Kips.

Since the unload and reload stiffness is assumed equal and no strength degradation upon cycling is incorporated in the analysis, the repeated curves during cycling between 500 Kips and 800 Kips are identical within each case. As can be seen from the figures, two way cycling occurs in the upper third length of the pile. This length is quite unsensitive to the soil stiffness, whereas for the stiff soil case the pile has a substantial reserve capacity towards the bottom compared to the case with the lower soil stiffness.

## 4. ANALYSIS OF PILE MODEL TESTS

In connection with the build-up of the Geotechnical Laboratory at Veritas a program analysis system for interpretation of triaxial test results was developed. The similarity in the measurements to be performed in the model pile tests and in the triaxial test allows us to apply the already developed program package with minor modifications. The program and data analysis system has been described in DnV-report 77-250 and gives examples of the output in forms of tables and graphs.

Analysis of the test equipment and the effect of the limited diameter, the constant boundary stress, the rubber membranes, etc. compared with the field conditions can be performed with an axisymmetric finite element program like AXIPLN described in Section 3.

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# TENSION PILE PLANNING STUDY
# SUBPROJECT CNRD 13-1

## FINAL REPORT

## TASK 3

## LABORATORY TEST PLANNING

# 1. INTRODUCTION

This portion of the planning study is devoted to the development of a laboratory pile test program to study different aspects of frictional resistance between pile and soil. The pile here being the laboratory test pile and the soil will presumably be samples from the site where proposed subsequent field load tests are to be performed.

The purpose of such a laboratory investigation is mainly twofold in that it will give preliminary soil behaviour information of proposed test site, and in addition such tests, properly performed, will hopefully give new improved insight into the mechanics of shear transfer during cyclic loading.

From previous investigations (see Task 1) it is recommended that laboratory model studies, using the same soils under pressures similar to those existing in the field, should be performed in parallel with field tests. In such tests accurate measurements of total and pore pressure are vital. Also the need for fundamental studies of the slip-plane behaviour of clay soil is suggested.

The main goals of the laboratory test program described here are the following:

(i) Determine maximum static shear resistance of the subject clay and the amount and rate of degradation of this shear resistance due to cyclic loading.

(ii) Observe the pore pressure and total pressure variation due to installation, consolidation after installation, cyclic loading, subsequent consolidation and repeated storm simulation.

(iii) Observe a possible relationship between shear induced changes in effective stress and frictional resistance during cyclic loading, and hence attempt to interpret the mechanism of cyclic degradation of frictional resistance to changes in effective stress against pile soil interface.

Some considerations about the loads on a tension pile follows this chapter. A description of the laboratory model pile test equipment developed at VERITAS then follows. Following this the feasibility of the model pile test is discussed. Then some considerations regarding the soil sampling for use in the model tests are made and finally a laboratory test program to obtain the above mentioned goals is presented.

## 2. LOAD CONSIDERATION

The loading experienced by a foundation pile for a TLP is dependent on the environment in which the platform is operating (i.e. site and time dependent). The total load is combined of loads resulting from water current, wind, waves and the pretension in the tethers. The relative magnitude of these components to the total load will vary with time and site location with the pretensioning mainly being static (constant).

In the Gulf of Mexico, the relative load from wind and current is expected to be larger than for a North Sea site, where the wave load constitutes the major load. During a storm loading condition the main oscillating force will result from wave loading, whereas loading from current and wind will be relatively constant compared to the wave force. Thus, for analysis and investigative purposes, the forces resulting from wind, current and pretensioning might mainly be treated as static and the wave force as the cyclic component.

It is therefore important to investigate the effect of varying the static load level with superimposed cyclic load. The laboratory test program described later will incorporate this.

In order to make the suggested program applicable to the proposed Gulf of Mexico site, information about average storm duration and frequency of storm occurence will have to be supplied by Conoco.

Also, during actual loading of a TLP short period oscillation is experienced due to vibration in the platform and tethers. The eventual effect of such vibration force on the shear transfer and degradation might be looked into. Such investigation, however, is dependent on information about magnitude (amplitude) and frequency of this phenomenon. We do at present not expect that short periods oscillations represents a major problem. However, the laboratory test equipment allows for an evaluation of the relative importance of this type of loading with a frequency range of the loading system in the range of about 0 to 10 cps.

# 3. MODEL PILE TEST EQUIPMENT

The model pile tests will be carried out in a confined test chamber developed at VERITAS.
A sketch of the model is shown on figure 11. It is designed with the aim to model a segment of an actual pile with surrounding soil mass as realistic as possible with limitation to size, economic feasibility and existing equipment.

## 3.1 Test Chamber

A soil specimen, remolded or intact, 350 mm high and 200 mm in diameter, will be surrounded by reinforced rubber membraines to separate the soil from the surrounding liquid in the test chamber. The rubber membranes will be reinforced to enable them to mobilize shear stresses at the soil sample surfaces and in that way simulate the in-situ conditions as close as possible. Adequate bonding between soil and membrane is assumed. The radial and vertical pressure chambers will be separated and allow for different vertical and radial stresses to be applied to the soil, thus allowing for anisotropic triaxial consolidation conditions. Back pressure inlets are equipped on top and bottom of the chamber.

The top and bottom circular membranes are both connected to a center piece through which the pile is installed. These center pieces are not restrained in vertical direction so that the membranes, and thus the center part of the soil mass, are allowed to move with the pile in vertical direction, and therefore not produce an artificial veritcal stress on the soil. This feature should allow for a more realistic modelling of actual pile -soil behaviour.

### 3.2 Model Pile

The model pile will be installed in the test chamber by pushing it through the top center piece, down through the soil mass and through the bottom piece. Total length of the pile is 600 mm and the outside diameter is 25 mm. The model pile is closed ended with a $60^0$ cone on the tip, has an inside diameter of 15 mm and is made of acid resistant stainless steel.

On the mid-length the pile is equipped with a 250 mm long instrumented section. Axial load will be measured at both ends of this section. The difference in measured forces during pile loading will correspond to the frictional resistance transferred from the soil to the pile over the length of the instrumented section.

A 100 mm long and ~15 mm wide piece of the pile wall is cut out from this section and equipped with force transducers on the back. The piece is put in place and sealed around the edge with elastic rubber seal. This will enable measurements of total pressure on the pile wall. On this same section is measured pore pressure through a porous stone brought to a transducer outside the test chamber by a sarane tube through the pile. This will enable measurements to allow for interpretation of the results on an effective stress basis.

It should be emphasized that the tests will model the behaviour of a pile section and the soil next to it for the static and cyclic stress strain conditions prevailing at a specified depth below mudline. The ratio between pile radius and radius of the soil specimen of about 1/8 should be adequate to eliminate effects from the boundary of the soil specimen. According to analysis on the variation of shear stress in the soil with distance from the pile, resulting from pile installation and loading, the shear-stress is minimal when a distance of 7 to 10 radii from the pile surface is reached.

### 3.3 Loading and Data Aquisition

The load equipment is the same as will be used for dynamic triaxial soil tests and consists of a load frame with an electro-hydraulic actuator for vertical load or deformation as well as the necessary pressure equipment for applying all pressure vertically and radially. The data aquisition system consists of pulse code modulators (pcm) and a tape recorder which provide a high degree of accuracy and resolution of the recorded analog signals from the different transducers. The performed tests can be either load or displacement controlled.

Recorded signals will be analyzed in our data laboratory and the test results can be printed and plotted automatically. Presentation of results can be in various forms. Plots can be made of shear stress versus effective horizontal stress, load, effective stress, total stress and pore water pressure versus time etc..

Continuous records of the transducer signals will be recorded on a four channel pen recorder and can be used as a back-up system.

# 4. FEASIBILITY OF LABORATORY MODEL PILE TESTS

The description of the test equipment and the soil sampling procedure in the two previous chapters shows that the stress conditions at the pile soil interface and in the immediate surrounding of a section of a tension pile can be simulated closely by the model pile equipment. This is important as the side friction will be heavily dependent on the effective normal stresses which controls the shear strength. Pore pressure generation due to pile installation and cyclic loading will be followed by pore pressure dissipation. This time-dependent process is heavily dependent on the dimensions and the drainage conditions. In the following a comparison of the field and model tests is given for the following situations:

- the initial state of stress prior to pile installation
- the stress changes due to pile installation
- the stress changes due to pile setup (pore pressure dissipation)
- the stress changes under axial loading
- long term repeated storm loading effects

## 4.1 Initial state of stress prior to pile installation

The samples for the model test will be consolidated under $K_0$-conditions at stress levels comparable to the field conditions in a consolidometer before installation in the model pile test chamber. In the test chamber the vertical and radial stresses can be varied independently. Determination of $K_0$ can be done in a special $K_0$-triaxial test where the condition of no radial strain is controlled by comparing the vertical compression and the amount of pore water, or simply by assuming that

$$K_0 = 1 - \sin \phi$$

The vertical effective pressure is equal to the submerged weight of the overburden soil which can be determined with good accuracy from the unit weight measurement of the soil samples to be taken during the site investigation. In the case of underconsolidation, i.e. pore pressures exceed the hydrostatic water pressure, the initial effective vertical stress might be determined by oedometer tests on high qualtiy samples or by direct measurement of the pore water pressure.

Comparison of the undrained shear strength of the model test soil samples with the results of the site investigation will clearly show major differences.

## 4.2 Stress change due to pile installation

According to cavity expansion theory the installation of a pile into the soil will cause an increase in the total radial stress and the pore pressure, which magnitude is dependent on the degree of expansion induced by the penetrating pile. If the expansion exceeds a certain value, limiting values for total stress and pore pressure will develop.

As far as we can judge from available literature on this subject the limiting pressure will be approached by the field test as well as the laboratory model test.

Although the material models and simplifying assumptions involved in the cavity expansion solutions available may lead to unrealistic values of the stresses the model test procedure will lead to a similar treatment of the soil close to the pile as in the field test. We expect thus that no large deviations will exist with regard to the stress conditions for the laboratory model test an the field test.

This assumption applies as well to the planned segment tests described in Ertec's report. A first indication of any major differences in the stress conditions can be derived by comparing the normal stress and pore pressure changes induced by the field segment tests with the results of laboratory tests carried out for corresponding initial stress conditions.

## 4.3 Stress changes due to set up

The main difference between the field test and the laboratory model test will be the time needed for dissipation of exess pore water pressures. The consolidation time will be roughly proportional to the square of the pile diameter and directly dependent on the coefficient of consolidation $c_v$. By using material from the field test site at confining pressures comparable to field conditions we expect that no serious deviations will exist regarding the coefficient of consolidation. The consolidation time required to achieve the same degree of consolidation for a 30" field test, a 3" segment test and a 1" laboratory test will thus be

| Field test | Segment test | Laboratory test |
|---|---|---|
| 1 | 1:100 | 1:900 |

The difference in boundary conditions between the field and the laboratory test regarding the radial extent of surrounding soil will tend to increase the difference further. This rapid consolidation of the model pile compared with the field test allows a large number of tests with a broader parameter variation regarding loading conditions to be carried out within a relatively short time period.

