

**INSTITUT FRANÇAIS DU PÉTROLE
LABORATOIRES DES PONTS ET CHAUSSÉES**

**SYMPOSIUM
ON THE
PRESSUREMETER
AND ITS MARINE
APPLICATIONS**

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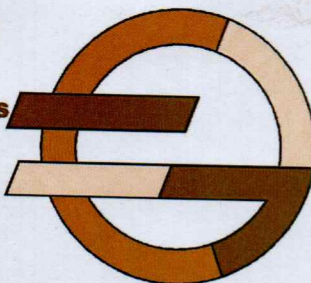
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**INSTITUT FRANÇAIS DU PÉTROLE
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P. HABIB*

ADDRESS

The applications of the pressuremeter to offshore operations calls for various introductory remarks.

Let us first acknowledge the absolute need of geotechnical reconnaissance for offshore operations. This is obvious for the Soil Mechanics Expert, whose first reflex is to know the soil and its mechanical properties, to put them the best use and to foil the tricks of Nature. The geotechnician also has access to a whole collection of accidents which have occurred on land when constructive enthusiasm (barring a case of false economy) has overlooked the necessary geotechnical prudence, and where this has resulted in a calamity, an accident, or simply in disorder. However, the geotechnician knows nothing at all about the sea, neither waves nor tides the realm of the sailor who, on the other hand, knows almost nothing about the seabed in which he casts his anchor. These are two different worlds, which had no reason to meet before the exceptional problems raised by the construction and operation of the great offshore oil structures. Anchorings of these structures sometimes hold sway over a radius of several kilometers, and the site is generally very poorly known in terms of its geological and geotechnical environment. We are all aware that geological maps are, if not non-existent, at least totally inadequate at the onset of the exploration of a new field, and the race to greater depths is not improving this situation. Hence the need to bring all the resources of the art into play, and to do everything possible to 'marinize', in other words, to make seaworthy the many different methods implemented on land : sample taking, static and dynamic penetrations tests, vane probe, pressuremeter. The results obtained with varying success using these methods, both onshore and offshore, are difficult to compare. They correspond to different approaches to

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the problems of foundation practice, and complete discussions have taken place in many seminars to correlate the different results obtained on land. These discussions are not always neutral, since commercial interests are often involved. Offshore, naturally, the investments required for reconnaissance are so large that discussions are simply sharper. The subject of our meeting today is restricted to the use of the pressuremeter and to the comparison of experiments conducted in different countries for offshore and coastal operations.

One cannot speak of the pressuremeter without evoking the name of Ménard. Louis Ménard discovered the principle of this invention, it must be recalled, when he was an engineering student at the Ecole Nationale des Ponts et Chaussées in Paris. The initial patent dated from 1955, the first intensive tests were performed from 1957 to 1960, and the first instructions for use drafted in 1962/1963. I remember meeting Ménard in 1954 at the CEBTP and being struck by his creative enthusiasm. The pressuremeter theory has progressively expanded, and Continuum Mechanics Studies have been made in many countries, with increasingly complex laws of behavior, while simultaneously the geographic spread of the process has undergone tremendous growth, to the point where the pressuremeter now forms part of the conventional arsenal available to the geotechnician for foundation design. The Ménard pressuremeter test is performed on land with simple equipment, which is ideal for the reconnaissance of a site in difficult conditions. Refinements have been added to the method, sometimes by Ménard, such as when he refined the interpretation of the test, especially by the analysis of delayed effects, or by others, by refining performances, particularly by means of self-boring techniques, a decisive step to guarantee the least possible disturbance of very sensitive or very compressible soils, and all soils for which the simple operation of taking intact samples already raised difficult if not insurmountable problems.

Offshore geotechnical reconnaissance brings an added constraint to all these problems, i.e. the problem of working from a ship, which multiplies the costs out of all proportion to those prevailing onshore. The fixity of the vessel in relation to the seabed test, the vessel's dimensions, and the time required to perform intermediate operations for in situ tests, are extremely costly requirements with repercussions on the performing of the tests. This applies to measurements which have to preserve very high sensitivity, considering that an accuracy of a few dozen kPa is required in the dilatation of the pressuremeter at water depths of 600 m, i.e. 6 MPa, without really involving a markedly exceptional situation.

The different papers which will be presented to you here, of which you have already received the texts, offer extremely interesting answers to some of these problems, and will convince you of the timeliness of the Symposium.

SESSION 1

Chairman : P. HABIB
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The pressuremeter method

F. BAGUELIN*, P. LE TIRANT**

THE PRESSUREMETER METHOD ONSHORE AND OFFSHORE APPLICATIONS

ABSTRACT

Since the pressuremeter was invented by Louis MÉNARD in 1954, pressuremeter method has developed for two types of testing : (i) testing with penetration of the probe into the ground by prior drilling or by being driven in, which is the conventional test developed by MÉNARD and now in common practice ; (ii) testing with penetration of the probe by self-boring, originally developed by the Laboratoires des Ponts et Chaussées.

The conventional onshore pressuremeter method today is widely used for engineering for structures. Furthermore, self-boring pressuremeter method opens up interesting possibilities for making basic analysis of soil behavior and for various fields of applications.

For offshore uses, an effort obviously has to be made to succeed in obtaining a better understanding of subsea soil properties and hence in being able to optimize the sizing of foundations and the anchoring of structures. With this in mind, the application of the pressuremeter is such that considerable progress can be made although the method is still in an early stage.

This paper includes two parts :

(i) The first part examines the development and validity of Louis MÉNARD'S pressuremeter method and then the advantage of self-boring pressuremeter method.

(ii) The second part reviews methods commonly used for offshore

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soil reconnaissance before going on to examine the present applications and advantages of offshore pressuremeter method.

INTRODUCTION

Since Louis MÉNARD invented the pressuremeter in 1954, pressuremeter method has developed for two types of tests (Baguelin et al., 1978) : (i) testing with penetration of the probe into the soil by prior drilling of by driving, which is the conventional test developed by MÉNARD and commonly used ; (ii) testing with penetration of the probe by self-boring, originally developed by the Laboratoires des Ponts et Chaussées.

On land, the conventional pressuremeter method is widely used for the engineering for structures. As soon as it began being used, it appeared as a valid alternative to more traditional methods of soil analysis. In the field of foundation engineering, it proved to be particularly effective, and preference was granted to it in 1972 in French technical recommendations published by the Direction des Routes et de la Circulation Routière (SETRA-LCPC, 1972). As of now, thousands of onshore structural foundations have been sized on the basis of the pressuremeter method. More recently, self-boring pressuremeter method has opened up interesting prospects for making fundamental investigations of soil behavior in various fields of practical applications.

At sea, it became evident that conventional coring or in situ measuring techniques were poorly suited and could not determine the actual mechanical properties of the soils in place. This situation gradually got worse with the increase in water depths because of the implementation conditions of the equipment used. An obvious effort had to be made if better understanding were to be obtained of subsea soil properties and, consequently, more effective sizing of structural foundations and anchoring. From this standpoint, the use of the pressuremeter should enable great progress to be made, although it is still in an early stage.

This paper consists of two parts :

(i) The first part examines the main principles of the pressuremeter method as it is practiced onshore, i.e. the development of the pressuremeter method by Louis MÉNARD, the value of the MÉNARD pressuremeter method, and self-boring pressuremeter method.

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(ii) The second part attempts to evaluate the value of the pressuremeter method for subsea soil reconnaissance by reviewing the following points : current reconnaissance methods, offshore applications of the pressuremeter method and the advantages of offshore pressuremeter method.

1. ONSHORE PRESSUREMETER METHOD

1.1. Development of the pressuremeter method by Louis MENARD

As a method for soil investigation, the pressuremeter method is the result of original development operations carried out by Louis MENARD from the three standpoints of equipment, test interpretation and making use of test results. The first two points were developed approximately at the same time between 1954 and 1957. They thus gave rise to the pressuremeter test. The third point was worked out a bit later, mainly between 1960 and 1963. It was by these three aspects that Louis MENARD broke away from the achievements of his predecessor, F. KOGLER, whose probe built in 1930 is almost unknown.

1.1.1. Equipment

The pressuremeter is a dilatable cylindrical probe which is generally run into a borehole or sometimes driven into the soil (slotted tube technique). The test at a given depth consists in measuring the pressure-volume relationship during the dilation phase. In addition to making a judicious selection of materials available at that time (rilsan, rubber canvas, etc.), Louis MENARD made two essential choices in designing his pressuremeter :

(i) the tricellular structure of the probe, so as to provide a flat deformation field corresponding to theory, an arrangement which is indispensable when the probe is positioned in a previously drilled borehole or is driven into the soil, inevitably leading to considerable deformation during testing ;

(ii) using water to inflate the measuring cell (as opposed to Kögler who used gas), do that a simple and accurate measurement of the deformation can be obtained.

1.1.2. Interpreting the test

As opposed to F. KÖGLER, L. MENARD proposed the correct interpretation of measurement data by making use of suitable theories: (i) obtaining a so-called pressuremeter module (E_M) by applying the theory of cylindrical expansion in elastic media to the linear phase of the test, a theory developed by LAME (1852) and showing the essentially deviatoric nature of the test during this phase;

(ii) the existence of an asymptotic value of pressure (limit pressure p_0) had been shown by research by BISHOP, HILL and MOTT (1945) for purely coherent soil, but MENARD (1957) extended these results to frictional media that may eventually be endowed with cohesion, and he brought out other interesting points in the plastic phase of the tests such as, the relationship existing between the sub tangent to the pressure axis and the radius of the MOHR circle. In this way, the significance of the pressuremeter test was clearly recognized.

Nonetheless, the use of theories in the opposite direction, i.e. to derive soil properties from test results, led to a dead end. The values of deformation moduli and "to an even greater extent" of the MOHR-COULOMB parameters deduced from the pressuremeter test were widely different from the values provided from conventional tests.

A great many scientists and engineers emphasized the disturbances provided by setting the probe in place and concluded that pressuremeter testing did not have a sound basis and so lost interest in it. As for L. MENARD, he rejected conventional concepts of soil mechanics and turned towards other methods.

1.1.3. Making use of tests results

L. MENARD clearly saw that the assumptions used by conventional theories (elasticity, plasticity) did not take into consideration the behavioral complexity of actual soils (e.g. mention can be made of non-linearity, viscosity, sensitivity, dilatancy or compressibility, etc.). These insufficiencies were responsible for the failure in applications of theories to pressuremeter testing, but wasn't there some possibility that they could also bring into question conventional theories of structures behavior? In the case of phenomena similar to the ones involved in pressuremeter testing, why not search for a direct correlation, having an empirical nature, between the two solicitations (pressuremeter test and behavior of structures).

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By undertaking actual foundation tests, L. MÉNARD proposed formulas relating:

- the punching pressure of foundations and the limit pressure,
- the settlement of shallow foundations and the pressuremeter modulus,

or again, by analyzing data from the literature and comparing pressuremeter test results and conventional test results, formulas relating:

- the shaft friction of piles and the limit pressure,
- the lateral reaction modulus of piles or subgrade reaction modulus of foundations rafts or screens and the pressuremeter modulus.

It is mainly in this form that the pressuremeter method has been applied in practice.

1.2. Validity of the L. MÉNARD pressuremeter method

First of all, the pressuremeter test must be considered as such and then as a method for foundation engineering with the operational method proposed by L. MÉNARD.

1.2.1. Validity as a soil mechanics test

Like other soil mechanics tests, the pressuremeter test must meet various operational conditions before being considered valid:

- first of all, inserting the probe into the ground must be considered to be an integral part of the test, which is not merely an expansion. This principle is peculiarly important when the probe is inserted in a borehole,

- secondly, a test procedure must be respected. In France since 1971, the reference is the test procedure developed by the Laboratoire Central des Ponts et Chaussées which takes up the general system adopted by L. MÉNARD. It establishes various test parameters (number and duration of increments, for example) and indicates the drilling processes to be used depending on the nature of the formation.

Given these conditions, the pressuremeter test has two main characteristics for being considered as a valid soil-mechanics test:

- (i) it is reproducible within limitations commonly accepted in soil mechanics;

(ii) it provides a significant reflection of variations in the mechanical properties of soils, either inside the same geological formation or for soils of different origin or nature.

As opposed to other conventional in situ tests, it provides two sorts of parameters, i.e. a deformation parameter and a resistance parameter which have a more or less conventional nature and hence a significance which is not absolute, but the same is true for all soil-mechanics tests. Compared with tests on core samples, it has the dual advantage of in situ tests of being able to survey large volumes of soil and to provide an image of variation in the mechanical properties within this volume.

1.2.2. Validity as a foundation engineering method

The design method associated with the pressuremeter test covers several types of geotechnical structures but has been especially developed for foundations.

This method is based on direct correlations between the test parameters and the parameters which characterize the behavior of actual foundations. Therefore, its empirical nature must be emphasized and it should be realized that the method can be applied only to conventional pressuremeter tests, i.e., tests performed with L. MÉNARD equipment and according to the test procedure requirements.

French experience and especially that gained by the Laboratoires des Ponts et Chaussées, which have commonly practiced this method and made an evaluation of it on actual structures, is as follows in the field of onshore foundations:

(i) The pressuremeter method has fewer operational limitations than other investigational methods. Indeed, it is practicable in almost all formations with the exception of soils made up of boulders, whereas mechanical tests using core samples can be performed only in fine silty or clayey soils, and penetrometer tests, of static or dynamic type, are limited by the presence of very hard layers at the same time as giving extremely erratic results which are often quantitatively useless in heterogeneous soils.

(ii) The pressuremeter method generally provides more accurate diagnoses and especially more favorable ones than other methods with regard to foundation behavior (higher punching pressures, smaller settlements), and yet experience shows that it does not lead to any unsatisfactory results.

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For these reasons the method was recommended as a basic method for foundation engineering in France for work to be done by the Highway Department (FOND.72 document). Nonetheless this does not mean that pressuremeter method is the only way of investigating a site. The geological context must be specified beforehand by appropriate methods, and additional investigations making use of other tests may be performed as well to solve specific problems.

The research performed by the Laboratoires des Ponts et Chaussées, in addition to several results from other origins, have enabled the accuracy of the method to be determined. Data from actual structures, as well as results from experimental sites show that, with regard to predicting the bearing capacity and settlement, the real values are no more than $\pm 50\%$ different from predicted values and that more often than not (in 80% of the cases) they are better than $\pm 30\%$.

On the whole, the pressuremeter method has led to a decrease in the size of foundations, in particular it has allowed more shallow foundations to be built.

Determining the reaction modulus of piles from the pressuremeter modulus has also had an impact on the more widespread use of drilled piles. By giving less pessimistic values than other methods and for a wider variety of soils, it has encouraged the use of foundations set on vertical piles having larger diameters when compared with more traditional solutions of small-diameter piles, some of which are battered.

1.3. Self-boring pressuremeter method

The self-boring technique began being developed in France beginning in 1965 but was not described in detail until a publication in 1972 (Baguelin, Jézéquel, Lemée, Le Méhauté, 1972). The term "autoforage" appeared in 1971 (LCPC Pressuremeter Test Procedure) and its English translation "self-boring" in 1972 in the above mentioned article.

At about the same time, HUGHES and WROTH in Great Britain were working on problems of in situ measurements of natural pressures in soils and in 1973 they described a device called a "Camkometer".

The leading feature of self-boring is that the probe is inserted into the ground with a very little amount of disturbance, if any, and in any case, a great deal less than those caused by coring techniques or conventional processes of inserting probes.

In 1972 as well, a theory of undrained testing was proposed which went beyond the restrictive assumptions commonly accepted in conventional theories (elasto-plasticity).

All these results could be combined to open the route for the in situ determination of the fundamental parameters of cohesive soils. Subsequent progress in performing the test and in theoretical developments enabled the range of application of the method to be extended to sandy materials.

Various special features were developed for the equipment:

- (i) a device for measuring the pore pressure at the pressuremeter soil-membrane contact, and
- (ii) a cyclic loading system for the soil (up to 10 000 cycles). These developments widened even further the range of fundamental research on soil behavior.

It was also of interest to start along another path concerning self-boring and, as L. MENARD had done for his pressuremeter, to search for direct correlations between the test parameters and the ones characterising the behavior of structures. This has already been done for problems of foundation punching and settlement, and partially for the behavior of piles subjected to horizontal stresses.

2. APPLICATION AND IMPORTANCE OF THE OFFSHORE PRESSUREMETER METHOD

For some 20 years, technicians and experts have been trying to adapt onshore techniques for soil reconnaissance by coring or in situ measuring to offshore applications with more or less success depending on the environment (water depth, meteo-oceanological conditions, etc..).

2.1. Current offshore soil reconnaissance techniques

To understand the importance of offshore pressuremeter applications, it is useful to begin by making a brief review of the reconnaissance methods now used and their fields of application and limitations (P. Le Tirant, 1979).

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2.1.1. Offshore coring techniques

Offshore coring techniques for geological and geotechnical purposes make use of a wide variety of processes depending on:

- (i) the water depth (from several meters to several thousand meters),
- (ii) the penetration to be achieved (from few meters prior to the laying of pipelines to more than 100 meters for the installation of structures on piles or anchoring points), and
- (iii) the presumed nature and properties of these formations.

Different coring techniques and devices can be classified in four categories depending on the process involved, the core-barrel driving principle and the maximum penetration to be reached.

a) Cable core drilling through the drill string from a surface support (ship or possibility exploration drilling platform) makes use of various methods, i.e. driving in the core sampling tube by jacking down or hammering or else rotary drilling. This is by far the most widely used process for soil reconnaissance prior to the installation of offshore structures requiring and understanding of soils at water depths of several tens of meters to more than 100 meters.

The quality of the core samples recovered, which is often quite deficient, nonetheless depends on various ways of implementation (H.M. Zuidberg et al., 1979 ; G. Henderson et al., 1979). Stabilizing the drill string by means of a packer taking up the reaction on the borehole walls or by means of a seabed framework (also applied for performing in situ measurements) provides an appreciable improvement in the quality of samples taken.

b) Coring with submerged probes set on the seabed and operated by divers or remote controlled (core barrel system), depending on the water depth, mainly uses the rotary drilling process and is chiefly applied with stiff soils, with a penetration depth eventually reaching 50 to 100 meters.

c) Vibrocoring implemented according to quite different processes (device set on the seabed or maintained as a plumb bob) is mainly applied in heterogeneous or coarse soils (with core samples up to 30 cm in diameter) or in relatively stiff superficial soils where coring by means of freefall core barrels is ineffective. Penetration

varies from few meters to 20-30 m depending on the equipment used and the formations encountered. The cores recovered are often disturbed by the vibrodriving of the core barrel in the soil but are still of great interest for identifying rough superficial materials such as moraines, boulder clays, etc.

d) Coring by means of freefall core barrels (commonly called "core barrels of the Kullenberg type"), which is generally done in the pre-reconnaissance phase of sites from oceanological vessels, is not limited by the water depth. The core samples recovered, which are generally less than about ten meters long, are almost always considerably disturbed and, as a result, are not very representative of the mechanical properties of the superficial soils.

2.1.2. In situ measuring techniques

The offshore application of in situ measurements and especially of penetrometer method is performed either through a drill string or by means of a framework set on the seabed.

Drilling using an in situ measuring device through the drill string is more and more commonly used. The devices used are:

- (i) mainly the penetrometer which is apparently used in different types of formations such as sand, silt and soft or compact clay;
- (ii) sometimes the vane probe in soft soils;
- (iii) relatively recently the pressuremeter (§ 2.2.).

Achieving correct measurements requires good stability of the drill string just as for coring (§ 2.1.1.), but the process can be used for performing measurements down to the maximum depth obtained by drilling.

Performing in situ measurements from a framework set on the seabed has the advantages of freeing the support from any instability constraints and can be operated under water depths of several hundred meters. The method is: (i) commonly used for the penetrometer, with penetration varying from a few meters to more than 20 meters depending on the nature and consolidation of the formation; (ii) being developed for the self-boring pressuremeter (§ 2.2.2.).

2.1.3. Conclusions

Each coring or in situ measuring method has obvious limitations of use. There is no universal method for offshore soil reconnaissance. Nonetheless, a few important remarks should be made:

THE PRESSUREMETER METHOD. ONSHORE AND OFFSHORE APPLICATIONS

- (i) Core samples taken offshore are always more or less disturbed, with the degree of disturbing depending on the nature and sensitivity of the soil, the coring procedure and the water depth,
- (ii) In situ measurements are taking on and will inevitably continue to take on increased importance with the increase in the water depths involved, with the quality of the core samples recovered necessarily varying inversely with the water depth (decompression, etc..).
- (iii) The penetrometer, which is perfectly suited to investigating sandy soils and compact clay, is much less advisable for the reconnaissance of silty and very soft clayey soils of the type often encountered offshore, especially in deltaic zones and beyond the continental shelf.

2.2. Offshore application of the pressuremeter method

Any pressuremeter test includes two successive operations, i.e. setting the probe in place and then expanding the cell together with data acquisition. These two phases involve various difficulties which are all the more pronounced as the water depth is great:

- (i) possibilities of intervention and direct control become almost impossible;
- (ii) tubing lengths create considerable mistakes if the measurement diagnosis is performed on the surface. These comments at least partially explain the as yet limited development of the offshore pressuremeter method.

2.2.1. Offshore applications of conventional type pressuremeters

At the very beginning of the 1960s, Technique Louis Ménard began developing the offshore application of the pressuremeter (L. Ménard and M. Gambin, 1965 ; M. Gambin, 1970 ; M. Gambin, 1979 ; J. Leblanc, 1982). Different implementation techniques were used at water depths ranging from several tens of meters to about 100 meters:

- (i) vibrodriving of the probe by a plumb-bob method from the ship;
- (ii) vibrodriving the probe via a guide tube running between the surface and the seabed, and which was first made to penetrate for several meters into the seabed ;
- (iii) vibrodriving the probe by means of a vibrodriver (such as the annular vibrodriver) sitting on the seabed;
- (iv) dropping the probe by gravity as for the Kullenberg type of coring.

Since 1980, Stressprobe Ltd. (W.M. Reid et al., 1982) have been using the Push-in Pressuremeter (PIP) developed by the Building Research Establishment (G. Henderson et al., 1979) via the drill string for offshore soil reconnaissance. The process requires that the drill string be correctly compensating for the reaction on the borehole walls (Fugro) or by means of a seabed framework weighing about 15 tons (Mc Clelland's Stingray). The probe is jacket down into the soil.

A device of the same type called the "Expansometer" is being developed by Syminex (A. Légier, 1982). Since core samples are obtained from the pre-bored hole, the exact nature of the formations crossed through can be determined.

The pressuremeter-penetrometer developed in 1981 by the Laboratoires des Ponts et Chaussées together with the Commissariat à l'Energie Atomique (J.F. Jézéquel et al. 1982) and set in place by an annular vibrodriver developed by TLM (Techniques Louis Ménard) has been operated successfully at water depths of 40 to 50 meters.

2.2.2. Developing the self-boring pressuremeter for offshore applications

The self-boring pressuremeter which has been developed for onshore applications by the Laboratoires des Ponts et Chaussées over the last 15 years has now been redesigned in two different versions for offshore soil reconnaissance.

(i) A device called PHYFI (submerged hydraulic drilling platform) built with the Service Central Technique des Ports Maritimes et Voies Navigables and operated by divers (F. Baguelin et al., 1982; D. Bisson et al., 1982) consists of a submerged framework designed to drive in a self-boring probe of the PAF 76 type at water depths of up to 40 meters.

(ii) A device called the offshore self-boring pressuremeter (PAM) built by the Institut Français du Pétrole with the assistance of the Laboratoires des Ponts et Chaussées (P. Le Tirant et al., 1981; P. Le Tirant et al., 1982; J.B. Fay and P. Le Tirant, 1982) mainly consists of a submerged framework which is remote-controlled from the surface for operating a LPC/IFP offshore self-boring probe (F. Brucy et al., 1982). This device which is effectively operational at water depths of up to 300 meters will soon be equipped for applications at water depths of 1 000 meters.

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2.3. Importance of the offshore pressuremeter method

The pressuremeter method, which has led to progress in foundation engineering for onshore structures, may also be a starting point for important advances in the design of offshore structures. Its widespread offshore use could possibly make for considerable improvements in understanding the geotechnical properties of marine sediments.

2.3.1. Data acquired by the pressuremeter

The pressuremeter method is the only in situ measuring process capable of both determining the stress-deformation relations of the soil to the failure and of performing cyclic loading tests of the soil in place, thus providing indispensable data for the design and then optimization of offshore foundations structures and for making a more accurate evaluation of their safety. As onshore, the data acquired offshore by a pressuremeter can be processed either by making direct correlations with the characteristic parameters of behavior of structures or in the form of elementary soil parameters (shear strength, modulus).

At sea, obtaining representative values on the spot provides a decisive advantage compared with tests using core samples which are always considerably disturbed. The application of the self-boring pressuremeter, which is made to penetrate into the soil without appreciable disturbing of the formations, considerably improves the quality of the data acquired from the standpoint of both deformability and behavior under cyclic loading.

2.3.2. Importance of the offshore pressuremeter as a function of the nature of formations

Use of the pressuremeter and especially of the self-boring pressuremeter appears of special interest for the reconnaissance of certain types of marine sediments where it is almost impossible to take representative core samples with the techniques now used. This is the case in particular :

- (i) with soft or very soft soils having a shear strength ≤ 5 to 10kPa such as are encountered in deltaic zones and cover most of the seabed beyond the continental shelf;
- (ii) with carbonate sediments (silts and sands) having a very fragile skeleton that is extremely sensitive to any disturbing and which cover very extensive areas in warm seas at shallow or medium depths.

2.3.3. Problems to be solved

The size and importance of fixed or floating structures installed at sea require greater and greater improvements in engineering which pressuremeter data alone can provide. Among the specific problems to be solved, considering the special conditions involved (size of foundations, types of stresses, forces due to the environment, etc..), mention will be made in particular of:

(i) the engineering of piles and groups of anchoring piles subjected either to horizontal, static, cyclic or dynamic loading or else to vertical, static or cyclic loading in tension or compression;

(ii) the design of conductor pipes for oil or gas wells;

(iii) evaluating the settlement of different types of structures sitting on the seafloor, i.e. pipelines, templates, etc.

The obvious similarity between the behavior of piles subjected to cyclic loading and the results of cyclic pressuremeter tests suggests that the efforts being undertaken will provide fruitful results to help with achieving the acceptable engineering and design of piles and groups of piles for foundations and anchorings.

The extension of petroleum exploration and production onto the continental slope requires an evaluation to be made of the danger of sliding in formations that are often very soft (surface cohesion $C_u < 2$ to 3 kPa), which are sometimes increased in some regions (e.g. Mediterranean Sea) by frequent earthquakes. The water depths involved, now amounting to as much as 2 000 meters, reduce the choice of reconnaissance possibilities.

The development of the offshore self-boring pressuremeter for a water depth of 1 000 meters will at least partially meet the most immediate needs.

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C O N C L U S I O N

This paper provides a very brief overview of the applications of the pressuremeter method and its possibilities for offshore development. Even if the term "pressuremeter" is inseparable from the name of Louis Ménard, one is struck today by the diversity of recent efforts that have been made to extend its range of application with regard to both the equipment used and the in-depth work done on methods for interpreting the results.

The meeting at this "Symposium on the Pressuremeter and its Marine Applications" of persons who have made contributions in this field will doubtless make it possible to provide both better knowledge concerning the different research projects undertaken and fruitful exchanges for the future of the pressuremeter method.

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J. LEBLANC*

MÉNARD PRESSUREMETER TESTING

RESUME :

This article presents the development of the pressuremeter by the "Société des Etudes Pressiométriques Louis Ménard" from its conception in 1954 by Louis Ménard to the present day.

Throughout its evolution, the pressuremeter has always consisted of the same principal elements : the control box for pressure/volume measurements and a dilatable probe with flexible connection. A continuous research program was set up to modify the instrument with technological progress. Adaptation of the standard equipment to offshore sites, the various models that have been fabricated and the device used for difficult soils called the slotted casing are described in brief.

Pressuremeter theory was originally derived from the expansion of a cylindrical cavity in an infinite homogeneous medium. In the pseudo-elastic domaine, the plane strain deformation is quantified by the pressuremeter modulus

$$E_m = 2(1 + \mu) V. \frac{\Delta p}{\Delta V}$$

On reaching the limit state in the plastic deformation domaine, the pressure is called the limit pressure "p₀".

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1. GENERAL HISTORY

The pressuremeter was first studied in 1954 by Louis Ménard for a final year project at the "Ecole Nationale des Ponts et Chaussées". Under the direction of Professor Kerisel the basic apparatus was constructed and the theory formulated. The analysis of the report concluded that "The persistence and imagination of Ménard should be rewarded; however he is not convinced of the practicality of such an instrument, but theoretically it is of great interest. The research is very original and stands out amongst the more usual topics in the geotechnical field."

The first patent was applied for the same year and the development was transferred to the other side of the Atlantic where Ménard was to study under the direction of Professor Peck at the University of Illinois.



Fig 1. Louis Ménard (right) with a colleague and the first pressuremeter probe

MÉNARD PRESSUREMETER TESTING

On returning to France, a company called " Les Pressiomètres Ménard" was set up in Paris. It was here, with the help of a small team of engineers, that the device was perfected for commercial testing. A number of patents followed as innovation led to the production of specialised equipment :

1959 : the slotted casing

1976 : high pressure testing

1981 : Electronic pressuremeter with complete interpretation of the test.

Before going into the equipment in detail, a short section on the theory follows.

11. PRESSUREMETER TESTING

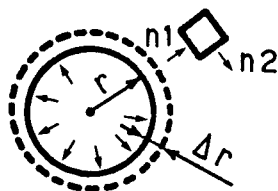
The test consists in the expansion of a cylindrical cavity in the soil.

At small strains, the study adopts the theory of plane strain deformation.

In a continuous elastic medium, the shear modulus is given by the formula

$$\frac{\Delta r}{r} = \frac{1}{2 G} \Delta p$$

Assumptions :



Stress strain relationship is linear

Similar behavior in tension and compression

Reversibility

The time factor is neglected.

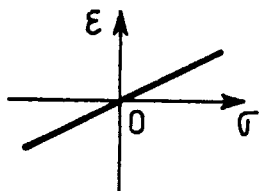


Fig. 2

The schema of strain ϵ with stress σ is shown in Fig 2.

Young's modulus E is defined by the formula

$$G = \frac{E}{2(1 + \mu)} \quad \mu = \text{Poisson's ratio}$$

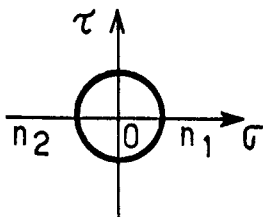


Fig. 3

In a real soil the expansion of a cavity is described by the Mohr's stress distribution of a circle centred at the origin of the principal and tangential stress axes fig 3. The stress field set up by the probe, assuming a strictly cylindrical expansion, satisfies none of the elastic medium conditions especially the reversibility (Fig 4).

However, over a limited deformation range, the condition is quasi satisfied, thus this domaine is called pseudo-elastic.

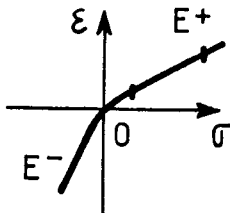


Fig. 4

In this range, the test resembles a normal shear test where E is given by :

$$E_m = 2(1 + \mu) G$$

It should be noted that Young's modulus and the pressuremeter modulus relate to different quantities.

Thus combining the equations,

$$E_m = (1 + \mu) \frac{r}{\Delta r} \cdot \Delta p$$

for a cylindrical probe,

$$V = 2 \pi r^2 \ell$$

$$\frac{\Delta V}{V} = 2 \frac{\Delta r}{r}$$

$$E_m = 2 (1 + \mu) V \frac{\Delta p}{\Delta V}$$

Poisson's ratio is taken as 0.33.

The relationship between $\frac{E}{E_{\text{Young}}}$ is given by a rheological

coefficient α which varies between 0.25 and 1 depending on the soil type.

The pseudo-elastic phase has an upper limit called the creep pressure p_f . Above this value, a plastic behavior is observed in which the deformation increases rapidly with pressure and time. The soil appears to rupture and the $p - v$ curve tends towards infinity, generally called the limit pressure " p_l ". At this point the cavity and the plastified zone increase indefinitely.

Time has a certain influence on the shape of the pressure-volume curves especially in the plastic phase. The pressuremeter test is carried out according to the method statement and the curve is interpreted using the rules laid down by Ménard.

A certain number (between 7 and 15) of pressure increments are applied to a cylindrical probe placed in a predrilled borehole, or created by the probe itself. Each pressure is held for 1 minute with readings taken at 30 seconds and 1 minute. The technician should choose the increment on visual inspection of the soil.

Even though the pressuremeter has changed in size and shape, a certain number of conditions have always been respected.

1) In order to satisfy the condition of cylindrical expansion, guard cells are required at either end of central measuring cells to reduce end effects. The volume of liquid injected into the probe is measured with each pressure increment. The pressure

applied to the borehole walls affects a relatively important soil mass and localised heterogeneities are smoothed out to give a characteristic $p - v$ curve for that depth. This may be assimilated to the behavior of a full scale foundation in the soil.

2) The requirement of constant pressure steps

Using the 1 minute reading from each pressure step with a correction for the calibration of the probe, a $p - v$ curve may be drawn as follows :

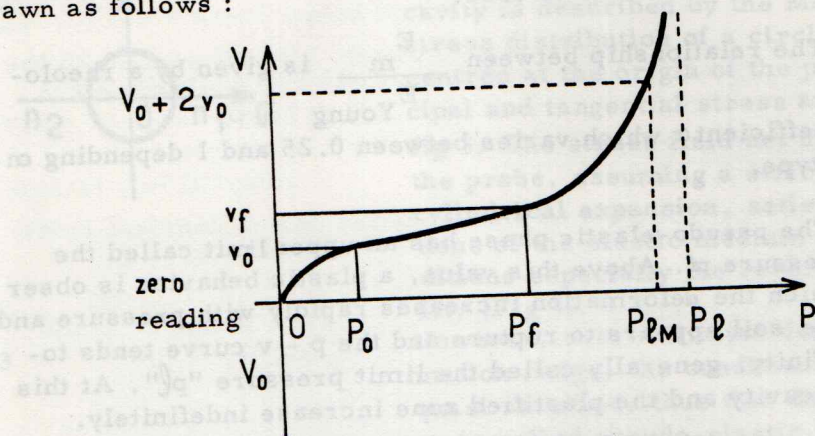


Fig 5 : Typical pressuremeter curve

The theoretical limit pressure may be calculated from the asymptote of the $p-v$ curve, but for convention, the pressure corresponding to a doubling of the initial cavity is used i.e. $2(V_0 + v_0)$ or an injected volume of $V_0 + 2v_0$.

Using the same operating procedure, cyclic tests may be carried out in the pseudo-elastic range. The moduli values obtained can be used to predict the fatigue and vibration effects on the soil (fig 6).

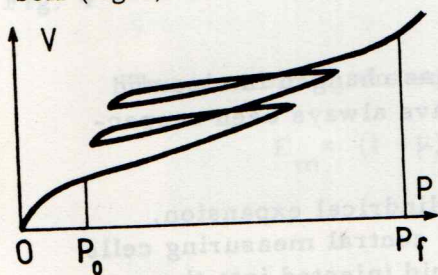


Fig 6

A number of moduli may be interpreted :

- first loading or virgin modulus E_v
- unload modulus E_R
- second or alternate modulus E_A

Using the ratios $\frac{E_R}{E_V}$ and $\frac{E_R}{E_A}$, certain conclusions may

be drawn concerning the degree of densification or fracturation of the soil.

111. EVOLUTION OF THE EQUIPMENT

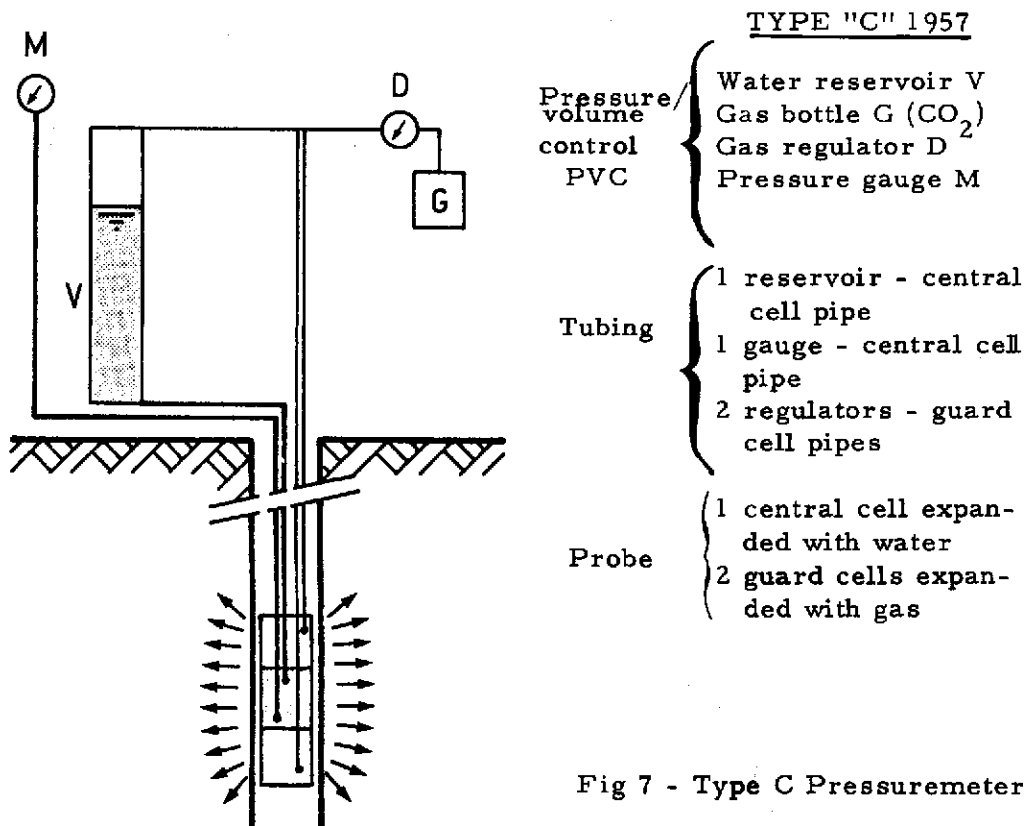


Fig 7 - Type C Pressuremeter

The CO₂ gas from the gas passes through the regulator to the water reservoir which injects a certain volume of water into the central cell. The guard cells are expanded directly by the regulated gas pressure.

The clear plastic reservoir is rated to 1 MPa (10 bars).

Pressure is increased in increments and cyclic testing is possible.

MÉNARD PRESSUREMETER TESTING

TYPE "E" 1961

With the experience of actual site investigations, the type "E" pressuremeter was designed for high pressure and more accurate testing.

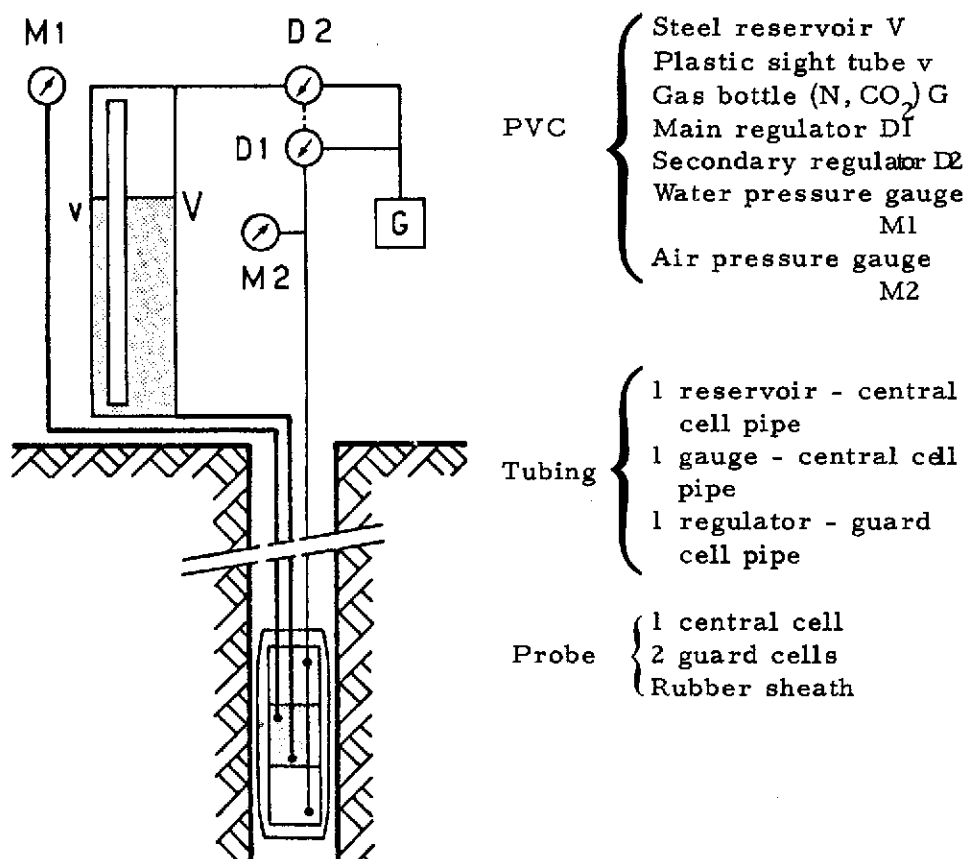


Fig 8 Type "E" Pressuremeter

Two important modifications appear in this model

- the secondary regulator allows automatic compensation for the hydrostatic head at the probe level.
- the steel reservoir and sight tube are directly linked, and allow pressures up to 2.5 MPa (25 bars) to be used.

TYPE "G" 1964

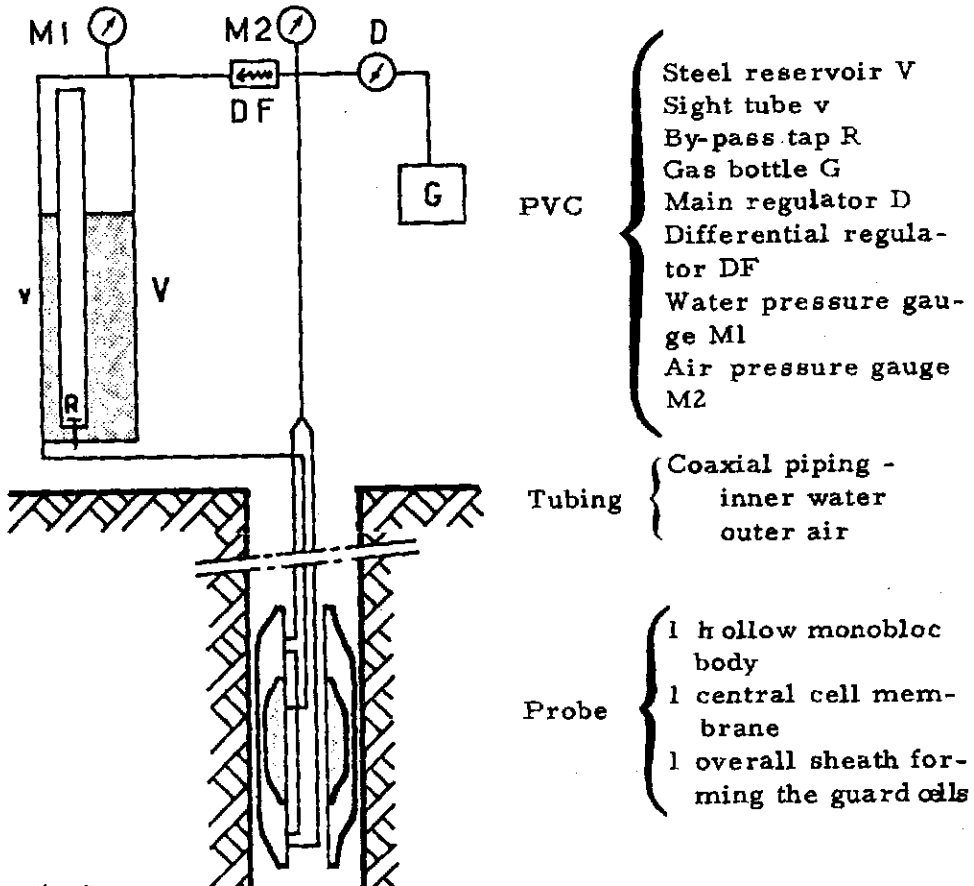


Fig 9

- . the by-pass tap allows a precision $\times 100$ since only the water in the site tube is injected during the test. This is ideal for rock testing where volume changes are very small.
- . the differential regulator compensates for both the hydrostatic pressure and the resistance of the rubber membrane.
- . the coaxial tubing reduces the volume variation due to expansion of plastic under pressure.
- . maximum pressure is now 10 MPa (100 bars)
- . the hollow probe overcomes the piston effect on removal

MÉNARD PRESSUREMETER TESTING

from the borehole.

- . the water in the central cell can be brought back to the surface by leaving pressure in the guard cells, this is especially useful in deep dry boreholes where the head is not externally balanced.

TYPE "G A " 1973

Based on the principle of the type "G" pressuremeter with a number of improvements. The unit is now housed in a protective box to ensure the safeguard of several of the more fragile instruments, notably the gauges.

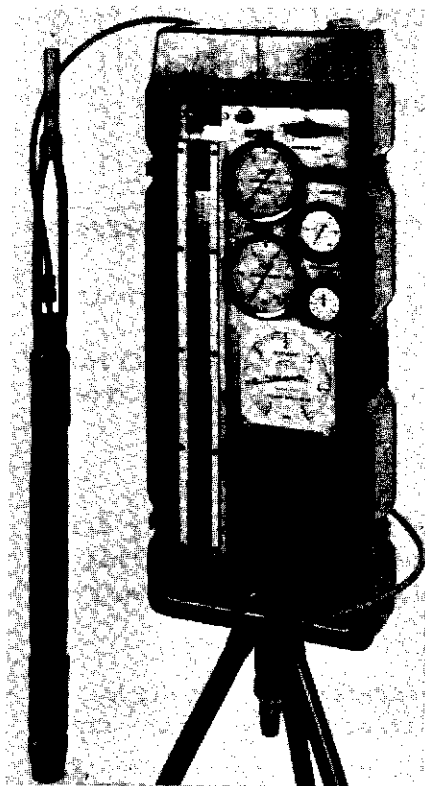


Fig. 10

PVC

- . 0.5 and 2.5 MPa gauges used for greater accuracy at low pressures
- . Zero set prior to testing
- . Jack plug sockets for gauges up to 10 MPa
- . Direct reading of the differential pressure which is fixed throughout the test
- . Simple inversion for the hydrostatic compensation and automatic decompression
- . Easy read volume scale
- . Auxillary reservoir for rapid refilling.