## 4.4 The stress change under axial loading

The load on the pile head is transferred through the pile into the surrounding soil and will tend to change the stress conditions. For tension piles this means a reduction in vertical effective stress and thus a reduction in the normal stress on the pile. This effect cannot be simulated directly by the model pile equipment, but may be taken approximately into account based on FEM-calculations. This effect is not expected to be of major importance.

The local transfer of shear stresses into the soil can be simulated closely. The effect of repeated loading on the strength and stiffness of the soil may thus be evaluated from the model pile test. The volumetric behavour of soil under shear stresses and axisymmetric conditions cannot be modelled realistically in any other available laboratory test and is a major benefit of this test equipment.

The generation of pore pressure due to cyclic loading and the simultaneous dissipation of pore pressure due to drainage and consolidation will have to be considered.

The time factor of 1 to 9 and 1 to 900 compared with the segment and large scale pile tests indicates that the model test may underestimate the pore pressure development due to cyclic loading. Experience with cyclic loading on clay indicates however that the relatively few major waves dominated the process of pore pressure generation completely.

The dissipation of pore pressure after installation will give a clear indication of the time available for cyclic loading where only minor, say less than 20%, dissipation of generated pore pressure takes place and will be a basic input to the load program regarding number of cycles, distribution of amplitudes and frequency.

### 4.5 Long term repeated storm loading effects

Repeated storm loading during the life time of a tension leg platform can be simulated with the model pile equipment.

Normally consolidated soft clay has a potential for volume reduction due to cyclic loading followed by consolidation which may lead to a gradual decrease in normal stress against the pile and thus a decrease in capacity. This effect can hardly be treated analytically and will require an unrealisticly long testing program for the field tests. Negative effects caused by this phenomenon can be detected by a model pile test with a multistorm load program.

## 5. SOIL SAMPLING

The model test chamber descirbed in Chapter 3 was designed to be able to utilize intact undisturbed samples 200 mm in diameter. Sampling of this size is not a routine procedure, however, so an alternative is to use a core of intact undisturbed sample in the center of the test chamber with remolded soil surrounding this. A third alternative is to use only remoleded material consolidated to the actual stress level.

The Norwegain Geotechnical Institute (NGI) has developed a large type sampler taking 200 mm diameter undisturbed smaples. this equipment operates in the same way as a regular push sampler, but has a cutting wire that can be activated near the lower edge and allows pressure to be applied at the same level to reduce the possibility of loss of core material. A 10 inch drill pipe is required. as a casing for this sampler during operation. NGI has successfully used this sampler onshore down to 20 m, but has no experience for deeper penetration nor for offshore application. It is difficult at present to evaluate the feasibility and operational problems connected with the use of this equipment as well as the costs involved in the operation.

Prior to the actual laboratory test program tests will be run in the laboratory to investigate the effect of using intact or remolded soil and a combination of the two. Since great uncertainties are connected to the 200 mm NGI-sampler it seems the most practical and efficient alternative will be to use a 3 inch undisturbed core with remolded soil surrounding this. If the above mentioned comparative investigation indicates inaccuracies or shows potential problems for a successful performance of the test program, efforts to utilize the NGI-sampler will be made.

For the test program suggested in the next chapter a total number of 25 200 mm samples are needed to perform the complete study. When using 3 inch undisturbed standard core samples with remolded soil around this to fill up the test chamber, an estimated number of 30 intact samples of 3 inch diameter and 2 feet (600 mm) length are needed. These samples should be taken from levels located at 10 to 15 meter depth, 30 to 35 meter depth and 55 to 60 meter.

# 6. MODEL PILE TEST PROGRAM

The model pile test program described in the following must be looked upon as preliminary. A definite test program will depend on the type of materials finally being selected and might have to be modified during test series according to results and intermediate findings.

Test series will start out with the installation of the model pile in the test chamber, and monitoring of total and pore water pressure will be made. When pore water pressure against pile compares with the level of back pressure (set-up effect finished) actual loading of the pile will start.

In general the test program will consist of four different patterns of loading, where total pressure, pore water pressure and frictional resistance will be monitored. These four patterns are described in the following.

## 6.1 Static tests

The test program will be started out by a series of static tension tests comprising constant rate of extraction tests in order to evaluate the ultimate static capacity of the clay. This will also give information for development of static t-z curves for input to static analysis programs. It is the intent to use three different levels of confining pressures in order to simulate three levels of depth along the pile. As mentioned in the previous chapter these levels have been suggested to about 15, 35 and 60 meters.

It is suggested that at least one sample for each confining pressure is tested, with one additional for the intermediate depth.

## 6.2 Multistage static Creep Tests

A multistage static creep-load test will determine the creep load which is defined as the load where the creep deformations (time dependent deformations at constant load) show a rapid increase. This load has been found to constitute a critical load level when studying friction piles in soft clay.

This test will be performed by loading the pile successively with fractions of ultimate capacity, and keeping the load constant until deformations cease or until yielding finally occurs. Again at least one sample for each of the three confining pressures will be used with one additional sample for the intermediate level. If results differ in these identical tests, additional samples will be needed to establish a valid representation.

## 6.3 Multistage Cyclic Tests

Various combinations of static and cyclic loads will be carried out according to figure 12.

These tests will be load controlled with a constant static load superimposed by a cyclic component as a fraction of ultimate capacity. This portion of the testing program will define cyclic t-z curves as a function of the permanent and the cyclic load level as well as the effect of the number of cycles.

As the amplitude of the cyclic load increases the deformation of the pile will become greater until a point where excessive deformation will occur. Different levels of static permanent load will be used and thus a set of curves showing deformation versus number of cycles, static load level and confining pressure will result.

After the pile has failed (reached a point of excessive deformation) the pile will be brought back to initial position. A two-way strain controlled

test with increasing amplitude about zero will be performed. In this way the development of frictional resistance degradation along the pile can be studied by plotting mobilized shear force against number of cycles. The two-way cyclic amplitude will be increased when (or if) a stabilization (lower bound) of the resistance is reached.
Along with these cyclic tests the monitoring of pore pressure and total pressure will possibly give information about the effective stress changes during the cyclic loading in light of the expected degradation in frictional resistance.
The two-way cyclic loading tests will give information about the cyclic minimum resistance of soil which is vital for estimation of degradation factors used in analysis programs.

## 6.4 Repeated Storm Loading Simulation

Simulation of long term behavior with repeated storm loading and intermediate consolidation periods will be performed as a last test of a soil specimen. Such tests might give information about long term effects with eventual regain or long term loss capacity upon consolidation and subsequent storm loading simulation.

The test will be performed as strain controlled two-way cyclic loading about zero with either constant increasing and decreasing cyclic amplitudes or with a random generation on the cyclic amplitude. Duration of a typical storm might vary, but a storm of about 5 hours will be planned. After storm loading the static ultimate capacity will be determined and then soil is left to consolidate and allow for excess pore water pressure dissipation with a possible regain of strength. Again the ultimate capacity can be tested and any strength recovery will be observed. This cycle of storm load and reconsolidation will be repeated and ultimate capacity together with pore pressure and total pressure variation will be monitored.

If no change in capacity results and pressure behaviour is repeatable (and thus predictable) testing will be terminated and interpretation of collected data with reference to principles of soil mechanics will be attempted.

## 6.5 Triaxial and Torsional Simple Shear Tests

Soil stress-strain and strength properties required for the analysis in static and cyclic loading will be determined from a series of triaxial and torsional simple shear tests on the same soil as used in the model pile tests. Various combinations of static and cyclic tests will be performed in order to simulate the stress conditions of the soil surrounding a pile.

VERITAS has a fair amount of experience on these kind of tests from previous research projects (eg. Kvalstad and Dahlberg (1980)).

## 6.6 Summary of laboratory test program

It is doubtful whether a soil sample which has failed during one pile test can be used for further testing but this possibility will be studied in the course of the testing.

The preliminary program for cyclic model pile tests is summarized in Table 2. According this plan a total number of 25 200 mm samples are needed to perform a complete study, namely:

- static tests, 4 samples
- multistage tests, 4 samples
- cyclic tests, 12 samples
- storm loading tests, 3 samples

Samples for triaxial and torsional simple shear tests, will be taken from the 3 inch samples mentioned in chapter 5.

## 6.7 Conclusions

The stress state around a pile when loaded is complex and standard laboratory equipment will not have the ability to fully simulate this stress state.

The laboratory model pile segment equipment described previously will more closely reproduce the stress distribution around an actual pile. The equipment can with some limitations simulate all the phases of a pile segment history, from initial condition through installation and set-up and long term loading.

Flexibility is assured with this equipment for both the imposed type of loading and the simulation of stress conditions around the pile.

Because of the small dimensions of the equipment (shorter consolidation time) the amount of time required for one load test sequence is much shorter than in-field conditions, a fact that leads to more flexibility in the number of tests and thus in variation of loading conditions.

By combining the results of the theoretical analysis and the results from the proposed laboratory test program with results from the field segment tests and large scale field tests (described in Ertec's report) a method for extrapolation to a prototytpe size pile is expected to be developed. Having results from three different scale tests the background for such prediction methods should be the best.

DET NORSKE VERITAS
ESTABLISHED 1864

# TENSION PILE PLANNING STUDY
## SUBPROJECT CNRD 13-1

FINAL REPORT

TASK 6

LABORATORY TEST PLANNING

## 1. INTRODUCTION

The schedule and budget of the laboratory model test program has been reevaluated compared with the original Veritas A9-proposal of December 1980 to fit into the proposed field test program and the time limitations given by Conoco and with the aims to give optimum input to the final planning of the field tests.

The test program follows a tight schedule which requires that operational problems have been sorted out prior to the start of the model test program.

The development and construction of the laboratory model pile equipment has been carried out separately by Veritas. Equipment rental has been based on the standard Veritas rental fees. Additional fees for rental of loading and data aquisition system will be billed on the same basis.

A close cooperation and contact with Ertec and Conoco during all phases of the project will be required if the field and the laboratory parts of the project shall give mutual benefit and optimum results.

## 2. SUMMARY OF PROJECT TASKS

The overall project which includes field tests and laboratory model tests has been split in a number of tasks. For practical reasons Veritas and Ertec operates with separate tasks and separate budgets. In this way the work and costs related to Ertec's responsibiiities, which will be carried out in the U.S., has been separated from the work and costs related to Veritas' responsibilities and the additional efforts of other Norwegian institutions within the total project.

Ertec's activities and budgets are specified in Ertecs final report dated August 28, 1981. In the following the activities of Veritas split on 10 different tasks are presented. Task 8 contains a summary of the consultance work to be performed by NGI and NTH.

Task 9 "Design Specifications for Tension Leg Platforms" and Task 10 "Participation in and evaluation of Follow-Up Field Tests" are related to work to be performed after completion of the main investigation by the end of 1982.

### Task 1 Modification, Improvement and Testing of Data Analysis System for Laboratory Model Pile Tests.

Description of Task: Perform necessary modifications to the data analysis program developed for analysis of triaxial tests in order to improve the efficiency in the data interpretation following a model test. Test software system, input and output with data from model pile tests to be performed prior to the test program.