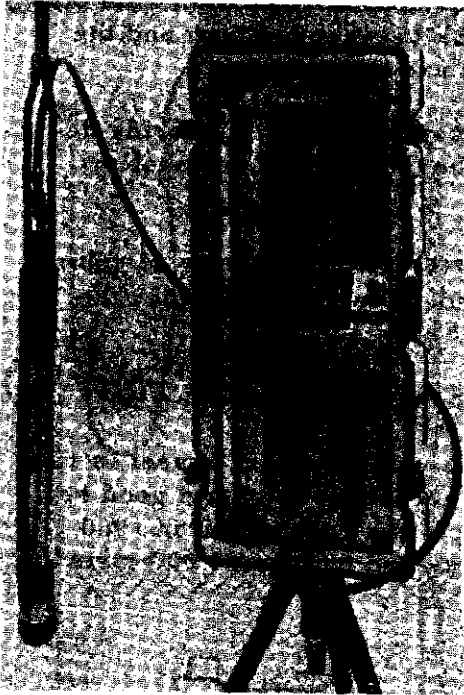
TUBING

- . Non return jack plug on the coaxial.

PROBE

- . Metallic protection of the sheath without significantly increasing its resistance.
- . Reinforcing rings on the sheath to prevent bursting in oversized boreholes. These rings act on the guard cells and have no influence on the central measuring cell.

PRESSUREMETER TYPE "GE" 1981



Advances in computer technology in recent years have been quite remarkable, especially in the field of microchip circuitry. The logical step in pressuremeter design was the automatic interpretation of the test on site with read out of 4 parameters :

Modulus "E"
Limit pressure " p_l "
Contact pressure " p_c "
Creep pressure " p_{cr} "

Fig 11.

Offshore Applications

The execution of pressuremeter tests underwater is no different to the normal procedure. The success depends on the quality of the borehole wall, and as always the drilling method should be modified accordingly. The drilling and testing equipment may be placed on the sea bed and operated by a diver or from a pontoon on surface. The differential pressure between the guard cells and the central cell is calculated from the total depth of soil + water .

The calibration should be carried out with the total length of tubing to be used during the investigation.

MÉNARD PRESSUREMETER TESTING

Unlike other forms of in-situ testing such as penetrometer or plate loading tests no material reaction is required. This solves the problem of having a rigid link between the boat and the test level. A wide range of methods are possible :

- underwater gravity drop system using a triggering device similar to the Kullenberg sampler, for shallow depths. Fig 12 (a).
- using a slotted casing vibrodriven on a string of tubing. The vibro driver is suspended from a winch over the side of a pontoon or boat. Fig 12 (b).
- vibrodriving from a rig placed on the sea bed. Fig 12 (c).

In all of these methods only a small support vessel is required and minimal modification is necessary to obtain good results. The first sea bed investigation was carried out in 1960 and by 1970 the Menard group had carried out over 4 000 tests all over the world.

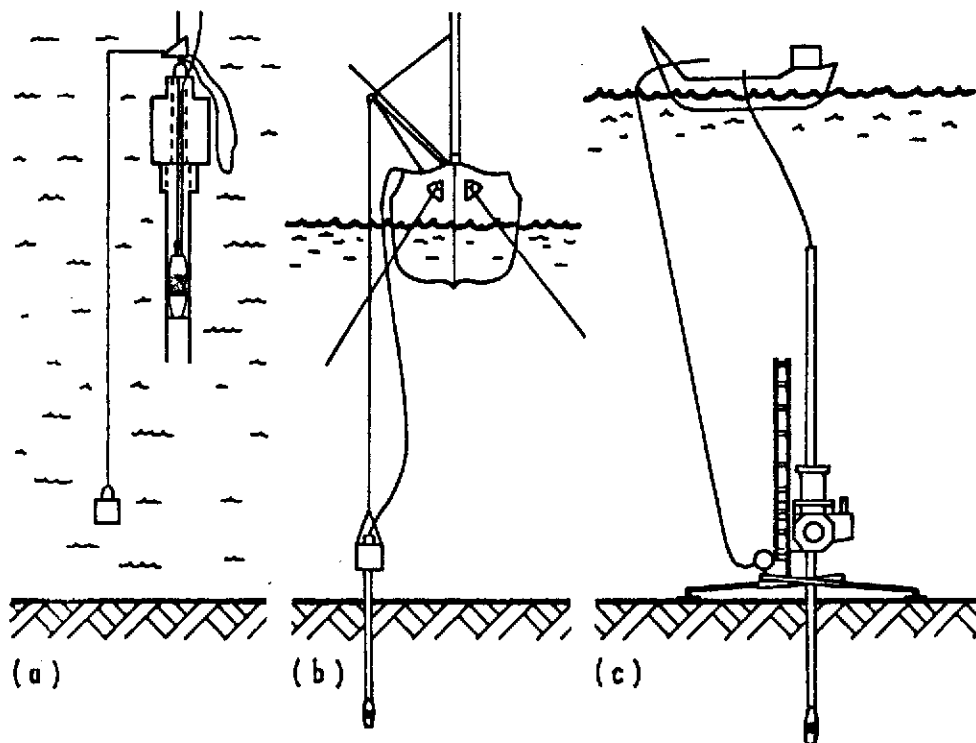
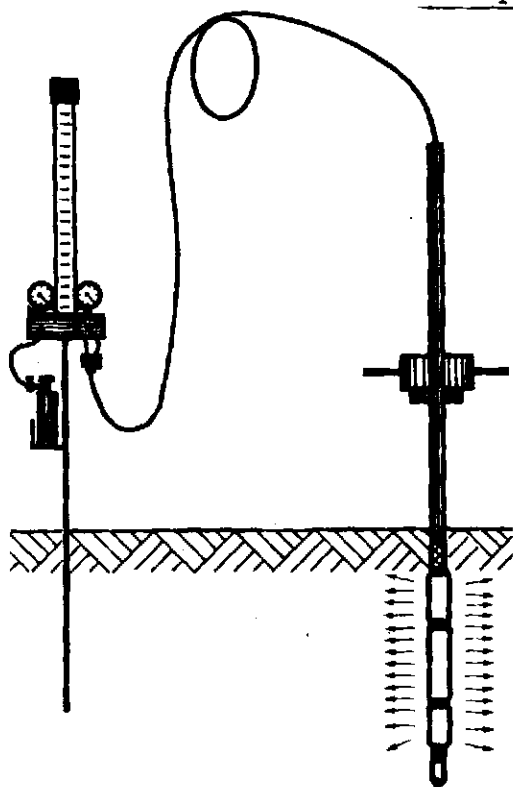


Fig 12.

IV. AUXILIARY EQUIPMENT

Mini pressuremeter 1967



Returning to the original idea of a testing device for compaction control, the Menard group produced the first tricellular probe in 22 mm dia with its own pressure/volume controller. Driving is manual and the testing may be carried out at 30 cms intervals to a depth of 15 m. The test procedure remains unchanged. Applications range from control, excavation bases and sites where machine access is impossible.

Fig. 13

Impulsion pressuremeter 1976

The device was developed for studying problems related to liquefaction and vibration. The principle consists in loading the soil to a particular pressure and sending low frequency impulses in order to measure the reduction in resistance due to instantaneous loads.

High pressure tester 1976

The pressuremeter capable of 60 MPa (600 bars) was designed for rock testing to the rupture level. Here oil is injected into the probe using a pump.

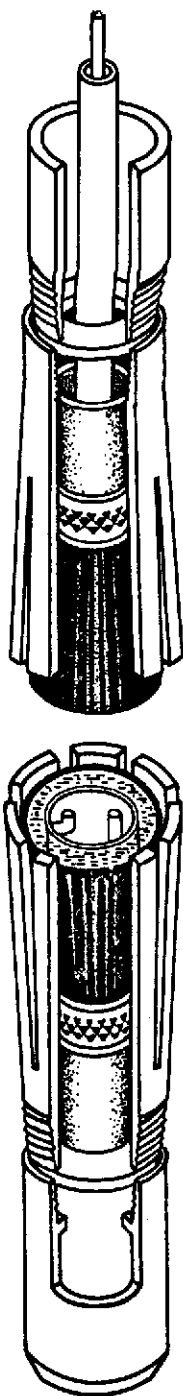
Slotted Casing (1959)

Fig. 14

In a large number of site investigations, problems arise due to the presence of varied soils in which it is difficult to form a satisfactory borehole. Numerous soil types may be cited : sands and gravels, conglomerates, fill materials, glacial deposits, boulder clays etc.. In these circumstances the slotted casing may be driven either directly or in a small diameter predrilled hole with a probe fixed permanently in the middle of the slotted section.

The test is then carried out assuming that the sections of the steel tube are part of the soil in that they smooth out small variations without affecting the test anymore than pebbles in a matrix (global shear test).

The guard cells are indispensable for keeping the measuring cell cylindrical and thus the pressure applied acts through the sections onto the soil.

The only limiting criteria is that the probe dimensions should be proportional to the size of the larger blocks i.e. a small probe will not give a good indication of the soil behavior.

Testing with the slotted casing has been carried out to a depth of 300 m and pressures to 8 MPa which covers the majority of soils encountered in foundation design.

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THE ROLE OF THE PRESSUREMETER IN NORTH AMERICA

ABSTRACT

To date, the pressuremeter has not had that great an impact on foundation engineering in North America. The pressuremeter is perceived to be a specialized tool which is only brought onto a site as a supplement to the standard penetration test or to the recovery of undisturbed samples for laboratory testing. Very rarely is the pressuremeter used as the only testing device.

The unique ability of the pressuremeter to measure in situ a deformation modulus is starting to be more widely appreciated across North America. In general, the pressuremeter modulus is assumed to represent the Young's modulus of a soil. Studies of the anticipated deformation of the soil when acted upon by a foundation are carried out using linear elastic theory. Ménard's empirical design rules for both bearing capacity and settlement remain virtually unknown.

Two or three firms in North America now offer self-boring pressuremeter capability. Unfortunately the geology of North America does not encourage widespread application of self-boring to foundation engineering. On the other hand, pressuremeters are being developed for particular applications such as in oil sands and shales, and in permanently frozen soil.

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INTRODUCTION

The modern pressuremeter was conceived in France in 1956. It was born, however, in the United States of America, and so the pressuremeter has dual 'citizenship'. It grew up and came of age back in France under the guidance and training of its father, Louis Ménard. Now, in its twenty-fifth year, the pressuremeter is starting to make an impact in North America. It is the intent of this paper to look at the way in which the pressuremeter fits into foundation engineering as practised in North America.

In order to understand the reluctance of North American engineers to adopt the pressuremeter, it is important to remember that North America is the home of the split spoon. By and large, North American engineers find the standard penetration test (SPT) method of foundation design satisfactory in granular soils. There is a wealth of experience in carrying out and interpreting the test, and foundation design rules based on the SPT are straightforward and not complicated. Figure 1 illustrates the three step approach to the design of shallow foundations in sand. Step 1: Determine the relative density of the sand from the split spoon blow count, N . Step 2: Obtain the bearing value which will limit settlement of the footing to 25 mm (knowing the load to be carried and adjusting the dimensions of the footing to agree with the stress). Step 3: Determine the factor of safety against ultimate failure of the foundation (using the bearing capacity factors, from the chart, in Terzaghi's bearing capacity equation to compute the ultimate stress, and dividing the ultimate stress by the stress determined in Step 2 to compute the factor of safety.

In cohesive soil admittedly, the split spoon method of foundation design does not work very well. Instead, in clay and clayey silt the unconfined compression test and the consolidation test reign throughout North America. It is in North America, after all, that Casagrande worked on the consolidometer and Terzaghi first lectured on his consolidation theory. The standard North American approach to foundation design in cohesive soil is, first of all, to take good quality samples of the soil in thin wall steel tubes. Secondly, to choose a safe bearing value for spread footings approximately equal to the unconfined compressive strength of the soil, or safe adhesion value for piles of about one-seventh the unconfined compression strength. Thirdly, to estimate the settlement of the foundation unit using elastic theory and consolidation test data.

There has been little incentive over the years for North American engineers to change their approach to foundation engineering. Where soil conditions are good, local building codes limit the types of foundation which can be used and restrict the loads they can carry. Where soil conditions are poor, well established foundation

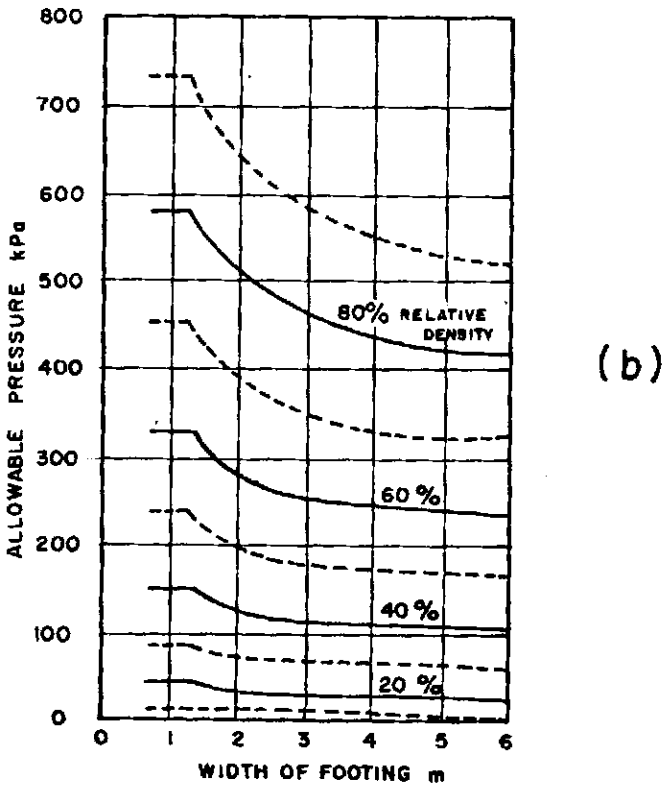
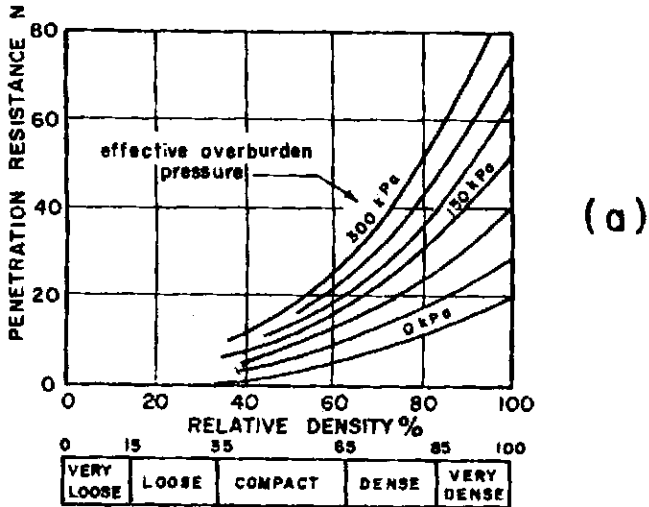


FIGURE 1: PARTS (a) & (b)

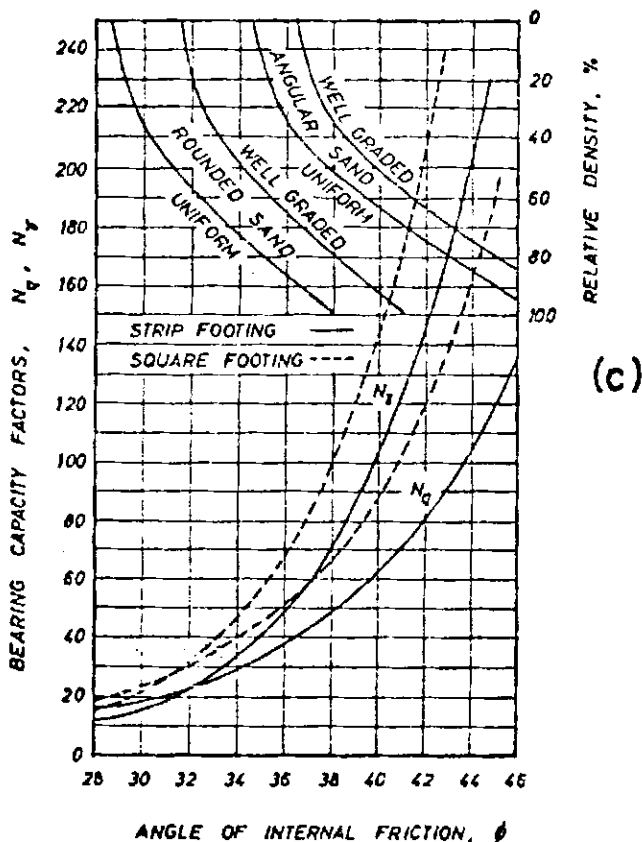


Figure 1: The SPT Method of Footing Analysis

contractors control the market. It is only in unusual soil conditions and/or in the case of non-standard foundation requirements (for nuclear power plants, say, or for offshore petroleum platforms) that more modern tests and more detailed engineering are thought to be warranted.

More modern tests can mean in situ tests such as the static cone penetration test (CPT) or the pressuremeter test. Or, more modern tests can include rapidly cycled triaxial tests for liquefaction studies and other more recently evolved laboratory tests. In the case of deep foundations for buildings and bridges, a large

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number of pile loading tests are carried out throughout North America each year; the instrumentation of the piles, the procedures which are followed to run the test, and the way the test results are interpreted become more sophisticated every year.

For marine foundations other than for harbour structures, the dominant in situ test in North America (as in other parts of the world) is the static cone penetration test. The pressuremeter is almost unheard of as an in situ testing device off the coasts of North America. On land and in harbours, neither the static cone penetration test nor the pressuremeter test is involved in more than a small portion of the tens of thousands of foundation investigations which are carried out each year in North America.

Most consultants, contractors and government agencies in North America view the pressuremeter as a special tool to be used for specialized tests where there is a particular problem to be solved. The unique feature of the pressuremeter is that it measures the modulus of deformation of the soil or rock in which the test is carried out. (Only the plate bearing test, of all the other in situ tests, measures a deformation modulus; the plate test is impractical for routine investigations.) It is this ability to measure deformation properties that makes the pressuremeter stand out in the eyes of North American geotechnical engineers. The sections of this paper which follow, discuss ways in which the pressuremeter is used to solve particular foundation engineering problems.

In all of North America there are probably only five or six firms which use the pressuremeter routinely for foundation investigations to the exclusion of the standard penetration test and laboratory tests on undisturbed samples. One such firm is Geopac Inc. of Montreal which uses the pressuremeter to monitor the efficacy of dynamic compaction it carries out across Canada under license from Ménard of France. Another firm which prefers the pressuremeter is Situ Technology Inc. of Vancouver; this is a small consulting firm with four or five employees which carries out both pressuremeter tests in a pre-bored hole (PBPT) and self-boring pressuremeter tests (SBPT). Soil Testing Services probably is the largest and most widely known firm in the United States carrying out pressuremeter based foundation investigations routinely; this company has more than fifteen years experience with the pressuremeter in the mid-western part of the country.

Predicting Settlement Deformation of Large Storage Tanks

A typical application of the pressuremeter is to measure the deformation properties of foundation soils for settlement predictions. One example of this kind of application is given by

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the study carried out for a proposed tank farm in central Alberta. The 40 to 80 metre diameter tanks will be used to store heavy crude oil at the site of an in situ petroleum recovery plant; the petroleum is contained in a bed of sand located at considerable depth in the ground (at a depth which is too deep to make open pit mining and surface recovery of the oil feasible).

The particular site is underlain by a thick deposit of glacial drift material which is commonly referred to as 'glacial till' or simply as 'till'. Till consists of a dense mixture of clay, silt, sand, gravel and boulders in varying proportions. In general, till is difficult to sample, and most soil specimens which are made available for laboratory testing are disturbed. In an effort to overcome sample disturbance, it is common practice to reconstitute the test specimens by subjecting them to isotropic consolidation. The consolidation pressure is chosen somewhat arbitrarily, but it is quite often chosen to be equal to the estimated total overburden pressure at the depth in the ground from which the specimen came. After consolidation is complete, the specimen is tested undrained in compression in order to determine Young's modulus.

Most investigators obtain both a first loading Young's modulus and a cycled modulus. The first loading Young's modulus is the secant modulus from zero deviator stress to a deviator stress level which is estimated to be one-third the maximum deviator stress, i.e. one-third the strength of the specimen. The cycled Young's modulus is given by the slope of the hysteresis loop on a stress-strain plot; the ends of the loop are defined by zero deviator stress and by a deviator stress which is equivalent to the one-third strength level. The slope of the last hysteresis loop after 3 or 4 cycles of unloading and reloading is the slope which is used to determine the cycled Young's modulus.

Subjecting the test specimens to a consolidation pressure causes them to attain a density which is higher than their density in situ. The higher density probably means that the Young's modulus values are higher than would be measured on truly undisturbed specimens, if such specimens could be obtained. It is universally agreed, however, that consolidation is required because back calculated E values (from settlement measurements) are always higher than E values which are measured on specimens of till which have not been reconstituted. The geotechnical engineer finds himself in this dilemma every time he wants to carry out settlement studies on foundations on till; he knows it is impossible to obtain the 'right' E values from sampling and laboratory testing.

A limited number of case studies shows that the pressuremeter can measure the Young's modulus of tills with reasonable precision. But a difficulty with the case studies, is that the pressuremeter

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tests are carried out and interpreted differently from case to case. For example, the time interval between pressure increments varies from case to case. And, in some cases, volume changes due to creep are subtracted before a Young's modulus is determined.

An additional difficulty when reviewing case records is that there are no Canadian standards for making the borehole in which to carry out pressuremeter tests. Given the presence of coarse gravel and boulders in many tills, the Canadian practice is to employ fairly large and powerful rotary or auger drills on major foundation investigations. Large, powerful equipment means that it is difficult to work with the small diameter tools required for conventional, that is to say Ménard type pressuremeter tests. One or two Canadian firms have developed large pressuremeter probes which, in turn, require purpose-built control units capable of supplying the large volume of liquid which is required to inflate the probe. Large probes offer the advantage of testing a large volume of till and so have a better chance of testing material which has not been disturbed by the hole making process.

At the site of the proposed tank farm in Alberta, both a large probe/control unit pressuremeter and a Standard Ménard pressuremeter (with NX size probe) were used at least initially. The large, 12 cm diameter by 1.0 metre long probe had to be abandoned early in the testing program; because of difficulty with keeping the borehole straight, it was found to be virtually impossible to insert the long probe into holes which were well calibrated but had a number of deviations and jogs in them due to the presence of boulders. The shorter, 0.5 metre long, standard probe was more easily manoeuvred up and down drill holes. It was found that the relative ease with which the Ménard unit could be manipulated and used far outweighed the disadvantage of having to work with small diameter drilling equipment in deep holes. The boreholes had to be deep since the pressure bulb below tanks which were up to 80 metres in diameter will penetrate to considerable depth.

Figure 2 is a plot of the three kinds of Young's modulus values which became available. The standard, first loading pressuremeter modulus, E_M , can be compared with the initial secant modulus, E_s , and the cycled modulus, E_c , values from triaxial tests. The scatter in results is appreciable, but there is a trend for the pressuremeter to give modulus values which are lower than those which are determined by consolidated, undrained triaxial tests. The consulting engineer involved on the project decided to represent the foundation soil by moduli values which were at the upper limit of the lower range of pressuremeter values. He reasoned that, due to disturbance to the walls of the boreholes, the pressuremeter values would generally be too low. Also, he reasoned, the triaxial values

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would be too high since the specimens were consolidated. Choosing the moduli given by the solid line on the figure was thought to be conservative but not unrealistic.

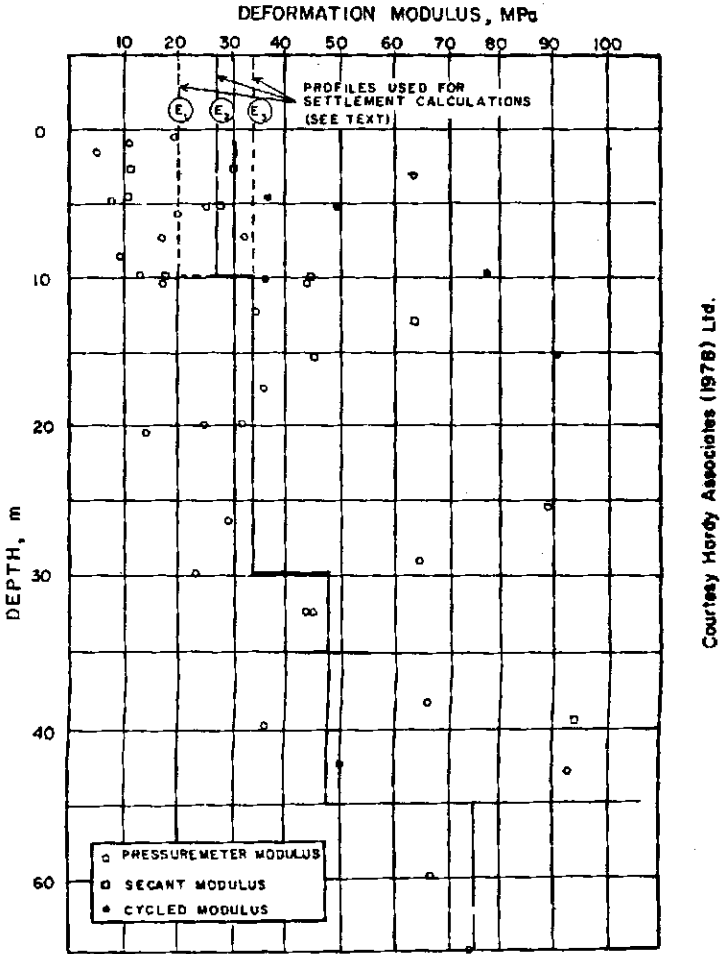


Figure 2: Deformation Modulus versus Depth

Courtesy Hardy Associates (1978) Ltd.

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An axisymmetric, linear elastic, finite element computer program was chosen to represent the design situation. The bottoms of the storage tanks were assumed to be infinitely flexible and the applied stress would remain uniform. A typical result of the settlement calculations is illustrated in Figure 3; here, the settlement profile across a 48 metre diameter tank is plotted for four different heights of tank (or depths of crude oil). The centre-line settlement of the highest tank is calculated to be 100 mm; the settlement of the edge of the tank was found to be 40% of the settlement of the middle of the tank. Based on records of the performance of large structures founded on till (which include the settlement records of a man-made ski hill), it was estimated that the final long term settlement of the tanks would be 20 to 30% greater than those calculated by elastic theory alone. It was also concluded that the elastic portion of the settlement would take place simultaneously with the application of load, i.e. between 70 and 80% of the ultimate settlement would take place immediately; the remaining settlement would occur within a year.

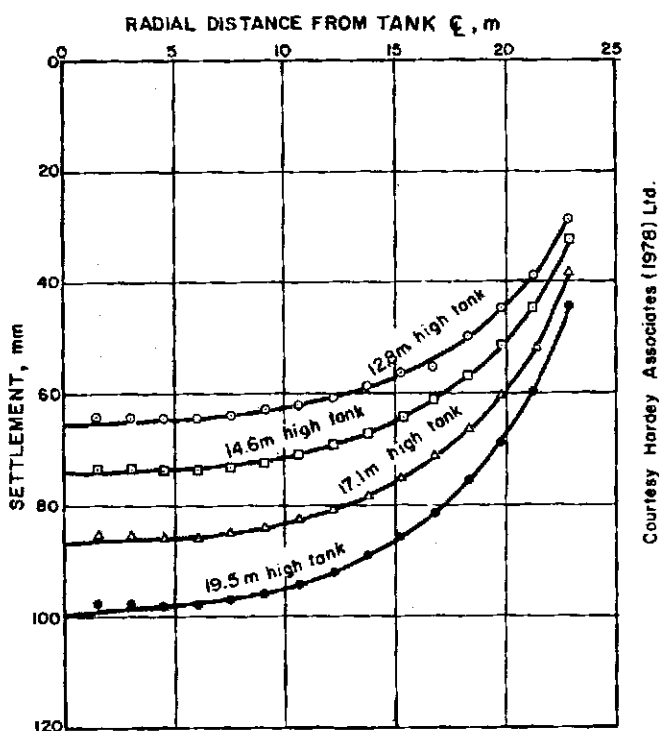


Figure 3: Settlement Across a 48m dia. Tank

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Settlement profiles through the ground were also plotted by the computer for various tank diameters and heights. These profiles pointed out the significant contribution to the total settlement which was made by compression of the upper 10 to 20 metres of soil. Even in the case of the large 80 metre diameter tanks, the upper 20 metres of soil accounted for more than 50% of the settlement (Fig. 4). Because of the scatter in the measured moduli of deformation of the soil, it was decided to investigate the influence of choosing higher and lower E values for the uppermost 9 metres. The range of E values which were investigated is shown on Figure 2. It was found that the smallest diameter tank (the tank which would be most sensitive to changes in shallow soil E values) would settle anywhere from 86 to 107 mm. The consultant concluded that the range of settlement was not significant.

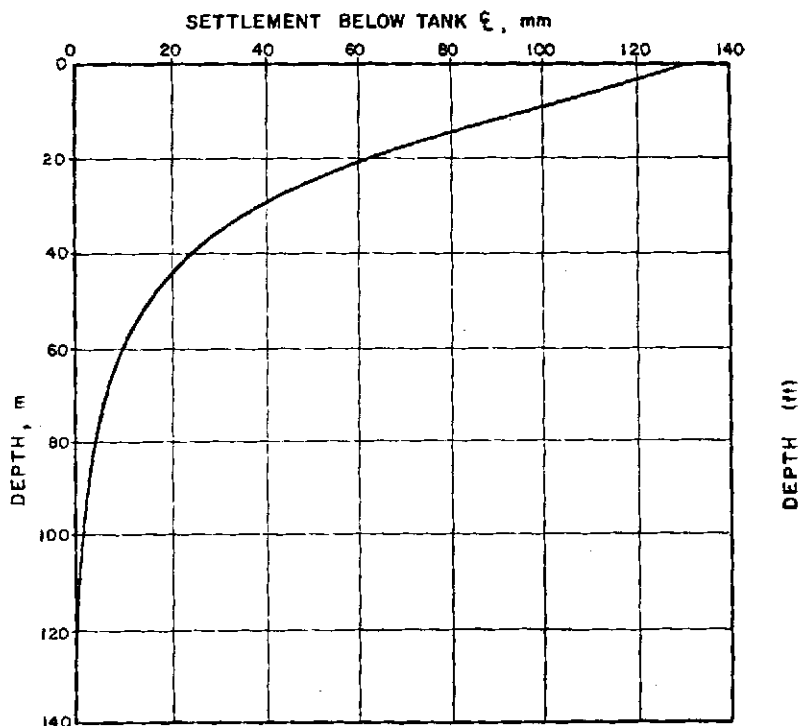


Figure 4: Settlement at Various Depths Below a 80m dia. Tank

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The foregoing example is probably typical of the way in which the pressuremeter is viewed in North America; it is a tool which has particular relevance to settlement studies.

ARTIFICIAL ISLANDS ON PERMAFROST

Much of the proposed production of oil and gas in the Arctic Ocean, particularly in Canadian waters, will be based on artificial islands. Nineteen of the twenty existing petroleum exploration islands in the Beaufort Sea are made entirely of dredged sand fill. In waters which are deeper than 20 metres the all sand artificial island becomes uneconomic because of the large quantity of sand required. So-called hybrid islands (Fig. 5) will be used instead; one hybrid island was built in the Beaufort in the summer of 1981. Structural islands of the type illustrated in Figure 5 are being investigated as possible alternatives to hybrid islands in water depths of 20 to 50 metres. Given the ice conditions in Canada's arctic, the Texas type of drilling platform and the semi-submersible rig are not considered to be suitable.

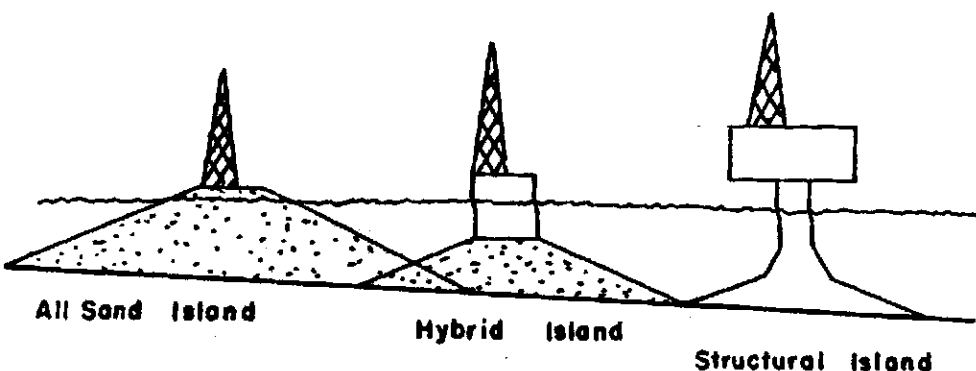


Figure 5: Artificial Islands

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From the geotechnical engineer's point of view, there are many unique and interesting problems associated with artificial islands in the arctic. One such problem is to devise a method for steepening the side slopes of the underwater hydraulic sand fill in order to make the islands more economical (here, the Sandisle in which the sand is contained in a rubber membrane and the side slopes are near vertical is one solution). Another interesting geotechnical problem is to devise ways of counteracting erosion by the arctic ice; most existing islands are surrounded by sacrificial beaches which disappear each winter season and have to be rebuilt.

Of particular interest to this conference, which focuses on the pressuremeter, is the attempt which is being made to understand the deformation of the foundation soil on which the islands rest. Because of the pressuremeter's ability to measure deformation properties in situ, it is starting to be used in the arctic. Pressuremeter tests have been carried out in unfrozen soil at the site of at least one proposed artificial island; the work was performed in the summer of 1981 by Hardy Associates of Calgary, Alberta. Studies of the liquifaction potential during earthquakes of the sand in existing artificial islands were also carried out in the same summer using the pressuremeter; the studies were by Golder Associates of Calgary with Situ Technology Inc. of Vancouver as sub-consultant.

One unique geotechnical problem which is associated with island building is due to the large expanse of permanently frozen soil, or permafrost, which underlies the Arctic Ocean. There is evidence that the permafrost stretches out 150 km or more from shore in the Beaufort Sea and that it is some 600 m thick. The permafrost under the ocean is warm in the sense that it is at a temperature of only minus one or two degrees Celsius. Warm permafrost creeps considerably under most loading situations, and it is for this reason that the permafrost is of considerable concern to island builders. One other concern associated with permafrost which is not discussed further here is the potential for thaw settlement of the islands; warm gases and crude oil moving up wells through the permafrost could provide the heat necessary to thaw lenses or pockets of water which would then seep away from under the structures.

Deformation of the foundations due to creep or thaw could conceivably lead to rupture of the conductor pipes tapping the petroleum or gas. The resulting environmental damage and depletion of a resource could be catastrophic.

It is very difficult to obtain good quality core samples of warm permafrost. Therefore, it is virtually impossible to measure

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the true deformation properties of this material in the laboratory. Both Dome Petroleum of Canada and the University of Manitoba have active research programs under way which seek to adapt the pressuremeter to testing permafrost in situ. Original work in the application of the pressuremeter to permafrost was carried out by Hank Johnston and Branko Ladanyi in northern Manitoba.

Research plans at the University of Manitoba call for comparison tests in permafrost between a standard Ménard hydraulic pressuremeter (manufactured by Roc-test Limited of Montreal) and a gas/electric pressuremeter (manufactured by the OYO Corporation of Tokyo). Preliminary tests point up the difficulty of using a hydraulic pressuremeter in freezing temperatures; the viscosity of the water/ethylene glycol mixture means that considerable time is required for fluid to flow from the control unit to the probe. The gas inflated Japanese unit does not suffer from the time lag problem, but it does introduce problems associated with maintaining the accuracy of electronic measuring devices in cold weather in the field.

There is no standard procedure for measuring the deformation properties of frozen soil. Since plastic flow, or creep, under constant stress is expected to account for the major portion of the deformation settlement of artificial islands founded on permafrost, it is important that the pressuremeter test measure in some way the creep potential of the soil. Once a more or less standard test for permafrost is devised for the pressuremeter, it will be possible to relate pressuremeter test results to the results of other tests on frozen soil. For example, facilities exist at the University of Manitoba in which pressuremeter and plate bearing tests can be run in beds of frozen soil; also, triaxial equipment is available to test prepared specimens of frozen soil in compression. Both short and long term pressuremeter, plate and triaxial tests are envisaged.

Ladanyi and Johnston tried to extrapolate the results of short term pressuremeter tests over a much longer time period. They assumed that permafrost would behave in creep much as do high temperature metals or ice. They also assumed that their pressuremeter tests measured only immediate and primary creep strains. Both slightly modified conventional pressuremeter tests, in which each load increment was left constant for 15 minutes, and special tests, where a given probe pressure was maintained for up to 6 hours, were run. (It will be recalled that in classical creep theory a strain versus time plot will exhibit both primary, i.e. non-linear, behaviour and secondary, linear behaviour after a certain amount of instantaneous deformation takes place on the application of a load (Fig. 6).)

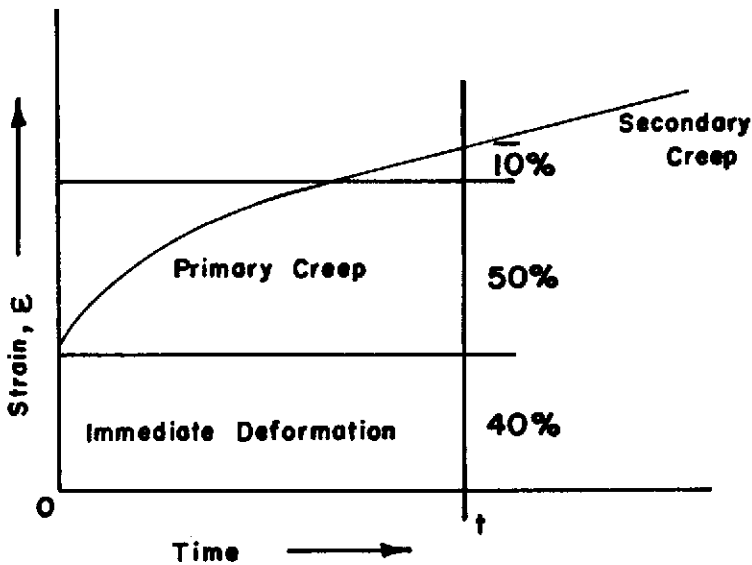


Figure 6 : Creep Strain versus Time

There is certainly engineering interest in being able to predict primary creep. For example at time t on the hypothetical deformation curve of Figure 6, both the immediate and the primary creep components would represent about 90% of the total deformation whereas the immediate settlement alone would account for only about 40%. The basic equation which Ladanyi and Johnston proposed in order to extrapolate pressuremeter creep curves is:

$$\Sigma = K\sigma^a t^b$$

where Σ = creep strain

σ = applied stress

t = time

a, b, K = material constants which
are temperature dependent

They developed this basic equation into a specific equation for the

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pressuremeter, and described the procedure to follow in order to determine the creep constants from actual pressuremeter test results.

One test of the appropriateness of the Ladanyi-Johnston creep equation for permafrost can be made by plotting the volume change of the probe against time on log-log paper. The Ladanyi-Johnston theory requires that the plot be a straight line. While admitting that very few of their pressuremeter results plotted as straight lines, Ladanyi and Johnston concluded, nevertheless, that the method was "feasible". The ultimate 'test' would be to see how well the theory fits actual curves from long term pressuremeter tests; such pressuremeter tests would have to last upwards of a year, whereas the longest pressuremeter tests to date have lasted only a few hours at most.

Just how pressuremeter derived creep information will be used to solve the problem of artificial islands on permafrost has not been determined. One possibility would be to use creep theory to draw up a family of pressuremeter curves for various times in the future, using actual short term pressuremeter test results for the initial curve (Fig. 7). If, say, the working life of an island

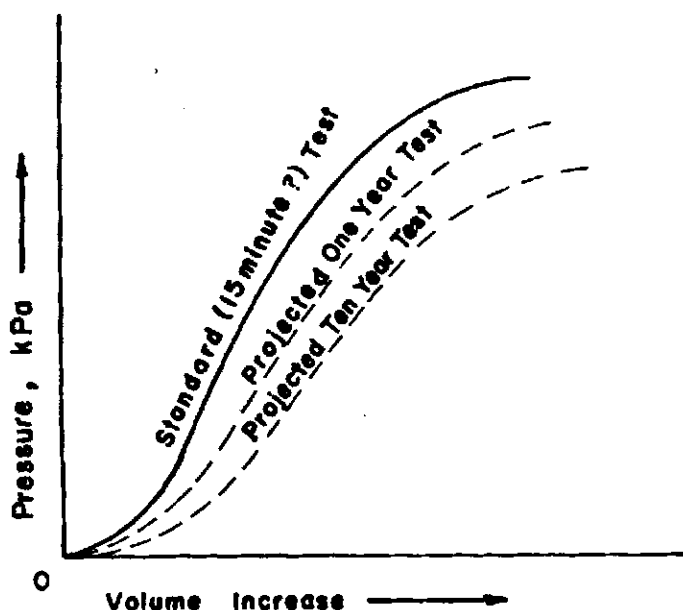


Figure 7: Real and Projected Pressuremeter Curves

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were to be 25 years, the appropriate shear modulus could be calculated from the curves in order to compute the settlement of the island at any particular time in its working life.

A second possible approach to solving the artificial island settlement problem might be to calculate the foundation deformation from regular, short term pressuremeter test results (as was done for the Alberta tank farm earlier). A suitable permafrost creep law could then be applied to the deformation predictions themselves in order to project the movements into the future.

WIRELIN DRILLING AND THE PRESSUREMETER

Since the theme of this conference is the pressuremeter and its marine applications, it seems appropriate to say a few words about how the pressuremeter has been or will be adapted to offshore drilling around the coasts of North America.

In particular it has been found necessary to adapt the pressuremeter to wireline drilling. To date, however, tests have been run in relatively shallow holes, and problems associated with remote pressuremeter testing in deep holes have not been tackled.

The difficulty with wireline drilling is that the drill stem stays down the hole (see Figure 8 (a)). Therefore it is necessary to drop the pressuremeter down through the stem. If the drill stem is simply pulled back to expose a length of hole for pressuremeter testing (b), the pressuremeter can not expand sufficiently to both fill the annular space vacated by the drill stem and test the soil. Hardy Associates of Calgary devised one way of overcoming this problem by making a pilot hole ahead of the regular drill hole with a sampler. Drilling was stopped just above the depth at which a pressuremeter test was required, and the drill stem was pulled up a distance equal to the length of the sampler (c). The sampler was dropped down through the stem and latched into position. A sample of the soil was recovered in the usual way by pushing the sampler into the ground with the drill stem (d), unlatching the sampler and withdrawing it from the hole. It was then a simple matter to withdraw the drill stem the required distance, latch the pressuremeter in place and push it into position in the pilot hole for the test (e).

Whether or not the sampler procedure would work in permafrost remains to be seen, but it is doubtful that it would work in all frozen soils. To facilitate pressuremeter tests in soils where the sampler will not work, another means of making a pilot hole is being devised. In this case either a small tricone bit or a special

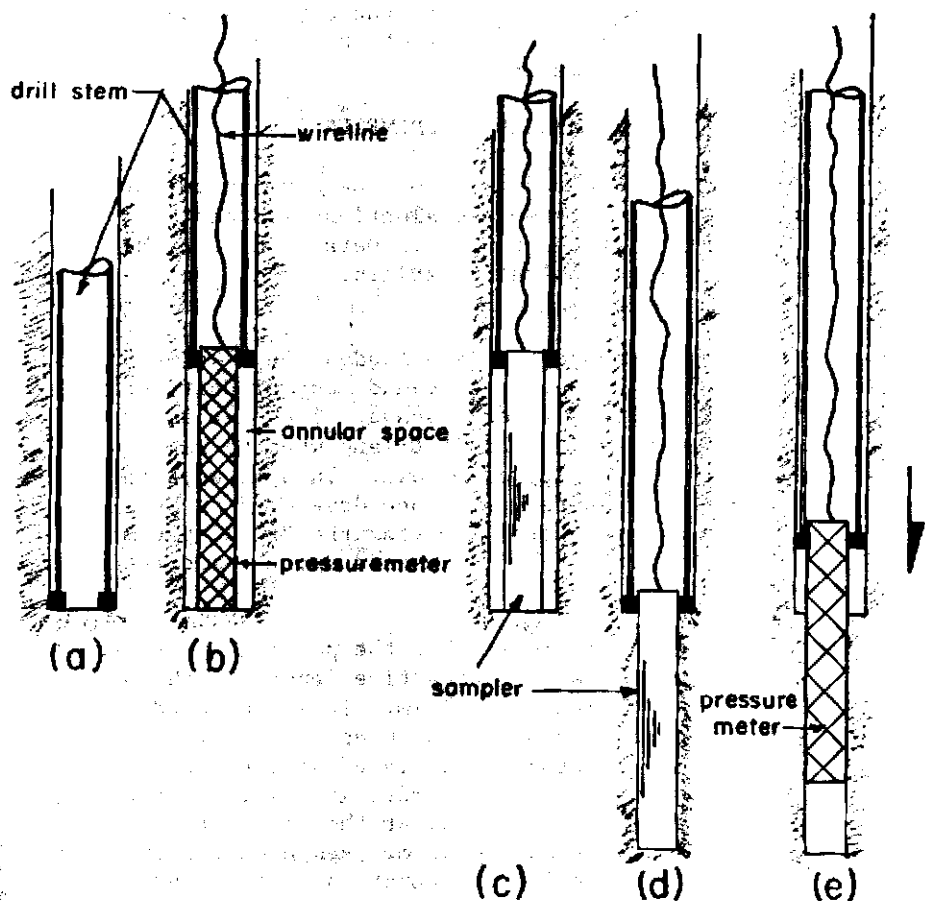


Figure 8: The Wireline Pressuremeter

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core barrel will be latched onto the bottom of the drill stem after it has been withdrawn the required distance. The pilot hole will then be made by rotating the drill stem in the usual way and using mud flush to remove the cuttings. Trials of these and other ideas on land are planned for next summer.