Time required: October to December 15, 1981.

Task 2 Laboratory Model Pile Tests

Description of Task: Planning of detailed laboratory model pile test program with modifications and adjustments as the project progresses. Provide laboratory equipment to perform test program. Carry out initial tests of equipment and procedures for sample preparation, pile installation, load application and test monitoring to optimize efficiency in the following test program. Perform the laboratory model test program and produce results in form of plots and tables ready for interpretation. Carry out standard tests and shear strength determination on samples before and after model test.

Time required: October 1, 1981 to December 1, 1982

Task 3 Theoretical Work and Improvement of Analytical Methods

Description of Task: Development work and improvement of available analysis methods for design of tension piles to allow predictions and comparisons with model pile, segment and field test results. Analysis of available data and adjustments and development of material models for improved design calculations. The work will require close cooperation with Ertec and should be coordinated with the SESAM-80 development work on the integrated dynamic pile analysis program DYNPEL. The material models presently being implemented in DYNPEL are of general nature regarding degradation effects and have been based on the present state of konwledge. The improved knowledge and material models expected to result from this project should be implemented in the DYNPEL module.

Time required: January 1 to December 15, 1982

Task 4 Prediction of Test Results and Comparison with Observed Behaviour

Description of Task: Perform predictions of laboratory model pile segment tests and field pile test prior to the start of each test program based on data available and test specifications. Carry out comparisons between predications and observed behaviour regarding capacity, degradation effects, load distribution and pile displacement.

Time required: January 1 to November 1, 1982

Task 5 Participation in Field Tests

Task Description: Review and comment on the field test program and field test results. Participation in the field tests and current exchange of results. This task will include travel costs, labor and housing not provided by Conoco. Detailed analysis of results covered by Task 3 and 4.

Time required: January 1 to December 1, 1982

Task 6 Reports and Recommendations

Description of Task: Reporting of all parts of the project work performed by VERITAS, as outlined in Task 1 to 4. Working out recommendations for the installation and design of piled foundations for tension leg platforms. Drafting, printing and reproduction costs are included.

Time required: February 1 to December 15, 1982

Task 7 Project Administration

Description of Task: Administration and coordination of the activities. Meetings with other participants, Conoco and eventual other sponsors and related travel expences.

Time required: Entire project

Task 8 External Norwegian Consultance Work

Description of Task: Expences related to consulting work performed by other norwegian institutions like NGI and NTH. The work can be split in the following subtasks:

- 8.1 Review and comments during the project, technical meetings
- 8.2 Independent predictions prior to laboratory and field tests
- 8.3 Participation during field and laboratory testing
- 8.4 Assistance in theory development
- 8.5 Review of final report and assistance in development of recommendations.

Time required: January 1, 1982 to February 1, 1983.

Task 9 Design Specifications for Tension Leg Platforms

Comment: General design specifications for TLP pile foundations should preferrably be integrated in a complete design specification which also includes environmental and soil data, loads and loading conditions, structural analysis, materials,fabrication, construction, transportation, installation, inspection and monitoring and removal of platform and connected installations.To achieve a consistent design philosphy the foundation design specifications will have to be adapted to the assumptions and requirements involved in the overall design specification, specifically with respect to the load and loading condition, the wave-structure-tether-pile-soil interaction and the safety requirements.

Description of Task: Work out specifications for site investigation, site evaluation (stability etc.) requirements, field and laboratory testing, determination of design strength and deformation characteristics for pile-soil interaction, analysis requirements for capacity and structure-pile-soil interaction, installation. Budget not specified at present.

Time required: Work to be started after completion of Task 1 to 8. To be completed within 3 to 6 months.

TASK 10 Participation in and Evaluation of Follow-Up Field Test

Description of Task: Review and comment on test program. Participation in the field tests. Detailed evaluation of test results regarding long-term effect of set-up/consolidation on the axial capacity and deformation behaviour of the pile. Comparison with results of long-term, multiple storm tests performed with the laboratory model test pile. Write report as addendum to Final Report and eventually revise general design specifications worked out under Task 9.

Time required: Presently unspecified.

## 3. REPORTS

### 3.1 Progress Reports

Progress Reports shall be submitted to Conoco at the end of each month and shall contain a statement of man-time and costs expended.Explanation of eventual deviations from the schedule and budget shall be given.

### 3.2 Interim Technical Reports

Two Interim Technical Reports are planned to be worked out during the project. The ITR shall contain a summary of the progress of the different tasks. These reports will be submitted to Conoco and Ertec, one at the end of April 1982 prior to the performance of the first pile test and the segment tests and the other in the beginning of August 1982 prior to the major field test. Emphasize will be put on presentation of findings of the laboratory model pile test results and also predictions of the field test pile behaviour based on t-z curves and degradation effects derived from the laboratory tests and analytical work.

### 3.3 Final Technical Report

The Final Technical Report will be submitted at the end of 1982. The report will present recommendations for TLP pile foundation design specifically for the deep-water sites Green Canyon and Viosca Knoll in the Gulf of Mexico. These recommendations will be based on the results of the field and laboratory testing programs, the theoretical work and practical experience with analysis methods, and the experience gained with respect to our ability to predict the behaviour of large scale piles subjected to realistic loading conditions (i.e. the major field test) from the results of soil investigations in the field and the laboratory, laboratory model pile tests and field segment tests.

Documentation of all phases of the research program will be included in the final report. Ertec will cover the field test part, Veritas will cover the laboratory model pile part and the recommendations will have to be worked out in close cooperation between Ertec and Veritas.

The comments from external norwegian consultants on the final report in general and spesificly on the recommendations will be presented as an addendum to the final report within February 1983.

### 3.4 Design Specifications

The results of Task 9 will be presented in an additional Report. Se comments and description of Task in Ch.2.

### 3.5 Follow-up Test

An additional report containing the findings from the planned follow-up test will be prepared and submitted within a short time after completion of Task 10. The report will contain recommendations for treatment of long-term behaviour based on the follow-up test and the long term, multiple storm tests on model piles performed in the main investigation.

## 4. BUDGET

A summary and a detailed budget of the cost related to work for which VERITAS will be responsible is presented in Appendix 1 of this report.

The hourly rates used in the evaluation of man costs are based on an estimate of the average hourly rate throughout the project. The present hourly rates will be adjusted per January 1,1982 and a major part of the project work will be carried out during 1982.

A budget has been worked out for each of the 10 tasks specified, except for Task 9 where we feel that further discussions with Ertec and Conoco will be required. See also comments to Task 9 in Ch. 2.

The additional sampling costs related to the laboratory model pile tests have been estimated and included in the budget for Task 2 based on the assumption that 3 inch standard sampling is a satisfactory solution. If the use of the 20cm diameter NGI - sampler is found to be a better alternative the costs involved will be estimated and a change proposal will be submitted to Conoco. Further technical discussions with Ertec and NGI to evaluate the practical problems connected to the use of the sampler is planned in September 1981. See also this Final Report on Task 3, Laboratory Test Planning, Chapter 5.

The total cost of the laboratory model test program including external consultance work from two Norwegian institutions is estimated to be NOK 3.715.057 (US$ 599.203 ). In addition costs involved in participation in follow-up tests is estimated to NOK 175.192 ( US$ 28.257 ). The costs involved in Task 9 "Design Specifications" have not yet been specified. We feel that further discussion with Conoco and Ertec will be required regarding this point.

## 5. SCHEDULE

The project schedule have been worked out with emphasis on adaption to the field test program as prepared by Ertec. The following milestones in the field program have had a major influence on the laboratory pile test schedule

| | Date |
|---|---|
| a) Site investigation/sampling complete | Last week Jan,1982 |
| b) Pile installation, first pile test and segment test | First week May,1982 |
| c) Major field test | Middle of Aug.1982 |

The start of the laboratory test program will start immediately after receiving the soil samples and is thus dependent on milestone a)

Interim technical reports summarizing the results of the laboratory study and presenting predictions of field test results will possibly influence the final planning of the field test programs and will have to be submitted in sufficient time prior to milestone b) and c).

As pointed out earlier the laboratory test program has been compressed considerably compared with the original A9-Veritas proposal to fit within the timelimits and milestones of the field test program.

Figure 13 show the time schedule of the main tasks and activities.

6. PROJECT ORGANIZATION

Based on the assumption that the total project will be performed as a part of Conoco Norway Inc.'s Cooperative Research and Development- Norway the organization chart presented in Figure 14 is proposed.

The organization of Ertec's project team is extracted from plate 34 of their final report. (see also Ertec's report page 68.)

Veritas' and Ertec's project team will work closely with Conoco through the duration of the entire project. A direct communication line between Ertec and Conoco, PES in Houston is essential for the operation part of the field work.

A technical committee is proposed with representatives of all involved parties.

Mr. Olav Furnes, Head of the Department for Ship and Offshore Structures in the Research Division of Veritas will be the project responsible. Dr. Rune Dahlberg Head of the Section for Geotechnics in the Industry and Offshore Division, Veritas will advise on all phases of the project and would participate in the proposed technical advisory group.

Project Manager for Veritas' activities and administrator of the project will be Mr. Tore J. Kvalstad. He will coordinate the activities of Veritas and the Geotechnical consultants and take active part in the model test program.

Resumes of key personnel involved in Veritas' part of the project are presented in Appendix II.

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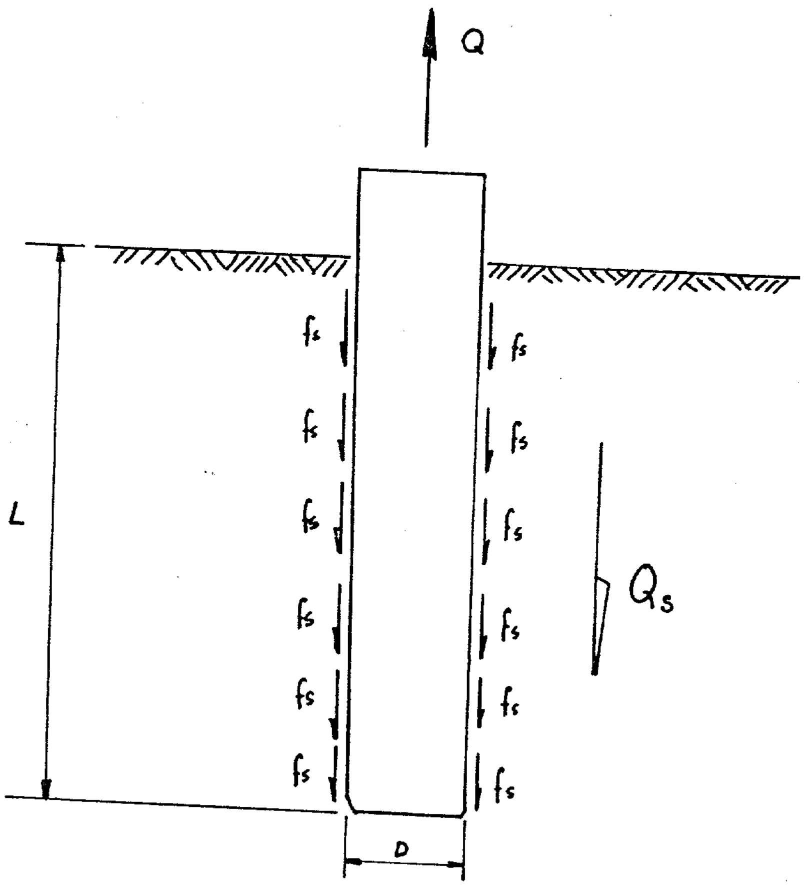
TABLE 1 CURRENT PRACTICE IN PREDICTION OF SKIN FRICTION FOR PILES IN CLAY