THE PAVEMENT PRESSUREMETER

Only brief mention will be made here of the development of a pressuremeter and a test for pavement evaluations and design. The pressuremeter and the test are described in detail in an ASTM publication; the use of the results is explained in a Transportation Research Board publication.

About six years ago, Transport Canada funded the development of a short, monocellular pressuremeter probe and control unit for use at airports in remote locations across Canada. The idea for the pressuremeter originated with Jean Louis Briaud when he was a research student at the University of Ottawa. In some ways the probe resembles the mini-pressuremeter probe developed by Ménard for compaction control. The control unit for the Briaud pavement pressuremeter differs entirely from the Ménard device, and the test procedure is original.

Figure 9 represents, diagrammatically, the pavement pressuremeter in use on an airport runway. The active length of the probe is 230 mm and its outside diameter is 31 mm. It is inflated hydraulically by means of a hand pump; the pump itself is the familiar triaxial test unit. The probe is inflated at a standard rate during the test, and the pressure is recorded at regular intervals. Part way through the test, that is to say at the end of the straight line portion of the pressure-volume change curve, the probe is deflated and re-inflated once. No attempt is made to carry out the test to the limit pressure.

The hole into which the pressuremeter probe is inserted is made by drilling through any existing asphalt or concrete surface and then by driving a rod to a depth of two metres. Standard EX drill rods fitted with a drive point are used; the rods have an outside diameter of 32.5 mm. Pressuremeter tests are run at intervals of 0.3 m in the hole. The 2 m depth of testing ensures that the pavement base course and the subbase are tested over a depth corresponding to the pressure bulb below a airplane wheel.

The cycled pressuremeter modulus is calculated for each test. The most effective way of using the test results is to assume that each modulus value is representative of a 0.3 m thick layer of soil. Multi-layer elastic theory can then be used to estimate the maximum horizontal tensile strain which would occur at the lower surface of

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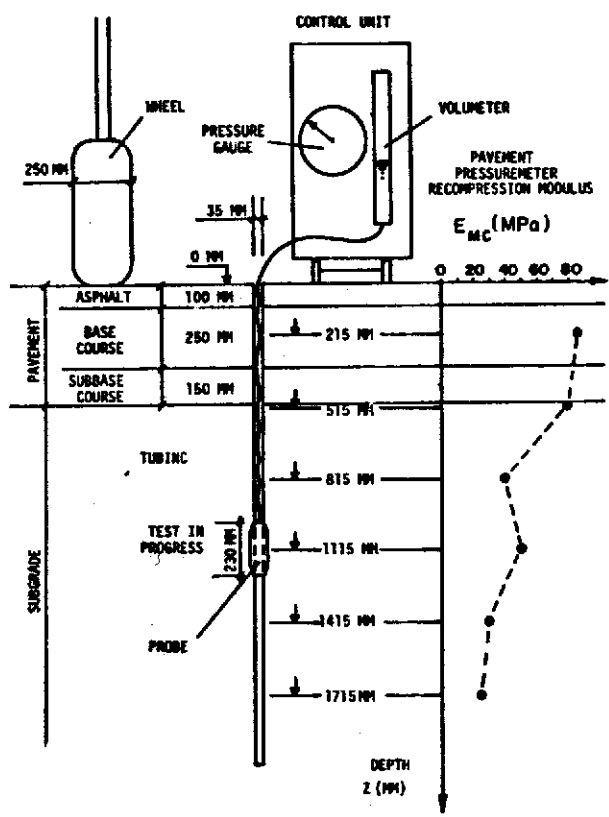


Figure 9. Pavement Pressuremeter

the asphalt under a given wheel load. The theory can also be used to calculate the maximum horizontal compressive strain at the top of the subgrade and the settlement of the surface of the pavement. Limiting strain criteria can be established for both the asphalt and the subgrade; if the calculated strains exceed the allowable limits, the service life of the pavement will not be up to standard.

A second way of utilizing the results of pavement pressuremeter tests is as a substitute for plate bearing test results. The more traditional method of designing and evaluating airport pavements is by means of plate bearing tests; in Canada the test is known as the McLeod Plate Test. The Canadian standard test uses a 450 mm plate and a reaction of many tens of tonnes is required to load the plate and deform the pavement. Charts are available which relate

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plate bearing values and the thickness of a pavement to the aircraft rating for the runway. The profiles of six pressuremeter test results can be converted to equivalent plate bearing values; in this way the very cumbersome and expensive plate test can be replaced by a convenient and inexpensive series of pressuremeter tests.

Whether or not multi-layer elastic theory is used or the equivalent plate bearing value is determined, immediate use can be made of the pressuremeter test profiles. By visually scanning the profiles of modulus values, an engineer can see at a glance if weak layers exist within the pavement structure or in the subgrade. For example, blocked underdrains could cause water to pond on the surface of the subgrade and the subgrade to soften; a low pressuremeter modulus value at the top of the subgrade would pin-point the problem immediately.

OTHER ROLES FOR THE PRESSUREMETER

A limited survey of pressuremeter users confirmed that the pressuremeter was being used in a number of more conventional ways in addition to those described in the preceding sections. Self-boring pressuremeters are being used to test the river delta deposits of the lower British Columbia mainland and the mud of San Francisco Bay. The horizontal capacity of piles is sometimes based on either self-boring pressuremeter test results or Ménard type pressuremeter test results. Even in Winnipeg, Manitoba, pressuremeters are used to estimate in situ horizontal stresses for tunnel design and to prove that cast-in-place piles can carry a load which far exceeds the local building code allowance.

Nevertheless, in North America the pressuremeter has a long way to go in order to displace the SPT, the CPT, and sampling and laboratory testing, on a day to day basis. For special roles, such as measuring the in situ stress conditions and deformation properties of oil sand deposits, the pressuremeter is already the predominant test.

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Mr. Don Diakiw drew the figures; both are colleagues at the University of Manitoba. The Natural Sciences and Engineering Research Council and the University of Manitoba paid the expenses involved in contacting and interviewing engineers for this paper.

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DEVELOPMENT OF LLT PRESSUREMETER AND ITS APPLICATION IN PREDICTION OF PILE BEHAVIOR UNDER HORIZONTAL LOAD

ABSTRACT

The most commonly used borehole loading method today is the pressuremeter method, which uses a rubber tube to apply a uniform load. There are a variety of devices using this principle in use.

This paper concerns apparatus, testing method, accuracy, etc. of the lateral load tester (LLT), a monocell pressuremeter developed by the authors that is the most widely used pressuremeter device in Japan, and estimation method of pile behavior under horizontal load based on the LLT test results.

Behavior of foundation pile when it receives horizontal load is generally treated as that of bending beam on elastic bed. In this treatment very important is to know the horizontal reaction coefficient of ground, that is, K-value. In linear elastic body K-value is constant. In actual ground, however, K-value is not constant and decreases with increase of displacement of pile due to non-linearity of stress-strain relationship of soil. The authors developed a new method which allows pursuit of pile behavior through whole process of loading. To be exact, the method enables us to determine K_i for optional displacement y_i and to determine the horizontal load H_i for y_i from a series of linear solution curves when $K = K_i$. Then, we can draw the behavior curve by connecting all plots on graph. The author has studied how to determine K-value for optional displacement according to many results of horizontal load testing of pile. This method was also developed successfully in combination with the results of LLT Test in borehole. Based on the results of LLT Test to be conducted in borehole for preliminary survey, we are now able to forecast the behavior of foundation pile under horizontal load with sufficient accuracy.

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PREDICTION OF PILE BEHAVIOR BY LLT PRESSUREMETER

I. Introduction

Probably the first device using the pressuremeter method in a borehole was the "seitendruckapparat", developed by Kögler in 1933. In the 1950's, research on the improvement of the accuracy of device on the basis of Kögler's idea was carried out in France and Japan. It is well known that in France Menard's efforts resulted in the Menard's pressuremeter. In Japan, during the early 1950's, Fukuoka developed the "Public Works Research Institute Type K-value tester" for the purpose of carrying out research on lateral K-value as it relates to horizontal resistance of foundation piles.

In 1966, the authors developed the lateral load tester (LLT), a monocell type device, on the basis of Fukuoka's concept.

The development of the pressuremeter in Japan, as explained above, comes from the necessity of assessing the stability of foundation piles in relation to horizontal loads. In a country like Japan, where earthquakes occur frequently, it is necessary to take adequate precautions to insure stability of structures against horizontal force produced during earthquakes. This is a very important factor in the design of the foundations of structures. Chang (1937) has dealt with the bending of beams on elastic beds. Broms et al. (1965) proposed a compound subgrade reaction method that takes plastic region into account. The authors, too, has carried out many experiments related with this problem since the development of the lateral load tester (LLT). The results of these experiments led to the proposal in 1969 of a relatively simple design method for deduction of the entire process of behavior of piles under lateral loads on the basis of LLT measurements taken in boreholes.

II. The LLT and Its Use

Figure 1 shows a schematic view of the LLT system. Pressurized N_2 gas from a gas cylinder passes through a regulator into a pressure volume meter. This pressure volume meter is a cylindrical water tank; it is joined to the probe by nylon tubing. Gas pressure forces water into the cell, inflating it. The amount of water sent to the cell is measured by reading of the water level in a stand pipe attached to the front of the pressure volume meter. The LLT has two pressure gauges; one for reading gas pressure within the pressure volume meter, and the other for reading actual pressure within the cell.

The probe has an overall length of 90 cm, with a diameter of 80 mm. The length of the cell is 60 cm. The cell is covered by a double rubber tube. Varying thicknesses of rubber tube may be used

PREDICTION OF PILE BEHAVIOR BY LLT PRESSUREMETER

in accordance with the ground being measured. For the most recent LLT model, the LLT-M, there is a choice of 3 probes, of 60, 70 and 80 mm outer diameters, respectively. Accordingly, various combinations of four tanks are also available, providing different capacities and degrees of accuracy.

As in other loading tests, an appropriate loading pattern in accordance with the objective may be selected with the LLT. However, the standard procedure is by stress control, increasing a constant load at 2 minute intervals. Deformation measurement is carried out at intervals of 15 sec, 30 sec, 1 min and 2 min for each loading stage.

Arrangement of measurement data is carried out in the following way: First, effective pressure of the ground P_e is read off from the pressure gauge indicating water pressure within the cell. Then, restitution of the rubber itself, which changes as the rubber tube expands, and hydrostatic pressure, which depends on measuring depth and borehole water level are corrected. In order to find borehole radius in relation to pressure P_e , the final amount of deformation of the cell at each loading stage is found by noting the expansion of the cell from the water level in the stand pipe after 2 minutes. The stand pipe reading is used to calculate the amount of deformation of the cell, that is, the final outer diameter of the cell. In addition, the progressive amounts of deformation ΔH at each loading stage for intervals of from 30 sec to 2 min are determined as creep index values.

Next, these values, pressure P_e , borehole radius r and amount of creep ΔH are used to draw P_e - r and P_e - ΔH curves. Figure 2 shows a typical illustration of these curves. At (1) the cell is set at the point for measurement; from (1) to (2) the cell expands freely; at (2) the cell reaches the borehole wall; from (2) to (3) is the recompression stage, pushing the borehole wall back against active earth pressure; at (3) a balance is reached earth pressure at rest; (3) to (4) the region where the curve becomes a straight line is interpreted as a quasi-elastic region in which passive earth pressure is produced; (4) is the point of yielding; (4) to (5) is the process of failure; and (5) is the point of failure, and the end of the test.

From the P_e - r and P_e - ΔH curves, earth pressure at the rest P_0 , yield pressure P_y and failure pressure P_f are determined. In addition, K value, the subgrade coefficient of the ground is calculated as the tangent of the slope of the P_e - r curve at (3) to (4), i.e., $k = \Delta p / \Delta r$.

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III Accuracy of LLT Test Results

Next, let us turn to the accuracy of results obtained by the LLT, such as K values. The reliability and accuracy of measurement results should be evaluated from the following 4 viewpoints: (1) the theory on which the design of the measuring device and measuring method is based; (2) the degree of accuracy of the process by which the final values are calculated on the basis of the measurements; (3) the effects of other factors such as disturbance of the borehole by drilling; and (4) human error due to limitations of ability or experience of the operator. Each of these factors are important. In number (3), problems arising from disturbance of the borehole wall are now being dealt with by the self-boring pressuremeter. Here, we will limit our analysis to an examination of number (2), error involved in the process of measurement.

There are a great number of factors that can lead to error in results, from the taking of measurements with the LLT to the final calculations. Figure 3 shows the various potential sources of error at each stage in the process of deriving analysis values from LLT measurements. In the first stage, measurement values (cell water pressure, tank water level, etc.) are obtained; in the second stage, correction values (rubber restitution, hydrostatic pressure, etc.) and in the third stage, assumed values (dimensions of loading surface, Poisson's ratio) are specified. These values we designate as fundamental values. Next, the plotted values (P_e , r and ΔH), as the function of the fundamental values are obtained. Furthermore, we obtain analysis value (K , E , P_o , P_y , etc.) as the function of the plotted value. The potential for error exists at each stage: Basic error comes from measuring error, correction error and error in assumptions. Plotting error comes from combinations of these basic errors. Finally the accumulated plotting errors determine each analysis value error.

Considering all the above types of error, we may suppose that the overall accuracy of LLT measurements of K value would be within two significant digits. Measurements using other types of pressuremeter methods may be considered to have about the same accuracy.

IV Factors Affecting Pressuremeter Test Results

There are many factors which may be thought to influence on the results of pressuremeter testing. The authors have conducted numerous tests both in situ and laboratory, in every type of soil

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to find the effects of these factors have on pressuremeter test results. Following 5 factors have been studied and the degree of influence of each factor for representing pressuremeter parameters is summarized in Table I:

1. Drilling method (state of disturbance of the borehole wall)
2. Structure of pressuremeter probe (monocell or tricell type)
3. Size of pressuremeter probe
4. Loading method (stress control or strain control)
5. Loading rate

Although evaluation of research is still in progress and final values are not yet available, Table I shows the trends that have been noted. In preparing this table, test results were divided into 4 classifications representing the relative degree that each factor influenced pressuremeter test results. The classification "large" and "very large" represent factors that cannot be ignored in evaluating pressuremeter test results. Factors rated "small" and "very small" may be ignored.

Table I: Factors Affecting Pressuremeter Test Results

Pressuremeter Parameter Factor	Earth pressure at rest P_0	Yield pressure P_y	Failure pressure P_l	K-value and Elasticity modulus
Drilling method	Very large	Large	Large	Very large
Structure of probe	Very small	Very small	Small	Very small
Size of probe	Very small	Very small	Very small	Very small
Loading method	Very small	Very small	Large	Very small
Loading rate	Small	Small	Large	Small

The factor exerting the greatest influence on pressuremeter test results is drilling method, which in effect means the state of disturbance of the borehole wall. We may conclude that whether or not satisfactory test results are obtained depends on whether or not a careful drilling procedure has been carried out to prepare test borehole. In comparison to this factor, the structure or size of probe used has very little influence on test results. It is safe to say that these factors are negligible in calculating K-value elasticity modulus for such applications as pile design.

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V. Understanding on Pile Behavior Under Horizontal Load and Basic Idea to the Forecasting

Figure 4 shows that the relationship between the displacement and the load of the pile when it is subjected to horizontal loads is a curve. Most of the data from the loading tests confirm the validity of the relationship mentioned above, and signify that the ground subgrade reaction coefficient K defined by Chang is not constant. Consequently, the application of Chang's concept of bending beams on linear elastic bodies to pile behavior problem under horizontal load is not entirely satisfactory for assessing. In consideration of this, the forecast method proposed here determines K -value at any displacement (design K -value) on the basis of K -value at a displacement of 1 cm (K_0 -value). Then the load required to effect this displacement is calculated. By the continued determination of this relationship the behavior of the pile during the whole loading process is determined. As shown in Figure 5, K_i value for optional displacement y_i value are decided. Then, from Chang's behavior curves for $K=K_i$, load H_i for each y_i value is obtained. Plotting the points of coordinate (H_i, y_i) , we obtain the pile behavior curve.

VI. Definition and General Expression of K -Value

The following general formula may be used to describe the behavior of piles under horizontal loading as shown in Figure 6.

$$EI \frac{d^4 y}{dx^4} + P = 0 \quad (1)$$

In this formula, subgrade reaction P is expressed by the following function:

$$P = \alpha \cdot B^l \cdot x^m \cdot y^n \quad (2)$$

Where

B = width of pile, x = depth and y = displacement.

Here, the functional expressions for K -value were apparent K -values in the non-linear model of Formula (2), substituted in for Chang's linear model, i.e., $P = K \cdot B \cdot Y$.
Thus,

$$K \cdot B \cdot y = \alpha \cdot B^l \cdot x^m \cdot y^n \quad (3)$$

Consequently,

$$K = \alpha \cdot B^{\ell-1} \cdot x^m \cdot y^{n-1} \quad (4)$$

Here, the value of K where unit width of pile $B = 1$ cm and unit displacement $y = 1$ cm, we designate as specific K -value k_0 , and define it: $k_0 = \alpha \cdot x^m$. This specific K -value corresponds to equivalent K -value, which is determined solely by the type of soil including the influence of depth (Figure 7 illustrates equivalent K -value, the value for K giving equivalent displacement values of pile top). By determining the index value ℓ and n in Formula (4) by horizontal load test data (Figures 8 and 9), the final expressions for subgrade reaction P and K -value thus become:

$$P = k_0 \cdot B^{\frac{3}{4}} \cdot y^{\frac{1}{2}} \quad (5)$$

$$K = k_0 \cdot B^{-\frac{1}{4}} \cdot y^{-\frac{1}{2}} \quad (6)$$

Consequently, if the specific K -values k_0 of a given site can be predetermined, subgrade reaction and apparent K -values for a pile with width B and displacement y can be determined from Formulas (5) and (6).

VII. Calculation of Pile Behavior Under Horizontal Load using LLT Test Results

K -value of the ground may be expressed as a function of pile width B and displacement y as determined by Formula (6). Using the values for B_1 , y_1 and K_1 from a pile in the ground, supporting K_2 for another pile of B_2 and y_2 in the same ground may be determined by the following formulas, based on Formula (6):

$$K_1 = k_0 \cdot B_1^{-\frac{1}{4}} \cdot y_1^{-\frac{1}{2}} \quad (7)$$

$$K_2 = k_0 \cdot B_2^{-\frac{1}{4}} \cdot y_2^{-\frac{1}{2}} \quad (8)$$

From the above, K_2 may thus be determined:

$$K_2 = \left(\frac{B_2}{B_1} \right)^{-\frac{1}{4}} \cdot \left(\frac{y_2}{y_1} \right)^{-\frac{1}{2}} \cdot K_1 \quad (9)$$

Next, if we ignore for the moment the differences between the load-in mechanism of the pile and of the LLT apparatus, applying LLT

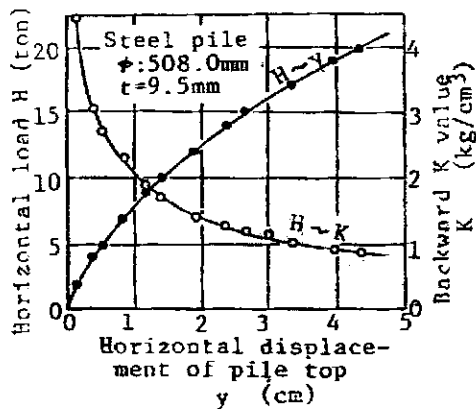


Figure 4 An Example of Horizontal Load Test Result of Pile

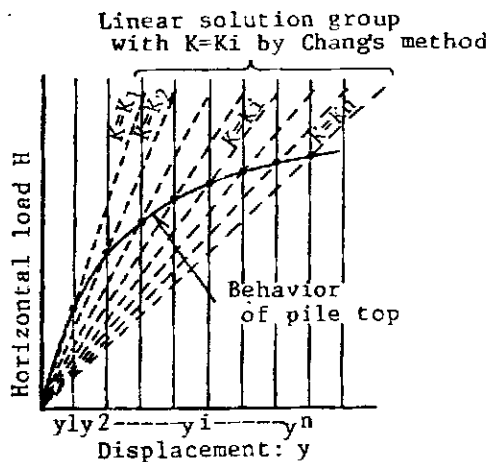


Figure 5 Assumption of Pile Behavior

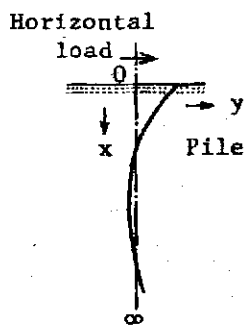


Figure 6 Pile Under Horizontal Load

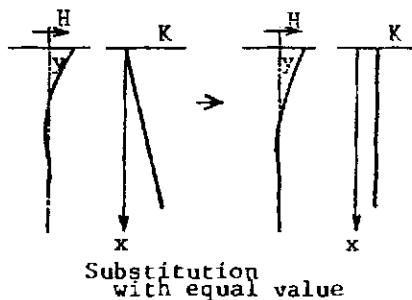


Figure 7 Idea of Equivalent K-Value

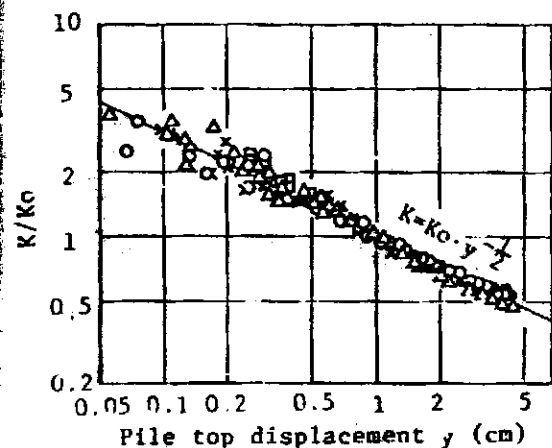


Figure 8 Relationships between K/K_o and y

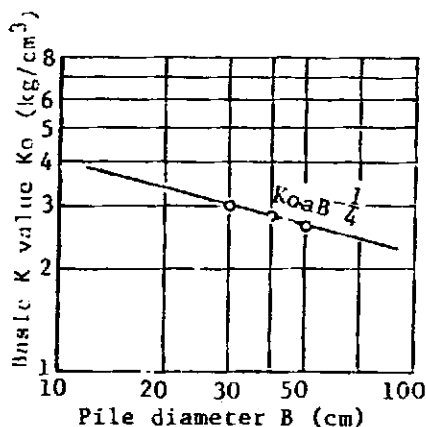


Figure 9 Relationships between K_o and B

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data as known piles, unknown pile K-values may be determined by Formula (9). In this formula, if

$$\begin{aligned} B_1 &= 2r_o && (r_o: \text{initial radius in LLT measurement}) \\ y_1 &= r_m - r_o && (r_m: \text{radius at midpoint of range determining measured K-value}) \\ K_1 &= K_m && (K_m: \text{measured K-value; see Figure 10}) \\ B_2 &= B && (B: \text{Pile width in centimeters}) \\ y_2 &= 1 \text{ cm} && (\text{pile displacement is taken in units of 1 cm}), \end{aligned}$$

$$K_o = \sqrt[4]{\frac{2r_o \cdot (r_m - r_o)^2}{B}} \cdot K_m \quad (10)$$

This means that by using LLT measurements (r_o , r_m and K_m), basic K-value K_o for 1 cm of displacement can be determined for a pile of a given width. In the same way, design K-value K when displacement is y , specific K-value of the ground is k_o and pile width is B may be determined by the formulas:

$$k_o = \sqrt[4]{\frac{2r_o \cdot (r_m - r_o)^2}{B}} \cdot K_m \quad (11)$$

$$K = \frac{K_o}{\sqrt{y}} = \frac{k_o}{\sqrt[4]{B \cdot \sqrt{y}}} \quad (12)$$

Still, we see from Figure 11 that there is a essential difference between the loading pattern of piles and the LLT. Therefore, it is necessary to pre-estimate the effect on K-value of this difference and correct for it. The value of this correction coefficient for soft ground in general has been determined as $\frac{1}{2}$ from a variety of experimental calculations and comparative experiments.

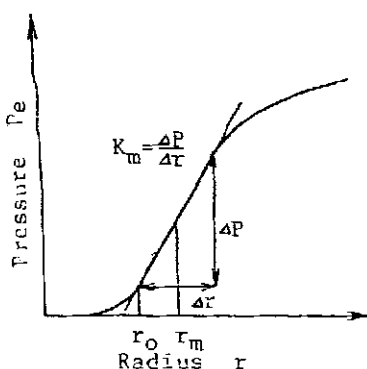


Figure 10 LLT Measurement Results Schematically Drawn

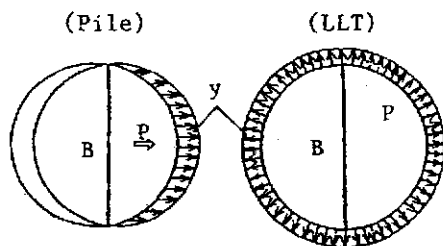


Figure 11 Comparison of Load Styles of Pile and LLT

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VIII Calculation of Equivalent Ko-Value

The K-value required by a given set of LLT measurements for a pile is calculated by the following formulas:

$$\text{specific K-value } k_o = \frac{\pi}{2} \sqrt[4]{2r_o (r_m - r_o)^2} \cdot K_m \quad (13)$$

$$\text{basic K-value } K_o = \frac{\pi}{2} \sqrt[4]{\frac{2r_o (r_m - r_o)^2}{B}} \cdot K_m \quad (14)$$

$$\text{design K-value } K = \frac{\pi}{2} \sqrt[4]{\frac{2r_o (r_m - r_o)^2}{B y^2}} \cdot K_m \quad (15)$$

Under actual conditions, K-value from LLT measurements is not uniform with depth. It can be seen from Figure 7 that in dealing with the behavior of piles under horizontal loads, it is convenient, if possible, to average the changing K-values to obtain the equivalent Ko-value, representative of the ground. Accordingly, the author has considered the problem of how to calculate equivalent Ko-value from LLT measurements. Figure 12 shows the basic approaches to this problem. In this figure, method ① takes into account the thickness of the layer by taking a weighted average. Method 2 considers that the main effect of subgrade reaction on lateral resistance of pile is toward the top of the pile, while effects fall off with increasing depth. Depths of approximately 1.5 times that of bending moment 0 has been taken as an appropriate one for which equivalent ko-values are to be calculated. Figure 13 shows examples of calculations using methods ① and ②, respectively. The figure shows the results from horizontal loading tests of a pile and also K-values calculated on the basis of these results using Chang's method. Figure 14 shows the relationship between equivalent ko-value, calculated by methods ① and ② and basic Ko-value, obtained by calculation using Chang's method. From these results, the calculation of equivalent Ko-value on the basis of LLT measurements may be obtained by taking the average Ko-values, weighted for layer thickness. Figure 15 shows the relationship between equivalent Ko-value, calculated according to method ① from LLT test results and basic Ko-value obtained by pile loading tests. The data for this figure consists of 27 measurements from 12 investigation sites in which horizontal loading tests of real piles and the LLT tests are being carried out. The piles used in testing have diameter of 4.3 cm to 3 m, the former being a steel model pile and the latter being cast-in-place concrete piles. In all, there are 10 types of pile. There is a good correlation between equivalent Ko-values obtained by LLT measurement and basic Ko-values obtained by loading tests.

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IX Calculation of Pile Behavior Under Horizontal Load using LLT Test Results

Figure 16 represents the arrangement of the above data to perform a series of calculations for application of LLT measurement results to actually pile design. By this method, pile behavior under horizontal load may be deduced by knowing the specifications of the pile (E, I and B) and by using LLT measurements. Table I shows an example of an actual calculation using the method. Figures 17 and 18 compare actual and calculated pile behavior for a model pile and a steel pile respectively.

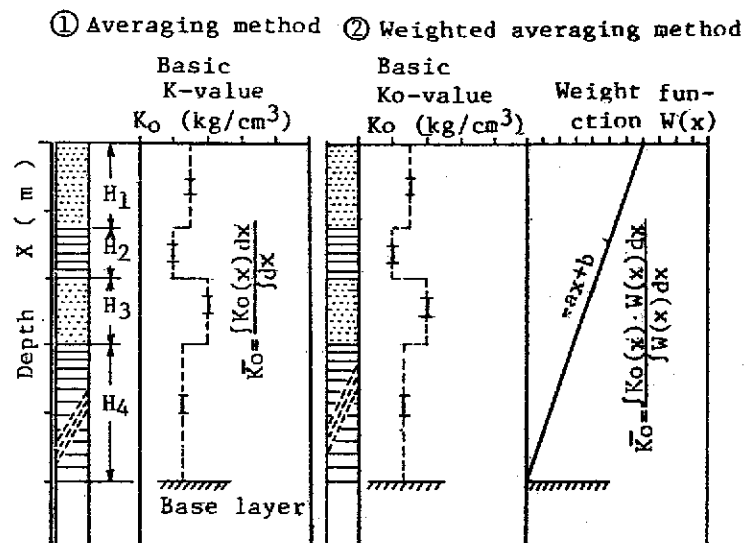


Figure 12 Basic Ideas of Calculating Equivalent K_o -Value

PREDICTION OF PILE BEHAVIOR BY LLT PRESSUREMETER

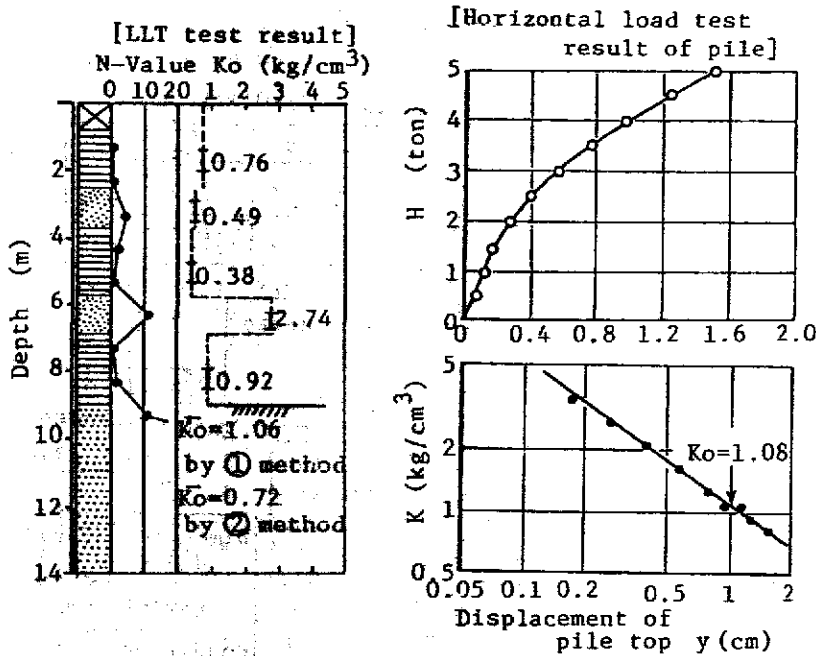


Figure 13 Another Example of Comparison

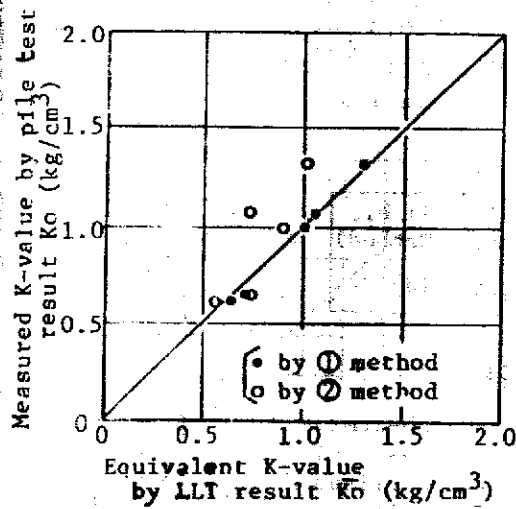


Figure 14 Comparison of K-Values by LLT and Pile Load Test

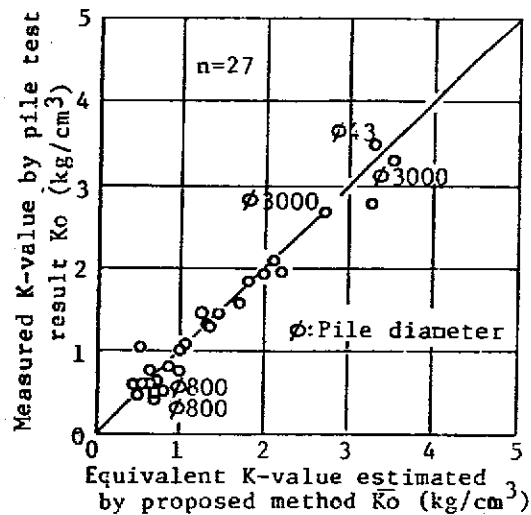


Figure 15 Comparison between Measured and Estimated K-Value

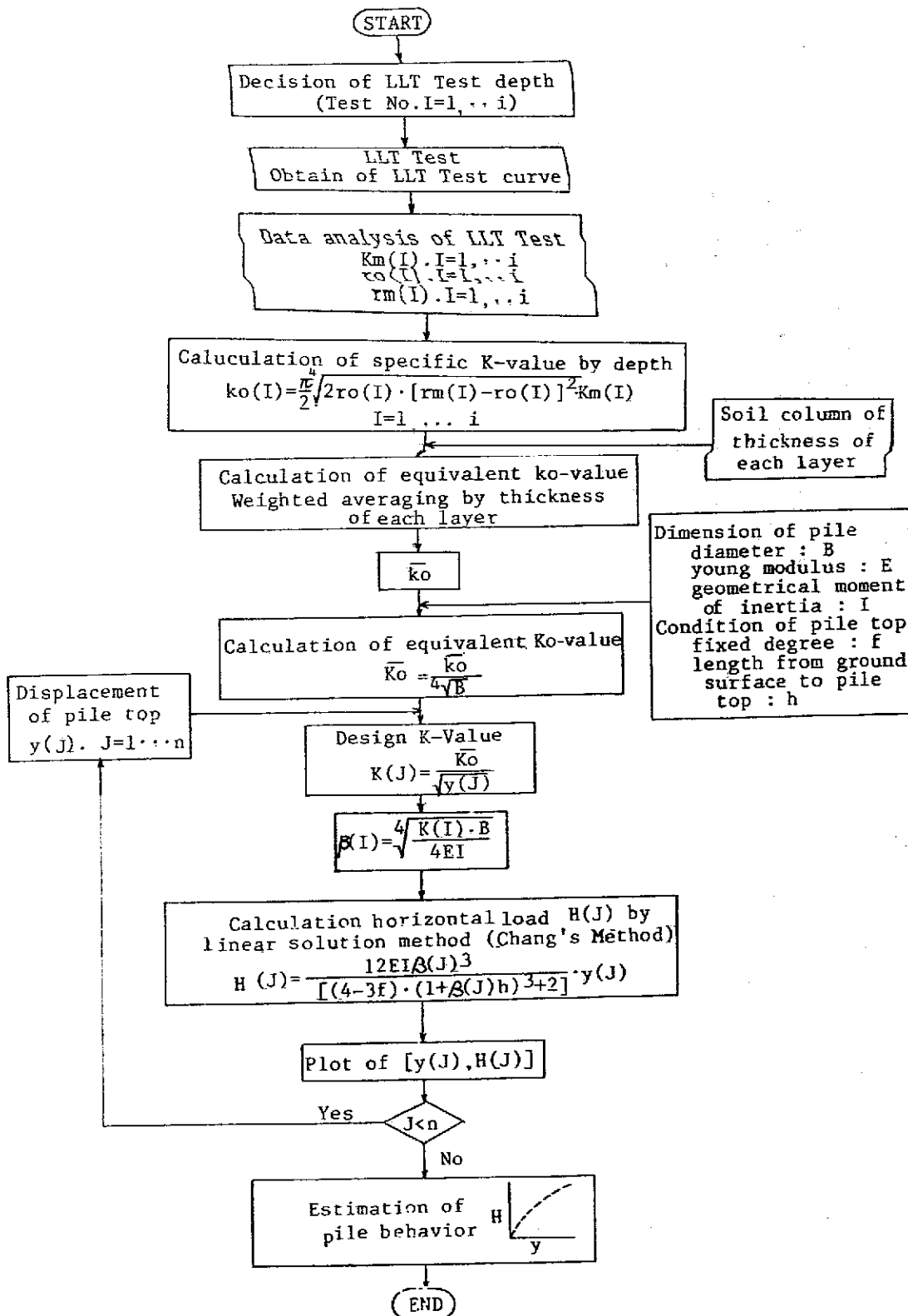


Figure 16. Calculation Procedure for Evaluation of Pile Top Under Horizontal Loads

Table II Calculation Chart of Pile Behavior

Kind of pile; ϕ : 43 mm t : 3mm steel pile
 $E=2.1 \times 10^6$ (kg/cm²) $I=7.58$ (cm⁴) $B=4.3$ (cm)
 Pressure applying height : $h=0.34$ (cm)
 Basic K-value by LLT $K_0=2.64$ (kg/cm³)
 $\frac{3EI}{h^3} = 0.21 \times 10^3$ (kg/cm) $\frac{B}{4EI} = 26.76 \times 10^8$ (1/kg·cm)

④	⑤	⑥	⑦	⑧	⑨	⑩	⑪
y	$\sqrt[3]{y}$	K_A	β^4	β	βh	$F(\beta h)$	H(kg)
(cm)	Con- version	$K_0 \times ⑤$	$2 \times ⑥$	$4 \sqrt{⑦}$	$⑩ \times ⑧$	*	$⑩ \times ⑪$
0.01	10.0	26.4	178×10^{-8}	3.66×10^{-2}	1.24	6.15	1.97
0.02	7.07	18.7	126 "	3.35 "	1.14	6.95	3.48
0.03	5.77	15.2	103 "	3.18 "	1.08	7.55	4.81
0.04	5.00	13.2	89.2 "	3.07 "	1.04	7.98	6.07
0.05	4.47	11.8	79.8 "	2.99 "	1.02	8.23	7.35
0.06	4.08	10.8	73.0 "	2.92 "	0.993	8.58	8.46
0.07	3.78	9.98	67.5 "	2.87 "	0.976	8.83	9.59
0.08	3.54	9.34	63.1 "	2.82 "	0.959	9.08	10.7
0.09	3.33	8.80	59.5 "	2.78 "	0.945	9.30	11.7
0.10	3.16	8.35	56.4 "	2.74 "	0.932	9.50	12.7
0.11	3.02	7.96	53.8 "	2.71 "	0.921	9.70	13.7
0.12	2.89	7.62	51.5 "	2.68 "	0.911	9.88	14.7
0.13	2.77	7.32	49.5 "	2.65 "	0.901	10.08	15.6
0.14	2.67	7.06	47.7 "	2.63 "	0.894	10.20	16.6
0.15	2.58	6.82	46.1 "	2.61 "	0.887	10.35	17.5
0.16	2.50	6.60	44.6 "	2.58 "	0.877	10.55	18.4
0.17	2.43	6.40	43.3 "	2.56 "	0.870	10.70	19.2
0.18	2.36	6.22	42.0 "	2.55 "	0.867	10.76	20.2
0.19	2.29	6.06	41.0 "	2.53 "	0.860	10.93	21.0
0.20	2.24	5.90	39.9 "	2.51 "	0.853	11.07	21.9

$$* F(\beta h) = \left\{ (1 + \beta h)^3 + \frac{1}{2} \right\} / (\beta h)^3$$

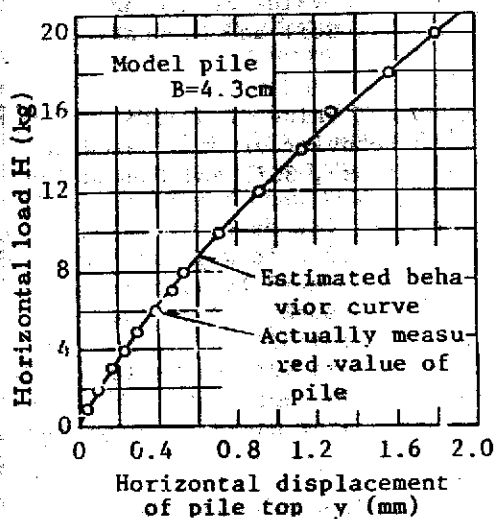


Figure 17 Comparison of Actual Measurement Value Estimated Behavior Curve

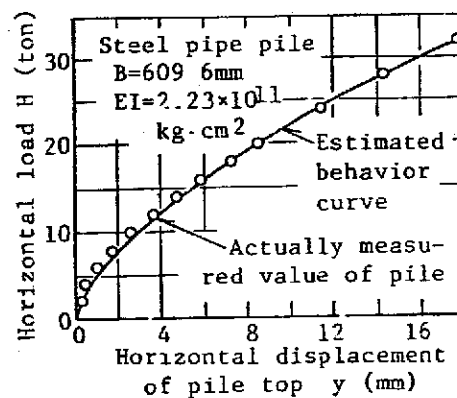


Figure 18 Comparison of Actual Measurement Value Estimated Behavior Curve

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data as known piles, unknown pile K-values may be determined by Formula (9). In this formula, if

$$\begin{aligned} B_1 &= 2r_0 && (r_0: \text{initial radius in LLT measurement}) \\ y_1 &= r_m - r_0 && (r_m: \text{radius at midpoint of range determining measured K-value}) \\ K_1 &= K_m && (K_m: \text{measured K-value; see Figure 10}) \\ B_2 &= B && (B : \text{Pile width in centimeters}) \\ y_2 &= 1 \text{ cm} && (\text{pile displacement is taken in units of 1 cm}), \end{aligned}$$

$$K_0 = \sqrt[4]{\frac{2r_0 \cdot (r_m - r_0)^2}{B}} \cdot K_m \quad (10)$$

This means that by using LLT measurements (r_0 , r_m and K_m), basic K-value K_0 for 1 cm of displacement can be determined for a pile of a given width. In the same way, design K-value K when displacement is y , specific K-value of the ground is k_0 and pile width is B may be determined by the formulas:

$$k_0 = \sqrt[4]{\frac{2r_0 \cdot (r_m - r_0)^2}{B}} \cdot K_m \quad (11)$$

$$K = \frac{K_0}{\sqrt{y}} = \frac{k_0}{\sqrt[4]{B \cdot \sqrt{y}}} \quad (12)$$

Still, we see from Figure 11 that there is a essential difference between the loading pattern of piles and the LLT. Therefore, it is necessary to pre-estimate the effect on K-value of this difference and correct for it. The value of this correction coefficient for soft ground in general has been determined as $\frac{\pi}{2}$ from a variety of experimental calculations and comparative experiments.

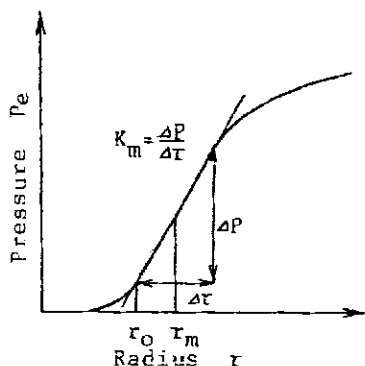


Figure 10 LLT Measurement Results Schematically Drawn

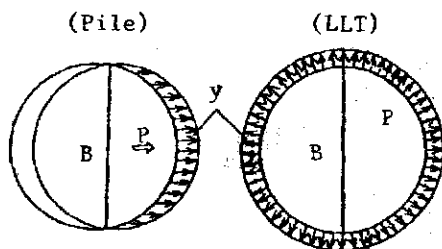


Figure 11. Comparison of Load Styles of Pile and LLT

PRESSUREMETER TEST FOR SOFT COHESIVE SOIL

INTRODUCTION

The results obtained from Menard pressuremeter tests conducted in Japan over 15 years have been applied to various problems in foundation engineering. However, it was pointed out that the modulus of deformation obtained from pressuremeter tests was only one third of the modulus back-calculated from laterally loaded piles (Yoshida and Yoshinaka, 1972). Shields and Bauer (1975) concluded that only the plate loading tests gave moduli comparable to those back-calculated from prototype footings, and that moduli obtained from the pressuremeter tests were $1/2$ to $1/3$ of the back-calculated moduli.

Among possible reasons responsible for such discrepancies, the influence of soil disturbance around a hole pre-bored and left unsupported is considered serious. In order to minimize disturbance of soil, self-boring pressuremeters were developed by Hughes (1973) and Baguelin (1972). Although the self-boring pressuremeter test is thought to be an improved testing technique, it has been reported that the undrained strength obtained from pressuremeter tests is considerably larger than that derived from vane shear tests (Amar et al., 1975). Clough and Denby (1980) obtained a little larger undrained strength of soft clay from self-boring pressuremeter tests than that derived from conventional triaxial compression tests and vane shear tests. However the undrained strength from the triaxial tests for reconsolidated specimens was equal to or slightly above the results from the pressuremeter tests. Windle and Wroth (1977) found that the values of undrained strength of stiff clays from pressuremeter tests were either in good agreement to or exceeding

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those derived from triaxial tests and cone tests. The values of undrained Young's modulus derived from pressuremeter tests agreed closely with those from large diameter plate tests. Denby and Clough (1980) pointed out that soil disturbance caused by improper insertion of a probe resulted in higher values of undrained strength obtained from the pressuremeter test.

This paper presents the results of tests obtained from instrument devised by the author for soft cohesive soil along the northern coast of Tokyo Bay. The consistency of parameters obtained from the self-boring pressuremeter was examined comparing the results of tests in two to three closely spaced boreholes. The parameters obtained from the self-boring pressuremeter tests were compared with those derived from the Menard pressuremeter.

SELF-BORING INSTRUMENT AND TEST-PROCEDURE

The outline of two different types of the self-boring pressuremeter (abbreviated as SBP hereafter) is schematically illustrated in Figure 1 and Figure 2. The type A shown in Figure 1 is the instrument named G-type probe of Menard pressuremeter (Technique Louis Menard, 1975) containing a self-boring mechanism. An instrument of 85 mm in diameter with the main cell of 210 mm long and the guard cell of 440 mm long is statically penetrated into ground by a vertical force transmitted through a pipe of 65 mm in diameter. Two piezometers are installed between the lower end of the probe and the top of the cutting shoe to measure pore-pressure built up during penetration. Further details of the probe of type A are described in the author's paper published elsewhere (Mori, 1981). The instrument type C shown in Figure 2 is same as type A in its self-boring mechanism and geometric dimensions. However, it only contains the main cell of 440 mm long.