| METHOD | | CURRENT PRACTICE | REFERENCE |
|---|---|---|---|
| TOTAL STRESS $\alpha$ | $f_s = \alpha S_u$ | $\alpha = 0.2 - 1.5$ decreasing with increasing $S_u$ | API (1977), TOMLINSON (1970) DnV (1977) |
| EFFECTIVE STRESS $\beta$ | $f_s = \beta\,\bar{\sigma}_{vo}$ | $\beta = (1-\sin\phi')\tan\phi'$ | BURLAND (1973) |
| | | $\beta = 1.5(1-\sin\phi')\tan\phi'\sqrt{OCR}$ | MEYERHOF (1976) |
| | | $\beta = \dfrac{\sin\phi' \cos\phi'}{1+\sin\phi'}$ | PARRY AND SWAIN (1977) |
| | | $\beta = (\dfrac{d+20}{2d+20})\,\mu\sqrt{OCR}$ ; $\mu$ dependent on Ip | FLAATE AND SELNES (1977) |
| | | $\beta = \lvert r \rvert \mu\ (1+\mu^2+\mu\sqrt{1+r})^{-2}$ | JANBU (1976) |
| COMBINED TOTAL AND EFFECTIVE STRESS | $f_s = \lambda(\bar{\sigma}_{vo}+2S_u)$ | $\lambda$ Varies with depth of penetration, decreasing with depth (0.1 ~ 0-4) | VIJAYVERGIYA AND FOCHT (1972) |
| GENERAL EFFECTIVE STRESS | $f_s = \beta\bar{p}_f$ | $\beta = \dfrac{3\sin\phi_{ss}}{3-\sin\phi_{ss}}\cos\phi_{ss}$ <br> $\bar{p}_f$ = dependent on OCR and plasticity index | ERSRIG AND KIRBY (1979) |
| CPT CORRELATION | $f_s = \alpha'\, q_c$ | $\alpha'$ depends on the soil type | DE RUITER AND BERINGER (1979) |

$f_s$ = ultimate skin friction
$s_u$ = undrained shear strength
$d$ = depth below ground surface
$\phi'$ = effective angel of internal friction
$\phi_{ss}$ = angle of shearing resistance at the pile-soil interface
$\bar{\sigma}_{vo}$ = initial vertical effective stress
OCR = overconsolidation ratio
$q_c$ = cone resistance
$\bar{p}_f$ = mean normal effective stress at pile-soil interface at failure

TABLE 2 PRELIMINARY MODEL PILE TEST PROGRAM

| | Consolidation stress, $\sigma_c$, kPa | Number of samples to be loaded | | | Comments |
|---|---|---|---|---|---|
| Static (constant rate of extraction) tests | 100<br>300<br>500 | 1<br>2<br>1 | | | These tests will be used to define the ultimate static capacity $F_{ult}$ and static t-z-curves |
| Multistage static creep-load tests | 100<br>300<br>500 | 1<br>2<br>1 | Load Level $F_{stat}/F_{ult}$<br>0.1, 0.2, 0.3, .......<br>0.1, 0.2, 0.3, .......<br>0.1, 0.2, 0.3, ....... | | These tests will define the creep load, i.e. the load where the creep deformations under constant load show a rapid increase, and thus the time dependancy of the t-z-curves |
| Multistage cyclic tests | <br>100,300,500<br>100,300,500<br>100,300,500<br>100,300,500 | <br>3<br>3<br>3<br>3 | Static Load Level $F_{stat}/F_{ult}$<br>0.2<br>0.33<br>0.50<br>0.75 | Cyclic Load Level $F_{dyn}/F_{ult}$<br>±0.1 ±0.2<br>±0.11 ±0.22 ±0.33<br>±0.12 ±0.25 ±0.37 ±0.50<br>±0.05 ±0.10 ±0.15 ±0.20 | These tests will define the cyclic t-z-curves as a function of the permanent and the cyclic load level as well as the effect of the number of cycles. |
| storm -loading tests with random load history | 300<br>300<br>300 | 1<br>1<br>1 | 0.2<br>0.33<br>0.50 | 0.2<br>0.33<br>0.50 | These tests will show the deformation behaviour under a realistic load-time history. |




Q = ULTIMATE AXIAL TENSILE LOAD

$Q_s$ = REACTION PRODUCED BY SHAFT FRICTION

$f_s$ = AVAILABLE SHEAR RESISTANCE AT PILE-SOIL INTERFACE

LIMIT EQUILIBRIUM: $Q = Q_s = \pi D \int_0^L f_s d\ell$

Figure 1 - Freebody of a tension pile

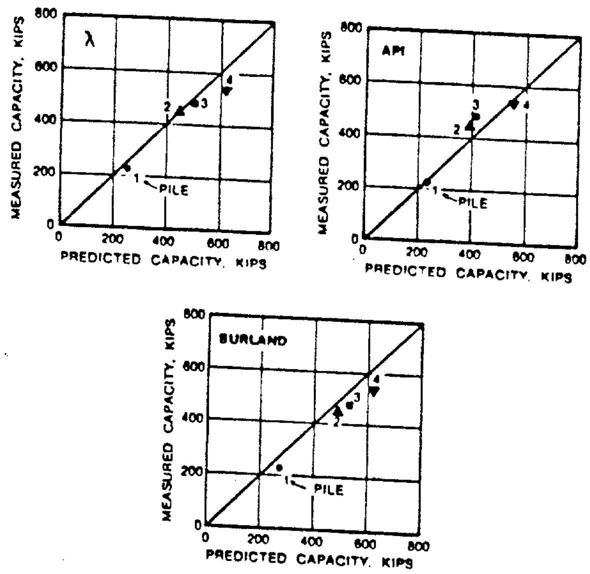



Figure 2 - Comparison of Measured and Predicted Ultimate Capacities for First Compression Tests of First Test Series (1 kip = 4.45 kN) (From Kraft, Cox and Verner (1981))

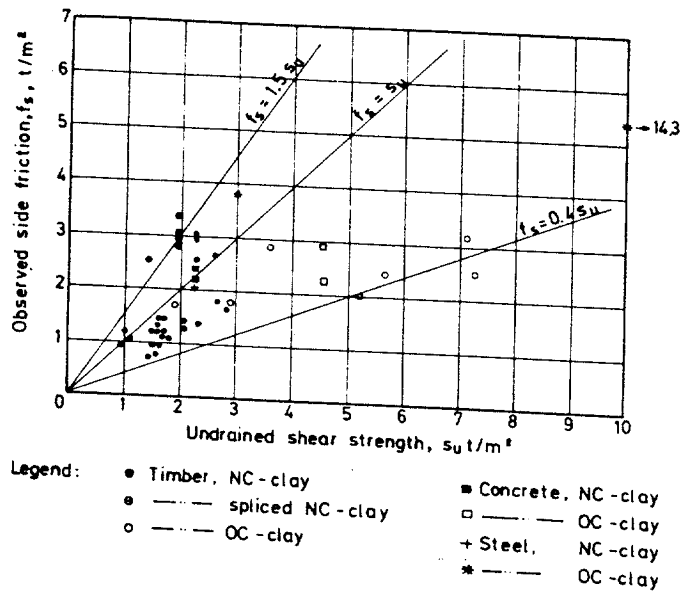



Figure 3 - Observed side friction versus undrained shear strength. (From Flaate and Selnes (1977))

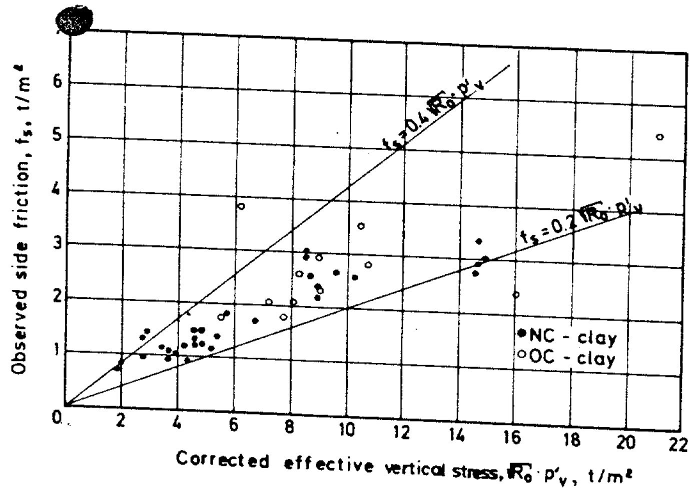



Figure 4 - Observed friction versus adjusted effective vertical stress. (From Flaate and Selnes (1977))

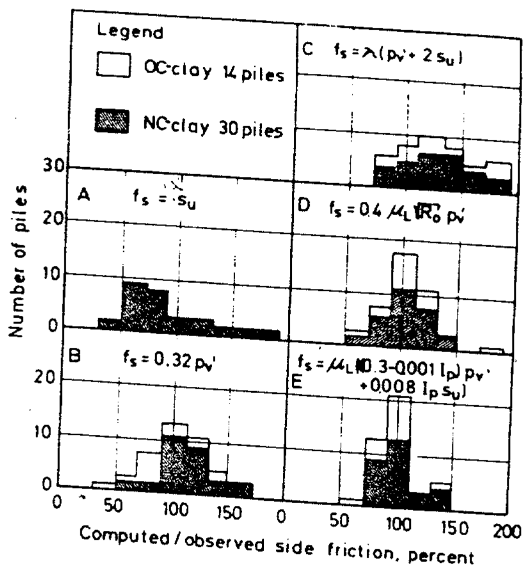



Figure 5 - Frequency curves for the quotioent of calculated to observed side friction for various formulas. (From Flaate and Selnes (1977))

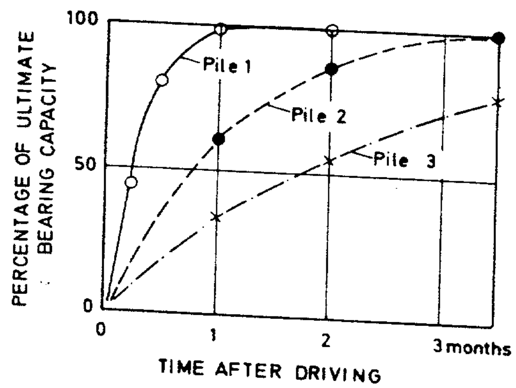



Figure 6 - Variation of pile capacity in clay with time after driving. (From Flaate and Selnes (1977))