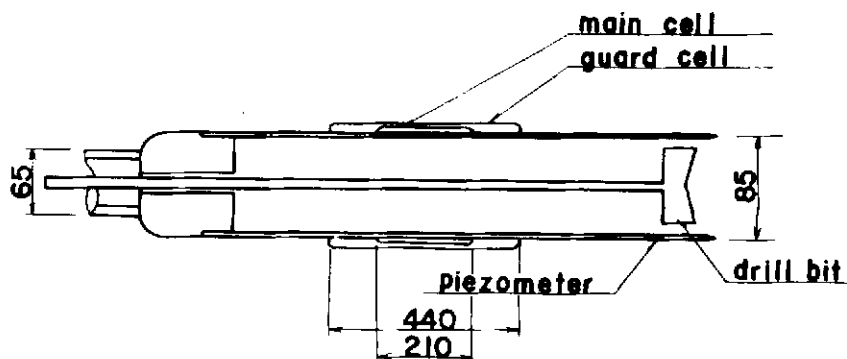


Fig.1 SBP Probe Type A

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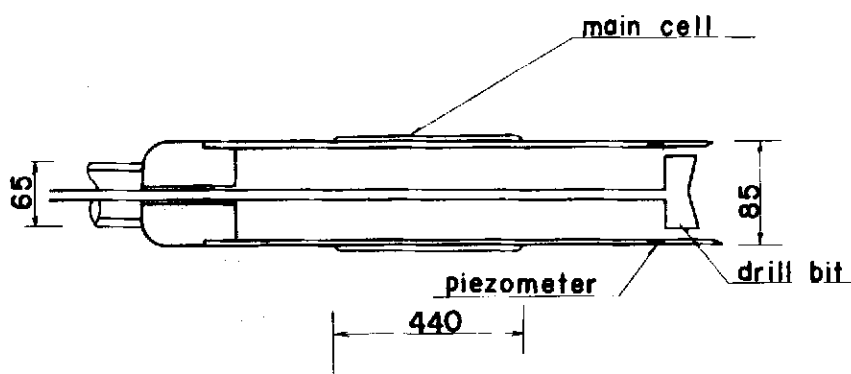


Fig.2 SBP Probe Type C

When the SBP instrument is penetrated to a desired depth, pore pressure is measured by two piezometers until excess pore pressure becomes less than 5 kPa. The effective horizontal stress is calculated by subtracting the hydrostatic pressure from the total stress applied to the probe. The testing procedures for type A is thereafter similar to the Menard pressuremeter test (Technique Louis Menard, 1975). For type C, the probe can be used for strain control testing by injecting water at a constant rate of volumetric variation, $\Delta v/v$, equal to 4 percent per minute by means of a piston pump with a precisely regulated cyclic motion.

TESTS IN SOFT COHESIVE SOIL

Soil Conditions: A site was selected in the Komatsugawa area located along the northern coast of Tokyo Bay. The ground was covered with loose delta sand of 8 m underlain with soft cohesive soil to a depth of 21 m. In Figure 3 are shown index properties of soil at the test site. The ground water table was about 2.30 m deep from ground surface during testing, and seasonal variation of the water table was expected.

The cohesive soil was classified as CL, MH or CH according to the Unified Classification, but its liquid limit was considerably lower than that of common Tokyo Bay mud whose liquid limit is usually 80 to 100%. The cohesive soils at this site contained 5 to 20 percent non-uniformly mixed sand (See Fig. 3) with a number of thin pipes or laminations. Small fragments of shells were also evident. Undisturbed samples were obtained at each meter in depth using a 75 mm thin-walled sampler with fixed piston.

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The over-consolidation ratio (OCR) was calculated from the results of oedometer tests conducted in accordance with the Japanese Industrial Standard. The OCR calculated is plotted against depth in Figure 4. The OCR values are close to unity in the middle of the layer, and higher than 2 near the upper and lower boundaries.

The undrained shear strength of samples was measured by consolidated undrained tests. Specimens were reconsolidated by isotropic pressure equivalent to the mean effective confining pressure in situ assuming the coefficient $K_0 = 0.5$. The undrained shear strength, c_u , was obtained from the deviatoric stresses at failure in undrained triaxial tests.

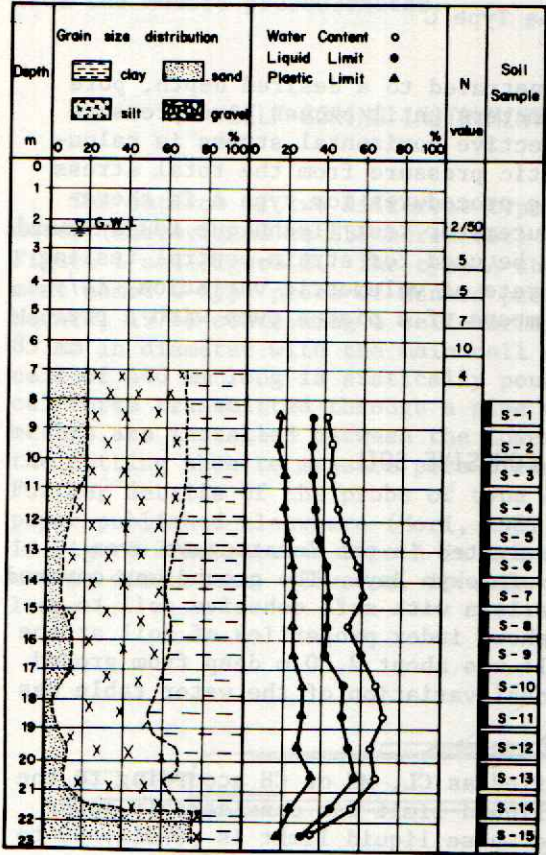


Fig.3 Soil conditions at the site of Komatsugawa, Tokyo

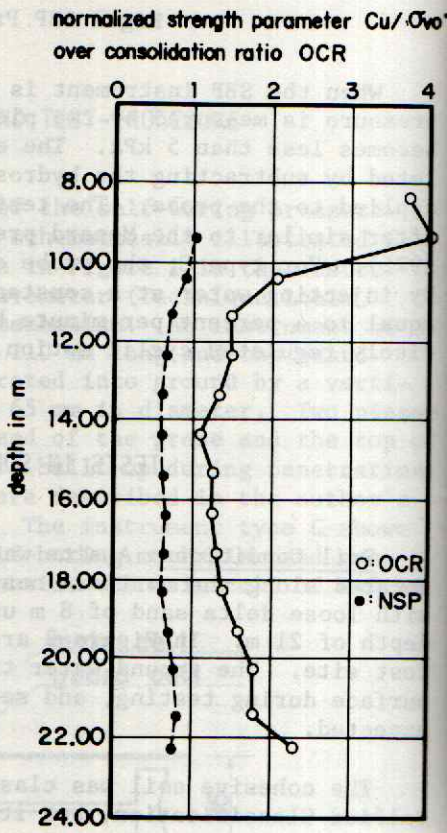


Fig.4 Profile of OCR and NSP at the site of Komatsugawa, Tokyo

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The secant modulus, E_{50} , corresponding to 50 percent of the deviatoric stress at failure was calculated from stress-strain curves of the undrained triaxial tests. The shear modulus, G , was calculated from the equation, $G = E_{50}/2(1+\nu)$, where Poisson's ratio, ν , was assumed to be 0.5 for undrained deformation. The average values of modulus E and G for three specimens trimmed out of each sample were obtained. The consistency of the test results appeared satisfactory, since the coefficient of variation with respect to these parameters was less than 0.19.

The undrained shear strength was normalized with respect to the effective overburden pressure and plotted against depth as shown in Figure 4. The normalized strength parameter (NSP); $c_u/\sigma_{v0}' \approx 0.5$, for $OCR = 1$ is higher than the NSP values for Tokyo Bay alluvium, i.e. $c_u/\sigma_{v0}' = 0.25 - 0.30$, obtained from the results of unconfined compression tests.

The SBP Test: The SBP tests were carried out in three boreholes drilled at an interval of 2 m from each other. The self-boring instrument penetrated to a desired depth was held immovable until the pore pressure built up during penetration dissipated. It took about two hours until excess pore pressure became less than 5 kPa. Subsequent to the measurement of effective horizontal stress in situ, the probe was inflated by pressure applied in steps. Each increment of pressure remained constant for 2 minutes in order to compare the results of the SBP tests to those of the MP tests.

Typical results of pressuremeter tests are illustrated by the conventional pressure-volume curves shown in Figure 5. In this figure the results obtained from the MP test are compared to those of the SBP tests. The curves of both tests are asymptoted for the pressure close to the limit pressure, but the slopes of curves of the SBP test within yield pressure are obviously different from those obtained from the MP tests. From initial tangent to the pressure-volume curves obtained from the SBP test, the pressuremeter modulus, E_p , was calculated by the equation,

$$E_p = 2(1 + \nu)v_0\left(\frac{dp}{dv}\right)_{p=\sigma_h'} = 2(1 + \nu)G \quad (1)$$

where v_0 : initial volume of probe (use $v_m = (v_0 + v_f)/2$ for Menard pressuremeter, where v_f : volume corresponding yield pressure)

ν : Poisson's ratio; assumed 0.5 for undrained tests

G : shear modulus assuming elasticity for small strain

The modulus E_p was also calculated from the curves of the MP tests using the average slope of a curve within the yield pressure. In Figure 6 the moduli obtained from the SBP tests and the MP tests are plotted against depth. The open circles represent the mean values of

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E_p obtained from the SBP tests conducted three times at each depth. Maximum and minimum values are also shown at both sides of the open circles. The mean values of the modulus obtained from the MP tests conducted twice are shown by closed circles. It is noted that the moduli obtained from the SBP tests are 2 to 3 times greater than those from the MP tests.

There was no remarkable difference between the limit pressures of the SBP and the MP tests. Parameters such as pressuremeter modulus, E_p , limit pressure, p_l , and yield pressure, p_f , were sufficiently reproducible. The deformation modulus was seriously influenced by a thin disturbed zone, but the limit pressure was not affected as the soil mass mobilized against the limit pressure was several times greater in diameter than the borehole, and this large soil mass was possibly free from disturbance caused by boring.

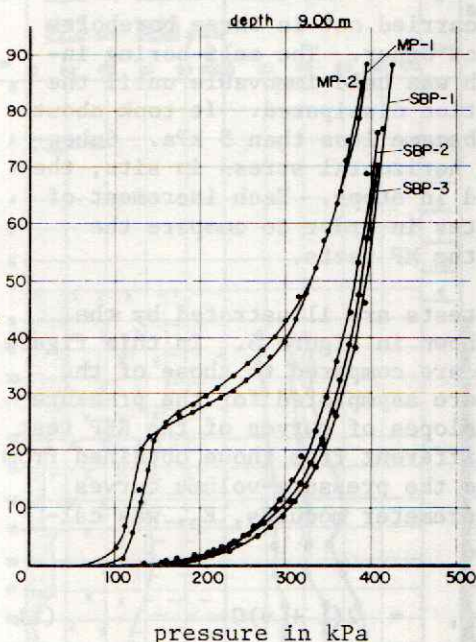


Fig.5 Results of the SBP test in comparison with the MP test, Komatsugawa, Tokyo

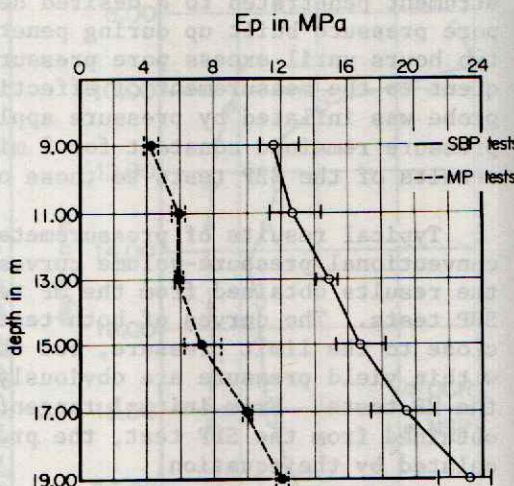


Fig.6 Pressuremeter modulus obtained from the SBP tests and the MP tests

Stress-Strain Curves: From the results of the SBP tests, stress-strain curves of an element in contact to the pressuremeter probe

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were produced applying the procedure proposed by Palmer (1972). The deviatoric stress, $\sigma_r - \sigma_\theta$, for an element in contact to the probe was obtained by the equation:

$$\sigma_r - \sigma_\theta = 2 \frac{dp}{d[\epsilon_n(\Delta v/v)]} \quad (2)$$

where p : pressure applied by the probe

$\Delta v/v$: relative increment of the volume of a cylindrical cavity

where $v = v_0 + \Delta v$

v_0 : initial volume of a cylindrical cavity

The circumferential strain, ϵ_θ , was computed from the equation:

$$(\epsilon_\theta)_{r=r_0} = (1 - \Delta v/v)^{-1/2} - 1 \quad (3)$$

where r_0 : initial radius of cavity

The shear modulus, G , was calculated from pressure versus circumferential strain curves by the equation:

$$G = \frac{d\tau}{d\gamma} = \frac{d\tau}{2d\epsilon_\theta} = \frac{1}{2} \left(\frac{dp}{d\epsilon_\theta} \right)_{\epsilon_\theta=0} \quad (4)$$

where τ : shear stress

γ : shear strain

Equation (4) is similar to that derived by Palmer, but the shear modulus is given as a derivative of shear stress with respect to shear strain which is twice as much as the circumferential strain.

The shear moduli obtained from both the SBP tests and the tri-axial tests are summarized in Figure 7. The shear moduli calculated by Equation (1) agreed with those obtained from stress-strain curves (Eq.(4)). The moduli obtained from the SBP test were a little smaller than those obtained from the CU-test for depths less than 13 m, and a little larger for depths more than 15 m.

Gibson and Anderson (1961) derived the following equation assuming that soil is ideally plastic beyond the yield point:

$$p_\ell = p_0 + c_u \left\{ 1 + \epsilon_n [E/2c_u(1 + \nu)] \right\} \quad (5)$$

where p_ℓ : limit pressure

p_0 : horizontal stress in situ

c_u : undrained shear strength

E : Young's modulus

ν : Poisson's ratio

PRESSUREMETER TEST FOR SOFT COHESIVE SOIL

The undrained shear strength obtained from different methods of calculation or testing are compared in Figure 8. The undrained shear strength calculated by Equation (5) using the p-v curves is about 70 to 80 percent of the strength obtained from the stress-strain curves, which was in good agreement with the strength obtained from the CU-test.

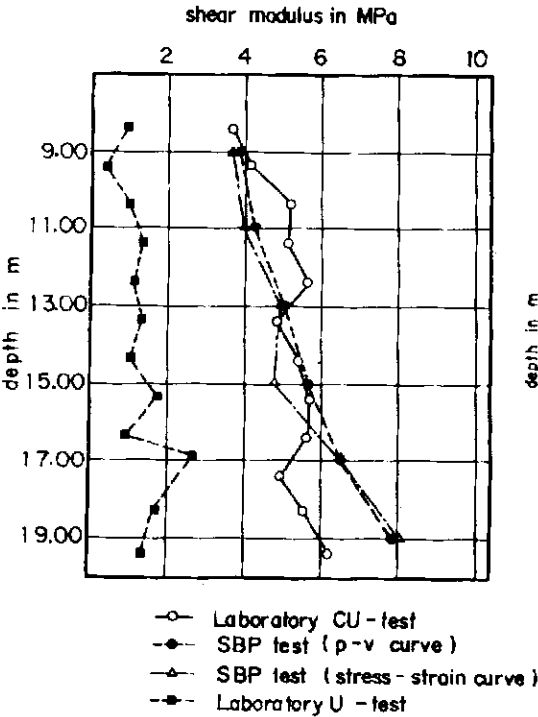


Fig.7 Shear moduli obtained from the SBP tests and laboratory tests

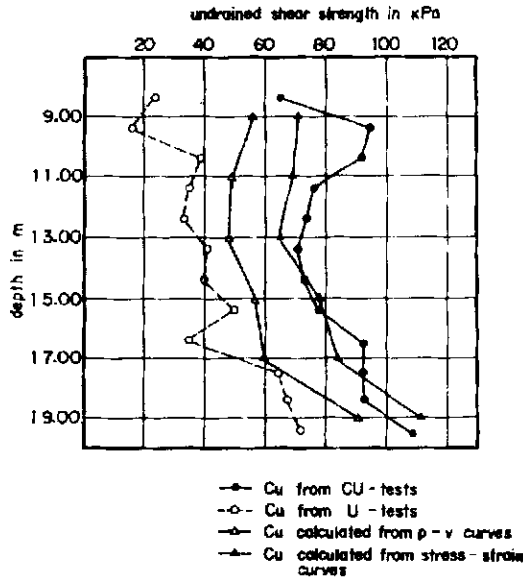


Fig.8 Undrained shear strength obtained from the SBP tests and laboratory tests

STRAIN CONTROL TESTS

In the case of the conventional stress control testing methods, the probe is inflated by pressure applied in steps. Each increment of pressure remains constant for 2 minutes. This method usually requires 40 minutes for completion, and may result in a partial drainage of soil around the pressuremeter probe. A series of tests with the value of $\Delta v/v$ equal to 50 percent will be completed in 12.5

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were produced applying the procedure proposed by Palmer (1972). The deviatoric stress, $\sigma_r - \sigma_\theta$, for an element in contact to the probe was obtained by the equation:

$$\sigma_r - \sigma_\theta = 2 \frac{dp}{d[\epsilon_n(\Delta v/v)]} \quad (2)$$

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$$p_\ell = p_0 + c_u \left\{ 1 + \epsilon_n [E/2c_u(1 + v)] \right\} \quad (5)$$

where p_ℓ : limit pressure

p_0 : horizontal stress in situ

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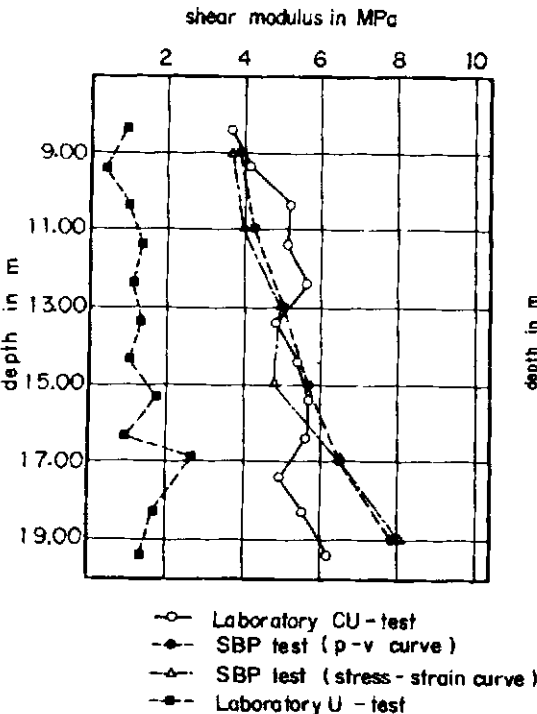


Fig.7 Shear moduli obtained from the SBP tests and laboratory tests

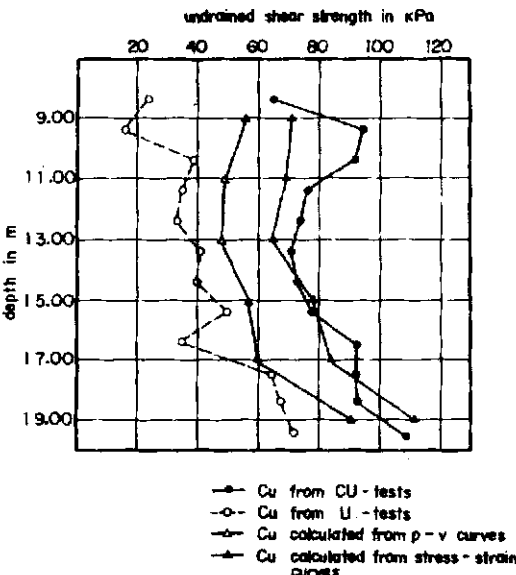


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STRAIN CONTROL TESTS

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minutes if the test is conducted at a constant rate of $\Delta v/v$ equal to 4 percent per minute. This rate of volume change is equivalent to a 2 percent per minute rate of circumferential strain. The strain control test automatically controlled and recorded produces more accurate data than the one manually controlled and measured.

In order to compare the results of the strain controlled test to those of the stress control test, additional SBP tests were carried out using the instrument types A and C. The strain control tests were performed using type C, and the stress control tests were performed using both types A and C.

Soil Conditions: The site for the additional tests was selected in a lot a few hundred meters distant from the site of the first tests described in the preceding Articles. The soil profile of the site was almost the same as that of the first site although the soil to a depth of 9.3 m contained more than 20 percent of sand, and the ground water table was 0.6 m below the ground surface.

The undrained shear strength of samples obtained at each meter of depth was measured by triaxial tests under two drainage conditions, i.e. the CIU-test and the UU-test. In the CIU-tests, specimens were reconsolidated by isotropic pressure equivalent to the mean effective confining pressure in situ assuming the coefficient $K_0 = 0.5$. The undrained shear strength measured by the CIU-test, c_{uc} , and by the UU-test, c_{uu} , and normalized by effective overburden pressure is plotted against depth in Figure 9, in which the profile of the OCR is also illustrated. Disregarding the data obtained at depths of 8 and 9 m due to high sand content, the average of the ratio, $R = c_{uc}/c_{uu}$, was 1.57 with a coefficient of variation, $\sigma_{n-1}/\bar{R} = 0.099$, for 7 variables, where $\bar{R}$ is the mean value of R .

The SBP Test: The SBP test was carried out at depths of 10.5, 12.5 and 15.5 m below the ground surface. The tests were repeated three times at each depth using three different boreholes located at an interval of about 2 m from each other. The stress control method was employed with the instruments types A and C. The strain control method was employed with the instrument type C only, as the instrument type A could not be operated by a piston pump system for the strain-control test.

In Figure 10 are shown the stress-strain curves derived from the strain control and stress control tests for specimens obtained at a depth of 15.5 m. In this figure, the undrained shear strength, in general, shows no significant difference between the strain control and stress control tests. Small differences in strength are considered due to heterogeneity of soil in the horizontal direction. The peak stress in the strain control test, however, appears at a smaller strain (1 to 2 percent) than that in the stress control tests.

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The stress-strain behavior of the majority of specimens tested by strain control showed a trend of strain softening which was not distinguishable from the results of the stress control tests.