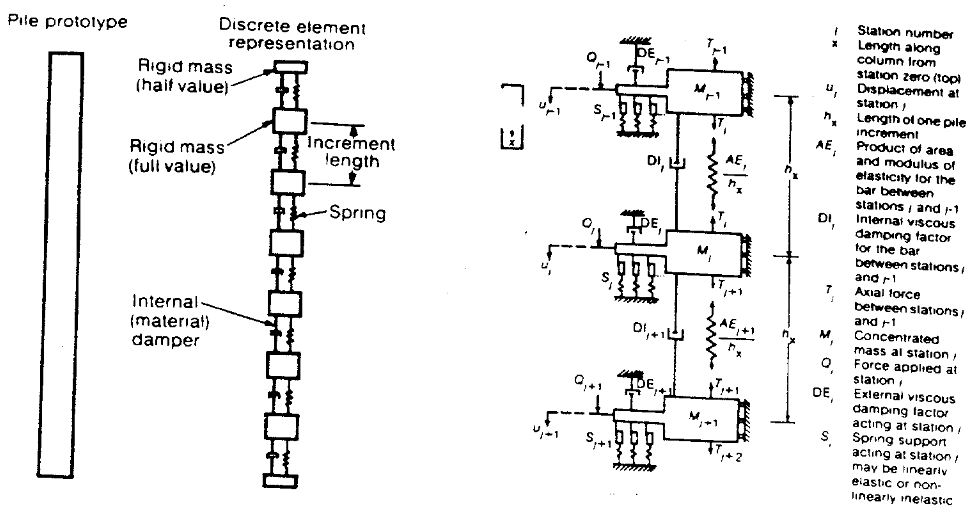
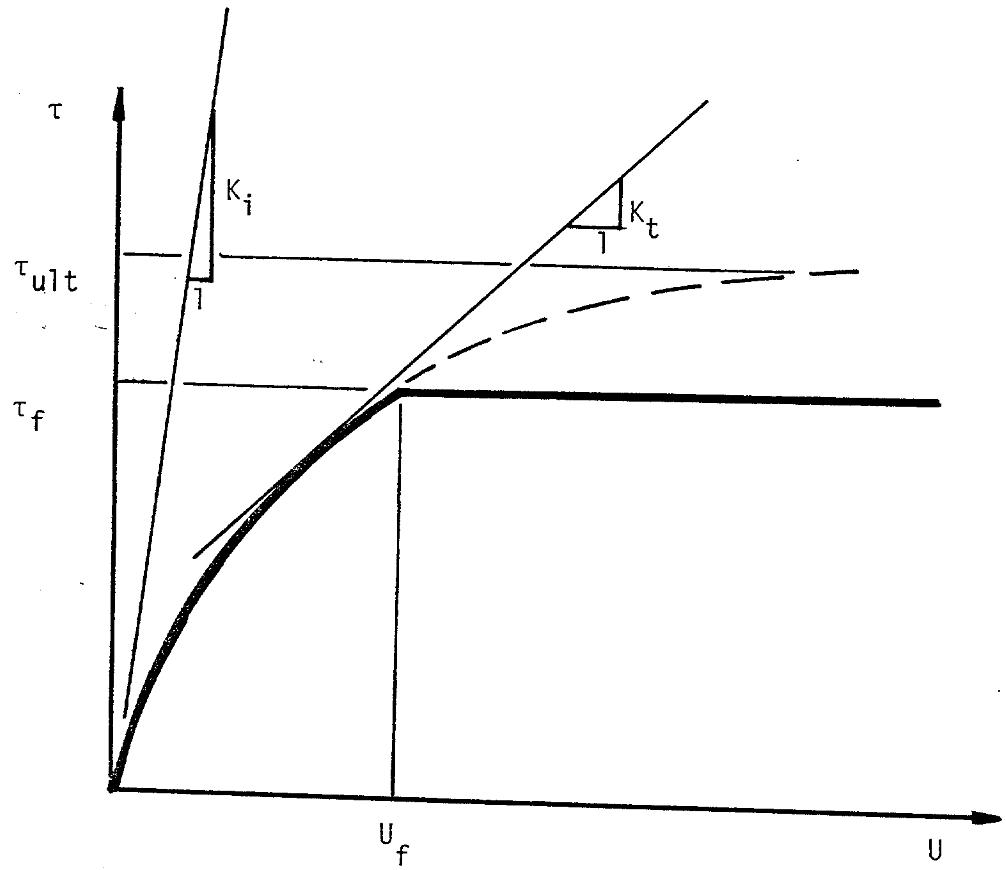



Figure 7 - Discrete element model for pile - soil representation (Matlock and Foo (1979))




$\tau_f = 0.8\ \tau_{ult}$ $\qquad$ $K_t = K_i\ (1 - \tau/\tau_{ult})^2$

$U_f = 4\tau_{ult} / K_i$ $\qquad$ $K_i = \kappa\ \gamma_w\ (\sigma_3' / p_a)^n$

Figure 8 - Hyperbolic load-deformation model (t-z curve)

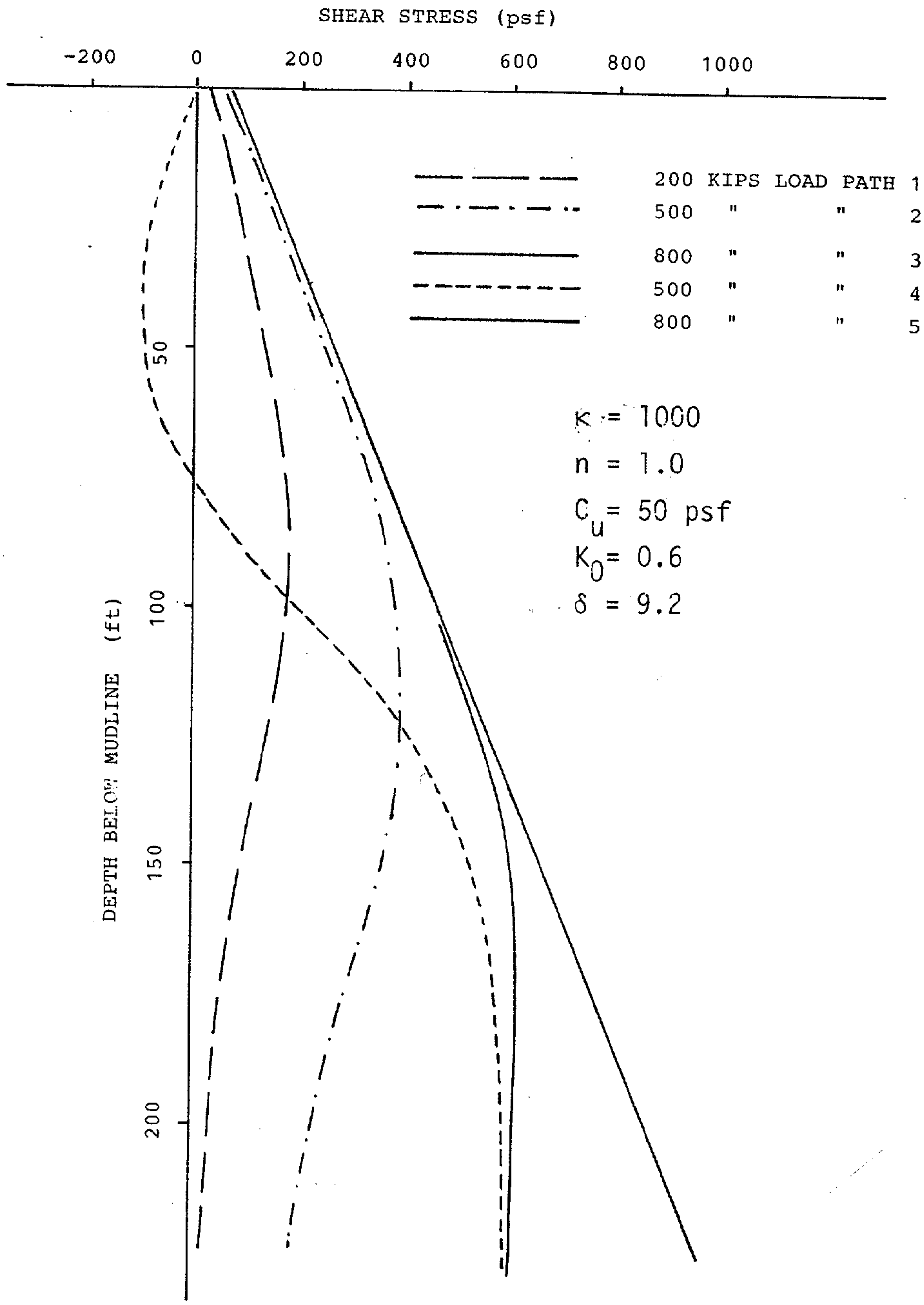



Figure 9 - Shear stress vs. pile depth during cyclic load test (low initial soil stiffness)

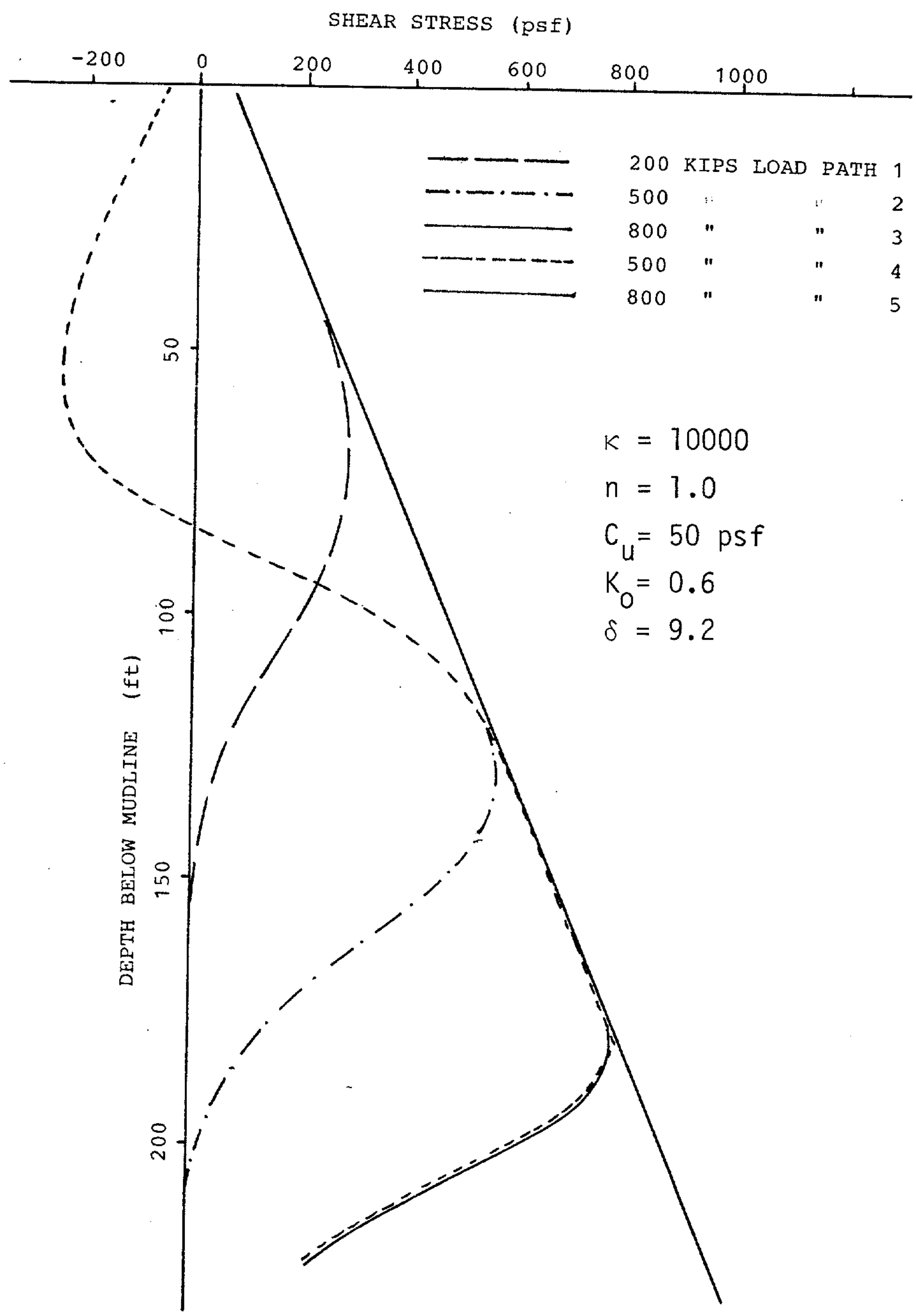



Figure 10 - Shear stress vs. pile depth during cyclic load test (high initial soil stiffness)

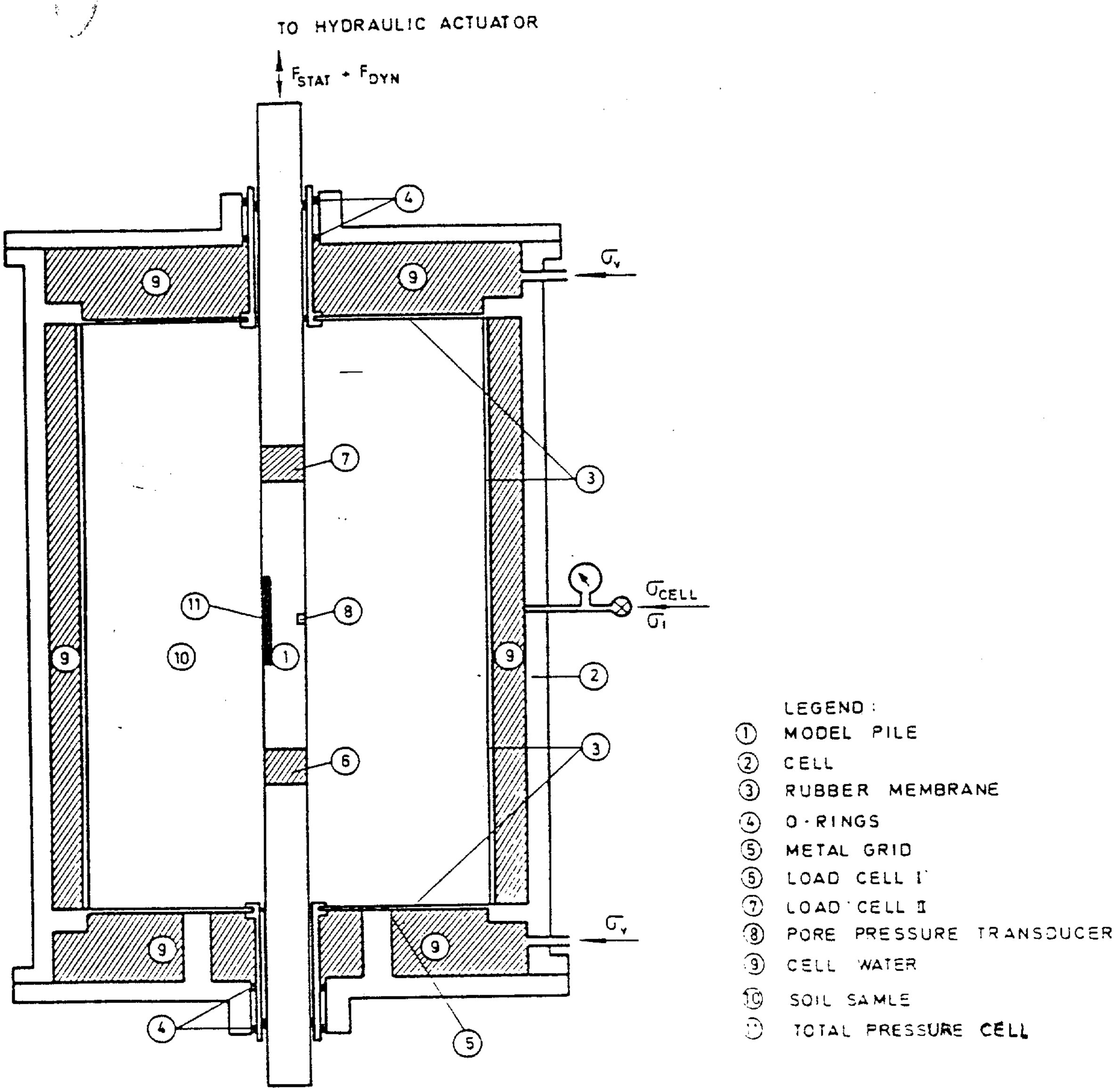



Figure 11 - Model pile test set-up.

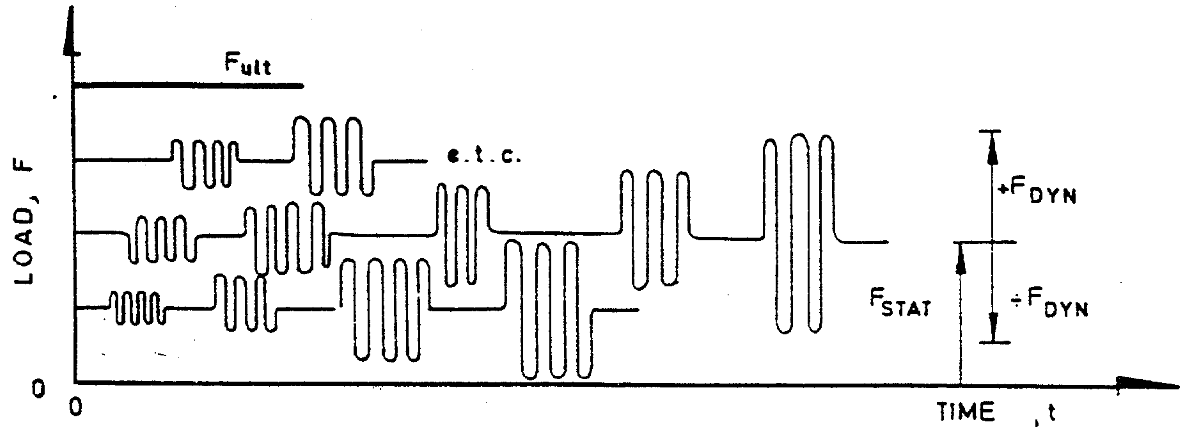



Figure 12 - Example of multi-multistage dynamic load-time history.

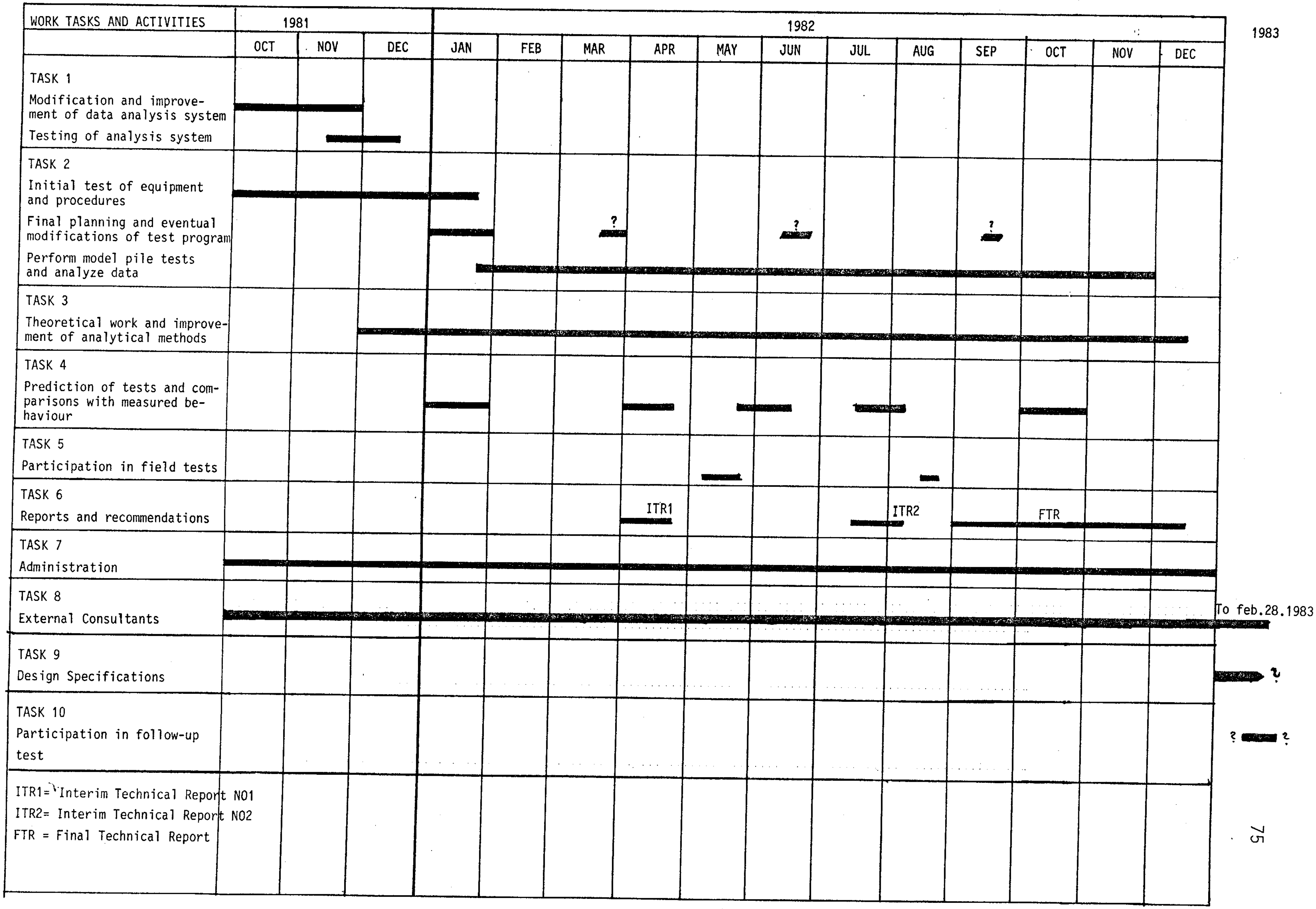



Figure 13 – Schedule of activities

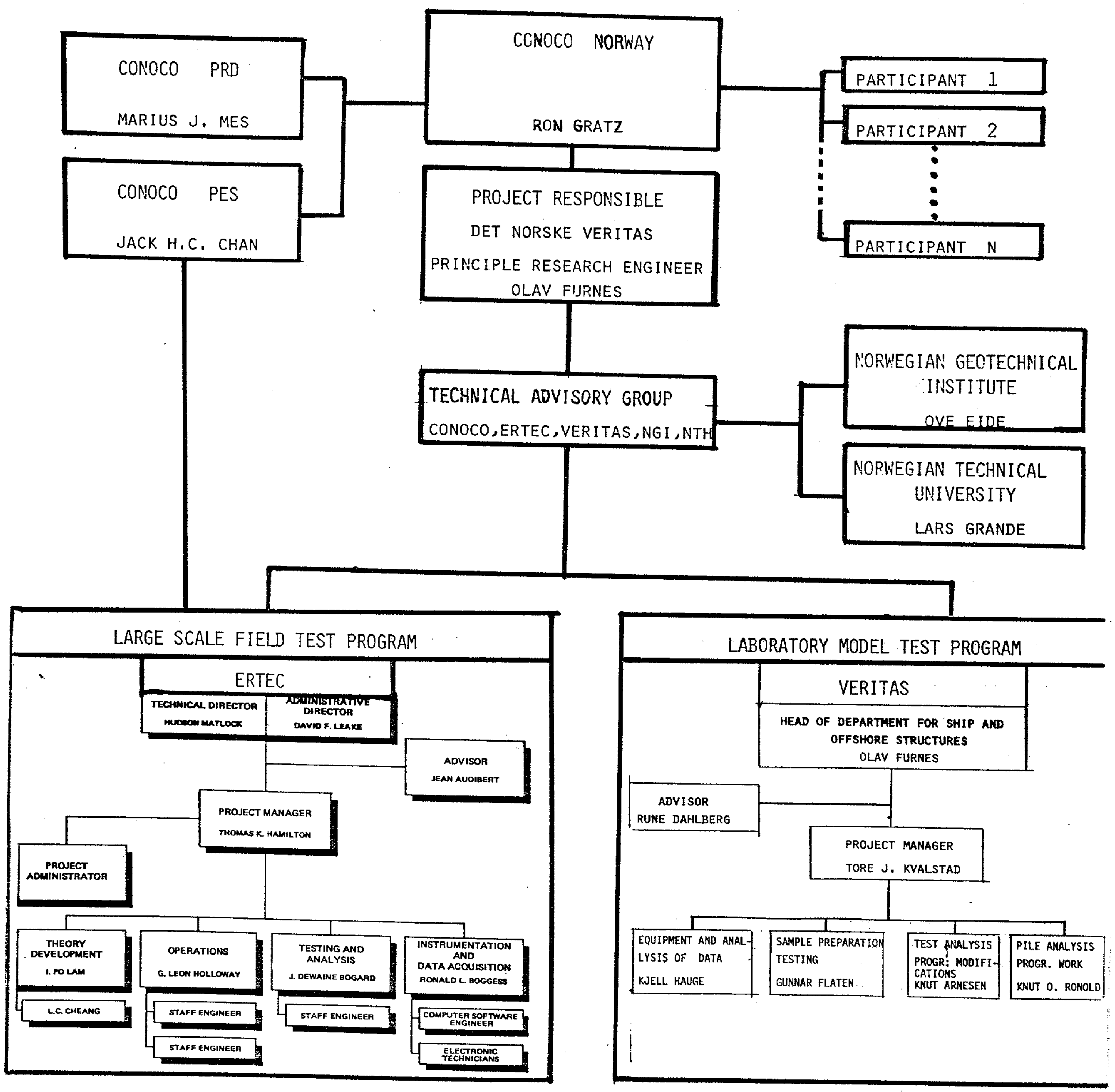



FIGURE 14- ORGANIZATION CHART

TENSION PILE PLANNING STUDY
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FINAL REPORT

APPENDIX I
BUDGET FOR VERITAS ACTIVITIES

SUMMARY OF PROJECT COSTS

Main Investigation (Task 1 through 8)

| | |
|---|---|
| Veritas Estimated Cost | 2.630.150 |
| External Consultance(NGI and NTH) | 908.000 |
| Subtotal | 3.538.150 |
| Contingencies 5% | 176.907 |
| Total Costs of Main Investigation | 3.715.057 |

Tasks 9 and 10

Task 9 is presently unspecified

Task 10

| | |
|---|---|
| Veritas Estimated Costs | 166.850 |
| Contingencies 5% | 8.342 |
| Total cost of Taskes | 175.192 |

TASK 1 - Modification, Improvement and Testing of Data Analysis System for Laboratory Model Pile Tests

| | | Rate | Quantity | Cost |
|---|---|---|---|---|
| A. | Personnel | | | |
| | Senior Research Eng. | NOK 450/hr | 40 hrs | 18.000 |
| | Research Engineer | NOK 400/hr | 240 hrs | 96.000 |
| | Technician | NOK 330/hr | 120 hrs | 39.600 |
| B. | Computer (datalab) | | | 15.000 |
| | | Total | NOK | 168.600 |

TASK 2 - Laboratory Model Pile Tests

| | | Rate | Quantity | Cost |
|---|---|---|---|---|
| A. | Personnel | | | |
| | Senior Research Eng. | NOK 450/hr | 200 hrs | 90.000 |
| | Research Engineer | NOK 400/hr | 800 hrs | 320.000 |
| | Technician | NOK 330/hr | 1600 hrs | 528.000 |
| | | | Subtotal | 938.000 |
| B. | Equipment Rental | | | |
| | Model pile test equipment | NOK 500/day | 150 days | 75.000 |
| | Load/Control Equipment | NOK 500/day | 150 days | 75.000 |
| | Data Acquisition system | NOK 400/day | 150 days | 60.000 |
| | Miscellaneous Lab. Equipment | NOK 200/day | 150 days | 30.000 |
| | | | Subtotal | 240.000 |
| C. | Soil Sampling | | | |
| | Undisturbed Samples;3½" diam. 2' long | NOK 1250/sample | 35 saples | 43.750 |
| | Shipment and handling (Test site - Oslo) | | | 10.000 |
| | | | Subtotal | 53.750 |

| | | |
|---|---|---|
| | | Costs |
| D. | Computer (datalab) | 50.000 |
| E. | Miscellaneous Expendables | 10.000 |
| | Total NOK | 1.291.750 |

TASK 3 - Theoretical Work and Improvement of Analytical Methods

| | | Rate | Quantity | Costs |
|---|---|---|---|---|
| A. | Personnel | | | |
| | Senior Research Eng. | NOK 450/hr | 160 hrs | 72.000 |
| | Research Engineer | NOK 400/hr | 400 hrs | 160.000 |
| | | | Subtotal | 232.000 |
| B. | Computer | | | 40.000 |
| | | | Total NOK | 272.000 |

TASK 4 - Prediction of Test Results and Comparison with Observed Behaviour

| | | Rate | Quantity | Costs |
|---|---|---|---|---|
| A. | Personnel | | | |
| | Senior Research Eng. | NOK 450/hr | 40 hrs | 18.000 |
| | Research Engineer | NOK 400/hr | 400 hrs | 160.000 |
| | | | Subtotal | 178.000 |
| B. | Computer | | | 25.000 |
| | | | Total NOK | 203.000 |

TASK 5 - Participation in Field Tests

| | | Rate | Quantity | Costs |
|---|---|---|---|---|
| A. | Personnel | | | |
| | Senior Research Eng. | NOK 450/hr | 40 hrs | 18.000 |
| | Research Engineer | NOK 400/hr | 120 hrs | 48.000 |
| | | | Subtotal | 66.000 |
| B. | Travel Expenses | | | |
| | Transportation (Oslo-Test Site) | NOK 12.000/trip | 3 trips | 36.000 |
| | Subsistence | NOK 570/day | 15 days | 8.550 |
| | | | Subtotal | 44.550 |
| | | | Total | NOK 110.550 |

TASK 6 - Reports and Recommendations

| | | Rate | Quantity | Costs |
|---|---|---|---|---|
| A. | Personnel | | | |
| | Senior Research Eng. | NOK 450/hr | 200 hrs | 90.000 |
| | Research Engineer | NOK 400/hr | 400 hrs | 160.000 |
| | Technichians, Draftsmon | NOK 330/hr | 200 hrs | 66.000 |
| | | | Subtotal | 316.000 |
| B. | Repreduction/Printing | | | 5.000 |
| C. | Shipping (Air Courier) | | | |
| | Oslo - Houston | | | 3.000 |
| | | | Total | NOK 324.000 |

TASK 7 - Project Administration

| | Rate | Quantity | Costs |
|---|---|---|---|
| A. Personnel | | | |
| Principal Research Eng. | NOK 450/hr | 160 hrs | 72.000 |
| Senior Research Eng. | NOK 450/hr | 160 hrs | 72.000 |
| Research Engineer | NOK 400/hr | 80 hrs | 32.000 |
| | | Subtotal | 176.000 |
| B. Travel Expenses | | | |
| Transportation (Oslo - Houston) | NOK 12000/trip | 5 trips | 60.000 |
| Subsistence | NOK 570/day | 25 days | 14.250 |
| Miscellaneous | | | 10.000 |
| | | Subtotal | 84.250 |
| | | Total | NOK 260.250 |

TASK 8 - External Norwegian Consultance Work

| | Rate | Quantity | Costs |
|---|---|---|---|
| A. Personnel | | | |
| 8.1 NGI personnel | NOK 400/hr | 160 hrs | 64.000 |
| NTH personnel | NOK 400/hr | 160 hrs | 64.000 |
| 8.2 NGI personnel | NOK 400/hr | 400 hrs | 160.000 |
| NTH personnel | NOK 400/hr | 400 hrs | 160.000 |
| 8.3 NGI personnel | NOK 400/hr | 120 hrs | 48.000 |
| NTH personnel | NOK 400/hr | 120 hrs | 48.000 |
| 8.4 NGI personnel | NOK 400/hr | 200 hrs | 80.000 |
| NTH personnel | NOK 400/hr | 200 hrs | 80.000 |

| | | Rate | Quantity | Costs |
|---|---|---|---|---|
| 8.5 | NGI personnel | NOK 400/hr | 160 hrs | 64.000 |
| | NTH personnel | NOK 400/hr | 160 hrs | 64.000 |
| | | | Subtotal | 832.000 |

B. Travel Expenses

| | Quantity | Costs |
|---|---|---|
| NGI (Oslo-Houston) | 2 trips | 35.000 |
| NTH (Oslo-Houston ) | 2 trips | 35.000 |
| " (Oslo-Trondheim) | 4 trips | 6.000 |
| | Subtotal | 76.000 |
| | Total | NOK 908.000 |

TASK 9 - Design Specifications

Budget presently unspecified. More detailed discussions with Conoco required.

TASK 10 - Participation in and Evaluation of Follow-up Field Test

| | Rate | Quantity | Costs |
|---|---|---|---|
| **A. Personnel** | | | |
| Senior Research Eng. | NOK 450/hr | 120 hrs | 54.000 |
| Research Engineer | NOK 400/hr | 240 hrs | 96.000 |
| | | Subtotal | 150.000 |
| **B. Travel Expenses** | | | |
| Transportation Oslo-Test Site | 12.000/trip | 1 trip | 12.000 |
| Subsistance | NOK 570/day | 5 days | 2.850 |
| Miscellaneous | | | 2.000 |
| | | Subtotal | 16.850 |
| | | Total | NOK 166.850 |

TENSION PILE PLANNING STUDY
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APPENDIX 2
RESUMEES OF PERSONNEL

CURRICULUM VITAE

| | |
|---|---|
| NAME | : Kvalstad, Tore Jan |
| DATE OF BIRTH | : 7th April 1947 |
| NATIONALITY | : Norwegian |
| LANGUAGES | : English and German |
| PROFESSION | : Civil Engineer |
| PRESENT POSITION | : Senior Research Engineer. Head of Section, Section for Geotechnics and Foundations, Research Division Det norske VERITAS |
| EDUCATION | : Civil Engineer (M.Sc.) from Federal Institute of Technology (ETH), Zürich, Switzerland, 1971. M.Sc. thesis on prefabricated, prestressed concrete bridges. |

PROFESSIONAL EXPERIENCE

1972 - 1976 : Scientific Assistant and from 1974 Research Engineer at the Institute of Foundation Engineering and Soil Mechanics, Federal Institute of Technology, Zürich, Switzerland.

- Finite element analysis and laboratory investigations for large earth dams in Switzerland, Greece and Morocco.

- Field investigations for various types of structures.

1976 - : Research Engineer (from 1979 Senior Research Engineer) at Det norske VERITAS, Research Division, Department for Ocean Environment and Geotechnics.

- Laboratory investigations and theoretical work on the effects of repeated loading on soil.

- Evaluation of full scale measurements on gravity platforms during installation.

- Work on:
  - pipeline stability and pipeline/soil/wave interaction
  - foundations for jack-up platforms
  - anchorage of floating structures
  - foundations for gravity platforms
  - foundations for piled platforms

PUBLICATIONS:


1. Foss, I., Dahlberg, R. and Kvalstad, T.J. (1978): "Design of Foundations of Gravity Structures against Failure in Cyclic Loading." Offshore Technology Conference, Houston.

2. Kvalstad, T.J. and Dahlberg, R. (1979): "Soil Reaction Stresses on the Base Structure of Gravity Platforms during Installation." 7th European Conference on Soil Mechanics and Foundation Engineering, Brighton.

3. Kvalstad, T.J. and Dahlberg, R. (1980): "Cyclic Behaviour of Clay as measured in Laboratory." International Symposium on Soils under Cyclic and Transient Loading, Swansea.

4. Foss, I., Kvalstad, T.J. and Ridley, T. (1980): Sea Bed Anchorages for Floating Offshore Structures." Paper prepared for FIP Commission on Sea Structures, Working Group on Foundations, Meeting in Delft, April 1980.

# CURRICULUM VITAE

| | | |
|---|---|---|
| NAME | : | Dahlberg, Rune Georg |
| DATE OF BIRTH | : | 24th April, 1939 |
| NATIONALITY | : | Swedish |
| LANGUAGES | : | English, some German and some French |
| PROFESSION | : | Civil Engineer |
| PRESENT POSITION | : | Principal Surveyor. Head of Section for Foundations and Soil Mechanics, Industrial and Offshore Division, Det norske Veritas |
| EDUCATION | : | Civil Engineer (M.Sc.) from Royal Institute of Technology (KTH), Department of Soil and Rock Mechanics, Stockholm, Sweden, 1965.<br><br>Doctor of Technology, Royal Institute of Technology (KTH), Department of Soil and Rock Mechanics, Stockholm, Sweden, 1975. <u>Thesis</u>: Settlement Characteristics of Preconsolidated Natural Sands (In-Situ Screw Plate, Pressuremeter and Penetration Tests). |

## PROFESSIONAL EXPERIENCE

| | | |
|---|---|---|
| 1965 - 1971 | : | Consulting and R & D work at the Swedish Geotechnical Institute, Stockholm. Foundation design for bridges, motor-ways, etc., and development work in the field of penetration testing. |
| 1968 - 1975 | : | Scientific Assistant and Research Fellow at the Royal Institute of Technology, Stockholm. Thesis work and teaching, Coordinator of the teaching program in Soil Mechanics and Foundation Engineering. (1968 - 1971 part-time at the Swedish Geotechnical Institute). |

Secretary of the Committee on Penetration Testing of the Swedish Geotechnical Society during 1968 - 1975.

Member of the Organizing Committee for the European Symposium on Penetration Testing, 5th-7th June 1974, Stockholm.

Technical Secretary for Discussion Group on "Interpretation of Results from Static Penetration Tests", European Symposium on Penetration Testing, 5th-7th June, 1974, Stockholm.

1975 - 1976 : Research Engineer at Det norske Veritas, Research Division, Geotechnical Group.

Co-reporter of the industry sponsored Research Project "Repeated Loading of Clay" (administrated by Shell and NGI).

1976 - 1980 : Senior Research Engineer and Head of the Geotechnical Group, Research Division. Project manager and coordinator for research projects related to soil mechanics and marine geotechnical engineering.

Member of the Committee on the "Use of Computer Services in the Field of Soil Mechanics and Foundation Engineering in Norway" (Norwegian Geotechnical Society).

1980 - : Principal Surveyor and Head of Section for Foundations and Soil Mechanics, Industrial and Offshore Division. In charge of foundation review and evaluation for certification of offshore structures including associated instrumentation systems and installation problems. Supervision of offshore site investigations and platform installations.

# CURRICULUM VITAE

| | | |
|---|---|---|
| NAME | : | Kjell Hauge |
| DATE OF BIRTH | : | 15. June 1954 |
| NATIONALITY | : | Norwegian |
| LANGUAGES | : | English |
| PROFESSION | : | Civil Engineer, Geotechnical |
| PRESENT POSITION | : | Research Engineer, Section for Geotechnics, and Foundations, Research Division, Det norske Veritas |
| EDUCATION | : | Bachelor of Science in Civil Engineering 1977, Master of Science in Civil Engineering 1979 from University of Colorado, Department of Civil Environmental and Architectural Engineering. M.Sc. thesis: "Evaluation of Dynamic Measurement System on the Standard Penetration Test". |

## PROFESSIONAL EXPERIENCE

| | | |
|---|---|---|
| 1977-79 | : | Research Assistant at Univ. of Colorado, Dept. of Civil Engr. Worked under Prof. G.G. Goble on pile dynamics, pile capacity and dynamic measurements of piles. Also structural analysis and soil mechanics. Research work and teaching. |
| 1979-80 | : | Norwegian Army Corps of Engineers. Military Service. Corporal. Assistant to the educational officer at the office for adult education. |
| 1980 | : | Present position at Det norske Veritas. |

- Evaluation of full scale measurements on piled platforms.
- Development of laboratory equipment for pile-soil interaction investigation-

Member of the Polytechnical Society (Den Polytekniske Forening).

CURRICULUM VITAE

| | | |
|---|---|---|
| NAME | : | Flaten, Gunnar |
| DATE OF BIRTH | : | 10th March 1931 |
| NATIONALITY | : | Norwegian |
| LANGUAGES | : | English and some German |
| PROFESSION | : | Senior Technician in Geotechnic and Machinery Engineer |
| PRESENT POSITION | : | Research Engineer, Geotechnical Laboratory, Research Division, Det norske VERITAS |
| EDUCATION | : | Construction Study at NKI-skolan, Sweden. Machinery Engineer at Norwegian Defense Research Institute (62) |

PROFESSIONAL EXPERIENCE

| | | |
|---|---|---|
| 1951 - 1952 | : | Laboratory Assistant at Norwegian Electrical Material Control (NEMKO). Approval of electrical equipment and apparatus. |
| 1952 - 1956 | : | Laboratory Technician at Norwegian Geotechnical Institute (NGI). Determination of Soil Properties. Design and building up of Geotechnical Laboratories. |
| 1956 - 1968 | : | Head of Laboratory at NGI. Management, purchases and research work. |
| 1968 - 1974 | : | Senior Technician at NGI. Building up and testing out geotechnical apparatus. Static and Dynamic analysis on soil material. Offshore geotechnical activities - soil profiles, identification and classification. |

1974 - : Research Engineer at Det norske VERITAS. Present position as responsible manager of the Geotechnical Laboratory. Soil testing under dynamic and static conditions with Triaxial and Torsional shear apparatus. Operating VERITAS Data-Lab to produce listing and plots required. Standard soil testing.

Member of The International Society for Soil Mechanics and Foundation Engineering.

CURRICULUM VITAE

NAME : Ronold, Knut Olav
DATE OF BIRTH : 14th January 1956
NATIONALITY : Norwegian
LANGUAGES : English, Danish, Norwegian
PROFESSION : Geotechnical Engineer

PRESENT POSITION : Research Engineer, Det norske VERITAS

EDUCATION : Civil Engineer (M.Sc.) from Norwegian Institute of Technology, Division of Soil Mechanics and Foundation Engineering, 1978. M.Sc. thesis "Dike Stability. Geotechnical Stability Calculations in Connection with Dikes and Dike Construction and with Special Reference to the Ribe Dike in Jutland."

PROFESSIONAL EXPERIENCE

1979 - : Research Engineer at Det norske VERITAS, Research Division, Section for Geotechnics and Foundations.

- Analysis of foundation problems related to installation of gravity platforms

- Analysis of full scale measurements on gravity base and piled platforms

- Development of geotechnical computer programs in Basic and Fortran.

- Development of dynamic pile analysis program for soil-pile-structure analysis SESAM-80

CURRICULUM VITAE:

| | |
|---|---|
| NAME | : Arnesen, Knut |
| DATE OF BIRTH | : 5.8. 1951 |
| NATIONALITY | : Norwegian |
| EDUCATION | : M.Sc., The University of Trondheim, The Norwegian Institute of Technology, 1974 Civil Engineering |
| POSITION | : Senior Surveyor<br>Geotechnical Group<br>Foundation/Concrete Structures<br>Industrial and Offshore Division |
| EXPERIENCE | : |
| 1975 - 76 | : Royal Norwegian Navy |
| 1976 - 77 | : Det norske Veritas<br>Participant in DnV's Systematic Training Program. One period in the Industrial and Offshore Division's concrete group, and another period in the Research Division's geotechnical group. |
| PRESENT WORK | : Det norske Veritas<br>Industrial and Offshore Division.<br>Surveyor, geotechnical group.<br>Appraisal of foundation design for fixed offshore structures, including site investigations, laboratory testing, geophysical surveys, foundation analyses and instrumentation systems.<br>Supervision of offshore site investigations and platform installations. |