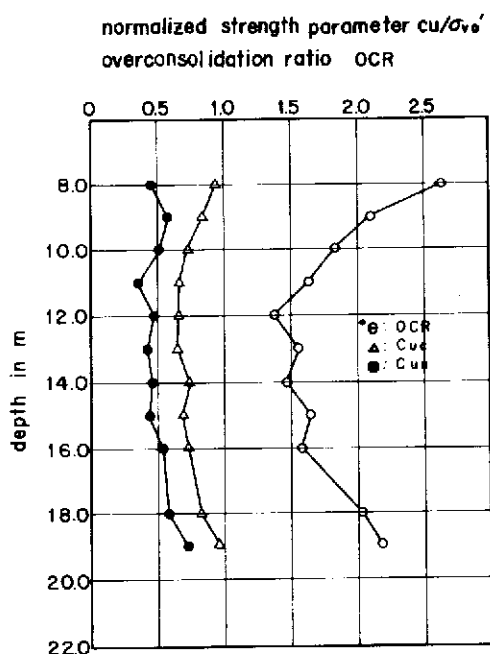


Fig.9 Profile of OCR and NSP at the second site

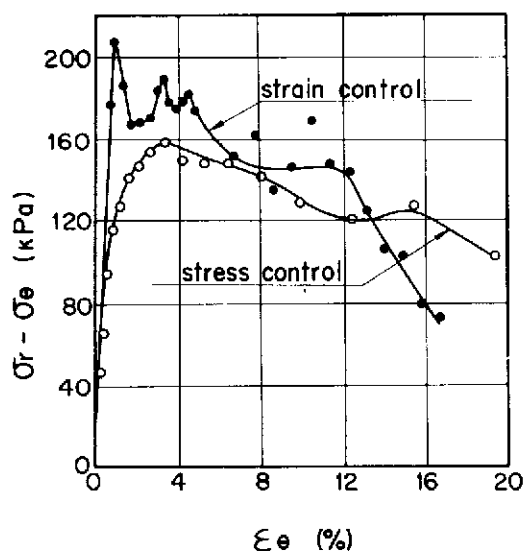


Fig.10 Stress-strain curves obtained from the SBP tests

The undrained shear strengths and shear moduli obtained from the SBP tests are summarized in Tables I and II. The values shown in these tables are mean values of strength and shear modulus obtained from the tests repeated three times at each depth. In terms of the strength and stiffness of cohesive soil measured at Komatsugawa, no significant difference was distinguishable between the use of type A or C instruments. However, discussion on the effectiveness of the guard cell used in the Menard pressuremeter, Type G, is beyond the scope of this paper.

The undrained shear strength obtained from the strain control tests appears slightly higher than that obtained from the stress control tests. The average values of undrained shear strength obtained from the CIU triaxial tests, c_{uc} , and the UU triaxial tests,

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c_{uu} , are also shown in Table I. It is noted that the strength measured by the SBP strain control tests agrees closely with that obtained from the CIU triaxial tests.

The shear modulus measured by the strain control tests is, as shown in Table II, about 30 percent higher in average than that measured by the stress control tests. The extent of this difference in shear modulus is not very significant from a practical point of view, but it is considered worthwhile to study the difference in the stress-strain behavior of various types of clay depending on the control methods of the SBP tests.

Table I. Undrained shear strength obtained from the SBP test

depth (m)	stress control type A (kPa)	stress control type C (kPa)	strain control type C (kPa)	c_{uc} (kPa)	c_{uu} (kPa)
10.5	61	55	61	60	38
12.5	66	62	65	66	45
15.5	84	75	93	85	57

Table II. Shear modulus obtained from the SBP test

depth (m)	stress control type A (MPa)	stress control type C (MPa)	strain control type C (MPa)
10.5	3.76	3.65	5.38
12.5	4.88	6.18	7.07
15.5	6.39	6.99	7.06

Measurement of In-situ Horizontal Stress: The horizontal effective stress in situ was calculated subtracting hydrostatic pressure from the total stress, p_0 , measured when the excess pore pressure built up by penetration of the instrument type C, dissipated to less than 5 kPa. The total stress, p_0 , was measured six times at each depth, and the mean values and the coefficient of variation for each are summarized in Table III. With a coefficient of variation less than 0.07, the total stress, p_0 , measured for Komatsugawa clay is thought to be consistent despite probable variation of properties in the horizontal direction. In Table III are also shown the K_0 -coefficients for three depths. In evaluation of the effective vertical stress, σ_v' , in Table III, the wet density of sandy soil above

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cohesive soil was assumed 1.8 t/m^3 taking into account of the wet density of 1.76 t/m^3 measured for an undisturbed sample at a depth from 8 to 9 m. But the effective overburden pressure at a depth of 10.5 m can be 10 percent higher than the value shown in Table III if the wet density of sandy soil is assumed 1.9 t/m^3 . The K_0 -coefficient is considered to be a parameter conveniently used for design purposes, but for sophisticated analysis of geotechnical problem the horizontal effective stress should be determined from the measurement at the exact depth of an element of ground.

Table III. Horizontal stress in situ

depth (m)	P_0 (kPa)	μ	σ_h' (kPa)	σ_v' (kPa)	K_0
10.5	147	0.066	51	84	0.61
12.5	163	0.055	55	97	0.57
15.5	211	0.054	83	116	0.71

μ : coefficient of variation for 6 samples

CONCLUSION

The conclusive results of the study presented in this paper are summarized as follows:

- (1) The modulus of deformation of soft cohesive soil obtained from the SBP test was 2 to 3 times greater than that from the MP tests.
- (2) There was no significant difference between the SBP and MP tests with regard to the limit pressure of soft cohesive soil.
- (3) The shear modulus calculated from the SBP test in soft cohesive soil coincided with that obtained from the CIU-triaxial test.
- (4) The undrained shear strength obtained from the SBP test was well correlated to that obtained from the CIU-triaxial test.
- (5) The strain control test yield a higher shear modulus and a smaller strain at failure as compared to the stress control test.
- (6) Reproducible values of the horizontal effective stress can be obtained from the SBP test.

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V. N. DENISOV , N. S. CHETYRKIN, A.V. GOLUBEV*

LONG-TERM INVESTIGATION OF SOIL DEFORMABILITY BY AUTOMATIC PRESSUREMETER

SYNOPSIS. The principle of operation of the PA-108 automatic pressuremeter developed by NIIOSP is described in the paper. The apparatus registers pressure in the hydraulic system and volume of pressure fluid pumped in by delivery pump-batcher, on the tracing tape. Automatic control of the strain gauge indications in unloaded condition is provided. The pressuremeter operates in the regime of given load and the regime of predetermined deformation. The first experiments of studying of attenuating creep of hard clays are described. Experimental curves of creep and results of determination of exponential creep parameters are discussed. Borehole radius increment during the first hour of loading made about 60% of the total increment on the average in the experiments described. The long-term modulus of deformation of hard clays investigated is 2-3 times less than the "instantaneous" modulus.

I. INTRODUCTION

There is a certain tendency all over the world to develop automatic apparatuses for laboratory and in-situ testing of soils. Automation of the soil loading process and of registration of measuring devices indications improves reliability and accuracy of the experimental data as it excludes the observer's subjectivity when carrying on an experiment and putting down the

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results obtained; there is no need any more in operator's presence which simplifies the organization of a long-term continuous testing and raises investigator's productivity of labour.

II. THE AUTOMATIC PRESSUREMETER

The PA-108 automatic pressuremeter was developed in NIIOSP for the purpose of automation of the in-situ soil testing /1/. For its construction two inventions were used - "Dilatometer" on the authors' certificate No. 266310 /2/ and "Device for Soil Deformation Determination in Boreholes" on the authors' certificate No. 750326/3/

The pressuremeter axisymmetrically loads a zone of prebored hole with the cylindrical probe.

The PA-108 pressuremeter carries out testing in the regime of predetermined velocity of bore-hole walls displacement and in the regime of given pressure maintainance. The first regime is used for determination of soil deformation modulus E, the second- for creep parameters determination.

A. CONSTRUCTIVE FEATURES

When loading soil in the regime of predetermined deformation, radial displacement of the borehole walls with given velocity is provided by the delivery pump-batcher of adjustable delivery, which pumps in through steel tubes under rubber membrane of the probe chosen by an operator quantity of fluid from a measuring cylinder. The minimum velocity of radial displacement of the PA-108 membrane is $3 \cdot 10^{-6}$ m/min. The displacement is measured with the accuracy up to $6 \cdot 10^{-6}$ m. Radial displacement value is calculated using volume of the pumped in fluid. The volume measurement is carried out with a special gauge placed into the measuring cylinder. When testing the measuring cylinder is filled in automatically from a reserve tank which contains the supply of fluid sufficient for the whole experiment.

In the regime of given load on the borehole walls it is envisaged to create as soon as possible required pressure in a hydraulic system and maintain it on the achieved level. For this purpose the pump-batcher

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is switched on maximum delivery and a contact manometer provides an automatic switching it off and on for pressure maintenance at a given stage.

Thus, a given value of pressure is being maintained with the accuracy of $\pm 3 \cdot 10^{-3}$ MPa. To set a certain pressure stage it is necessary to fix pointers of the contact manometer. Pressure in the hydraulic system is measured with a pressure gauge with the accuracy up to $2,5 \cdot 10^{-3}$ MPa.

The indications of pressure and soil displacement gauges are registered on the tracing tape. When testing these gauges periodically are unloaded with the aid of special devices without disturbance of the regime of an experiment and their indications at the moment are fixed on the tracing tape, too. As a result, four curves are typed on the tape. To determine pressure and displacement values it is necessary to measure the distances between corresponding curves and multiply them by calibration factor. This constructive feature of the PA-108 pressuremeter makes it possible to take into consideration fluctuation of gauges indications caused by changing of weather conditions during the long-term experiments and by other reasons. The PA-108 pressuremeter tracing tape can move with two different velocities - for short and long-term experiments.

B. CARRYING OUT EXPERIMENTS WITH PA-108 AND SPADE WORK ON THE RESULTS

The spade work on the tracing tape with the experiment results consists of composing of a table which contains readings of time " t_i " and the corresponding values of pressure on the borehole walls " P_i " and their radial displacement " U_i ". These parameters are then used for drawing graph "pressure-displacement" and creep curve.

Soil deformation modulus, as it is known, depends under other equal conditions on the method of investigation: when testing in the regime of given load-which is accepted in the USSR as the leading one and standardized - on duration of loading at every stage depending, in its turn, on accepted criterion of conditional stabilization of soil deformation; in the regime of predetermined deformation it depends on soil

displacement velocity. Comparative pressuremetric experiments were carried out in different regimes of loading in NIIOSP /4/ which showed that close values of the modulus E were obtained when velocity of predetermined deformation and an average velocity of soil displacement under stage loading with maintenance of the pressure at every step up to conditional stabilization of deformation coincide.

For more complete evaluation of soil deformation properties it is necessary to study its creep. Parameters of creep are determined from the results of long-term soil investigations. Usually these investigations are carried out in laboratory because the organization of the whole day round long-term observations in-situ finds many difficulties. At the same time, well known disadvantage of the laboratory testing connected with an unavoidable disturbance of soil structure during sampling makes field investigations preferable, because soil is tested here in less disturbed condition which makes it possible to evaluate its properties more objectively.

Self-boring pressuremeter proved to be the most undisturbing one among varieties of the apparatus. However, application of pressuremeters with probes lowered into a hole prebored in stiff stable soils will not cause large inaccuracy in the results of experiment. Taking it into consideration the authors applied the PA-108 pressuremeter for field investigations of soil creep.

As is known /5,6/, loading of soil for determination of the creep parameters can be carried out either by steps without changing the place of loading (or the sample), or carrying out investigation on neighbouring plots with identical soil conditions (or on the samples-twins) with different loads. In both cases soil deformation is being fixed during the whole experiment. Both methods have their own advantages and shortcomings and to make one's choice one should take into consideration the type of soil, period of time available for research work and other factors.

In the experiments described hard layered clays of $1,6 \text{ t/m}^3$ density with the natural water content and porosity 60,9% were investigated. The depth of testing was 8-9m. The 108mm in diameter boreholes were bored by the URB-2M rotary drill used also for lowering into the borehole and lifting from it the pressuremeter probe with the hydraulic tubing. The PA-108 pressure-

LONG-TERM PRESSUREMETRIC INVESTIGATION

meters at the investigation site are shown on figure 1.



Fig.1. The PA-108 pressuremeters on the investigation site

Soil loading was being carried out by stages with maintenance of the pressure at every stage for 17-28 hours. Pressure at the first stage was somewhat higher than overburden pressure and equal in different experiments $0,162-0,165\text{MPa}$. The pressure at every next stage (three in succession) was increased by $4,8 \cdot 10^{-2}\text{MPa}$ in comparison with the preceding stage. Nominal value of pressure in the hydraulic system was maintained with accuracy of $\pm 3 \cdot 10^{-3}\text{MPa}$. Graph of the stage loading of soil typical for carried out experiments is shown on figure 2 (experiment No.16). The borehole radius increment during the first hour of loading made about 60% of the total increment on the average in the experiments described.

III. ANALYSIS OF THE RESULTS OF PRESSUREMETRIC INVESTIGATIONS OF SOIL CREEP

A. EXPERIMENTAL CREEP CURVES APPROXIMATION

Here, analysis of the results of the first pressu-

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Experiment 16

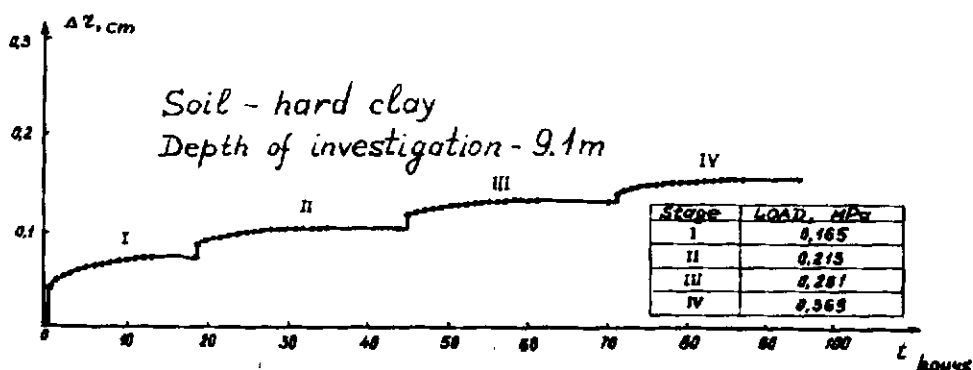


Fig. 2. Typical graph of the stage soil loading

remetric investigations of hard clays performed by the authors by the PA-108 pressuremeter is presented as an example. As one can see from figure 2, soil deformation under given values of load at every stage of loading are of attenuating nature. To describe creep the following approximating function was used

$$U(t) = \frac{(1+\nu)}{E_0} r_0 p \left[1 + E_0 \theta (1 - e^{-\lambda t}) \right], \quad (1)$$

where

$U(t)$ - radial displacement of the borehole walls under load;

ν - Poisson's ratio;

E_0 - deformation modulus (instantaneous) of soil,

r_0 - initial radius of a borehole;

p - pressure at the borehole walls;

λ and θ - parameters of the exponential core of creep.

It should be mentioned that the method of approximation used is not the single possible one; to describe creep of the same type hyperbolic, for instance, and other functions can be used.

For example, Yu.K.Zaretsky et al the first to use the pressuremeter (D-76 type designed at NIIOSP, Moscow,

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USSR) for investigations of plastic flow deformation of ice /7/ came to the conclusion that the deformation of ice in the zone of plastic flow can be described by the following expression

$$\epsilon_i^{(c)}(t) = \frac{B_1}{A_1} \sigma_i^n \cdot t^\lambda,$$

where $\epsilon_i^{(c)}$ - intensity of shear creep deformation,

σ_i - intensity of shear stresses

$$\sigma_i = \frac{1}{\sqrt{6}} \sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_1 - \sigma_3)^2 + (\sigma_2 - \sigma_3)^2}$$

B_1, n, λ - dimensionless parameters;

A_1 - parameter with dimension $\text{kgf}^n / \text{cm}^{2n \times \text{min}}$

It is necessary to note that maintenance of the constant pressure at the borehole walls with nonautomatic D-76 pressuremeter was a difficult task in conditions of the constant flow of ice.

Generally speaking, having combined different functions, having changed their parameters one can select the relation which will allow to get the best approximation of the experimental creep curve. Vyalov, however, supposes /5/ that "there is no need to describe creep curves analytically with great accuracy because of the large scatter of the experimental data unavoidable when investigating creep of any material (for metals 20% scatter is considered not very large). So, in most cases it is reasonable to use the approximating relations most convenient for practical calculations". On the same reason S.S.Vyalov recommends /5/ to accept the assumption of similarity of creep curves and isochronous curves every time when the assumption does not lead to substantial distortion of the experiment results.

Since

$$\frac{(1+\nu)r_{0P}}{E_0} = U_0, \quad (2)$$

where U_0 "instantaneous" radial displacement of the bore-

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hole walls,
expression (1) can be transformed as

$$U(t) = U_0 [1 + \theta E_0 (1 - e^{-\lambda t})] \quad (3)$$

Further it will be shown that the experimental creep curves obtained by the authors are described with quite sufficient accuracy by equation (3).

B. DETERMINATION OF θ AND λ PARAMETERS OF THE EXPONENTIAL CORE OF CREEP

First of all, using the Lamé solution (formula (2)) the "instantaneous" modulus of deformation E_0 is found. "Conventionally instantaneous" displacement U_0 assumed to be radial borehole walls displacement on reaching the nominal pressure of the experiment (the moment of the first switching off the delivery pump-batcher).

Parameter θ in the expression (3) can be determined as

$$\theta = \frac{U_k - U_0}{U_0 E_0} \quad (4)$$

obtained from (3) when $t \rightarrow \infty$. Here, U_k - the final radial borehole walls displacement (at the end of the loading period). One should note that the longer the stabilized deformation part of a creep curve in comparison with the intensive growth of deformation part of the same curve, the less inaccuracy of the assumption ($U_\infty = U_k$) is.

Parameter λ is determined using the method of statistical analysis of the experimental creep curve described in details in the monograph /5/ by S.S.Vyalov.

Using this method for calculations of the parameters θ and λ one can achieve quite sufficient for practical purposes accuracy of approximation of the experimental creep curve. On figure 3 the experimental creep curve is shown by the solid line, the approximating exponent is shown by dotted line.

The coefficients of correlation were calculated to evaluate the degree of correlation, their values never less than 0,99, proving strength of correlation, the

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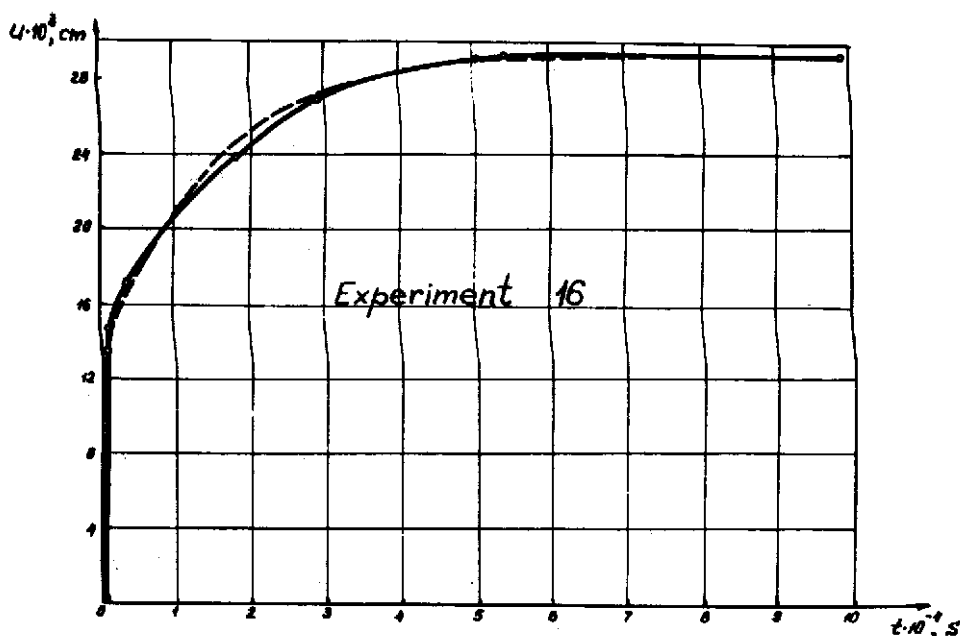


Fig.3. Experimental and approximating creep curves

———— Experimental
 ----- approximating

coefficients of variation being 0,0101-0,0203.

Subsequent statistic analysis of the results obtained includes determination of the confidence limits where the true values of θ_{tr} and λ_{tr} creep parameters take place for given confidence level α . These calculations are carried out by the standard method, that is why they are not given here. The final result is the equation (3) with the true values of θ and λ parameters and the confidence limits of their deviations.

G. MEASURE OF CREEP

The experimental creep curves in every experiment described were obtained for pressure $\sigma_1 = 0,048$ MPa. S.R.Meschian in his works (see, for instance /6/), following N.Kh.Arutyunyan /8/, shows that relationship between deformation and time $\epsilon(t)$ may be, in case of linear relation between ϵ and acting stress σ_1 , expressed

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by

$$\varepsilon(t) = \sigma_1 C(t, \tau),$$

where $C(t, \tau)$ - the measure of creep, that is, on definition, the creep curve corresponding to the unit load. The graphs of $\varepsilon_r(\sigma_1)$ relation for three experiments with stage loading are given at figure 4. It is easy to

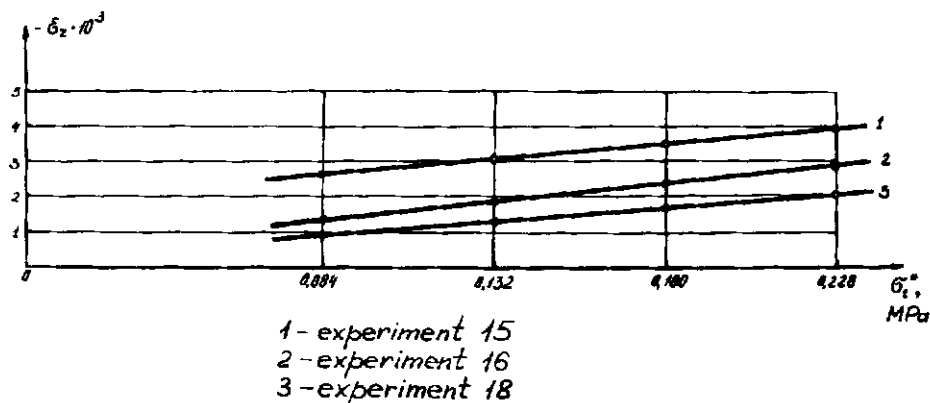


Fig.4. Relationship between soil deformation and stress

see that abovementioned relation obtained when $t = t_k = \infty$ is linear. Therefore, the expression for measure of creep may be obtained by division of the relation (3) with parameters calculated, by 0,048 MPa. Knowing the expression for the measure of creep it is not difficult to obtain a creep curve for any stress σ_1 , which does not cause plastic flow.

D. CALCULATION OF LONG-TERM DEFORMATION MODULUS

The long-term deformation modulus can be easily calculated by the following expression /9/:

$$\frac{1}{E_\infty} = \frac{1}{E_0} + \theta. \quad (5)$$

There is certain interest in comparison of the values of the "instantaneous" and long-term moduli of

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deformation. In table 1 the values of E_0 and the calculated using formula (5) values of E_{∞} with consideration of deviation of θ parameter are given as an example (three experiments concerned).

Table 1

No. of experiment	E_0 , MPa	E_{∞} , MPa	E_{∞} tr MPa
15	34,4	12,6 -16	14,3 $\pm$ 1,7
16	28,4	12,1 -14	13,1 $\pm$ 1
17	55,6	19,7 -20,5	20,1 $\pm$ 0,4

IV. CONCLUSIONS

Developed in NIIOSP the PA-108 automatic pressuremeter proved to be the reliable apparatus providing information of sufficient accuracy and objectivity read out from the tracing tape.

The apparatus carries out the pressuremetric loading of the borehole in regimes of given load and predetermined deformation.

Investigation of the hard clay creep gave hopeful results; the approximation of the experimental curves made it possible to calculate parameters of the exponential core of the attenuating creep. Apparently, similar investigation of soil of different kind or the same soil but with the larger loads may give nonattenuating creep curves approximated by creep core of another type.

The increment of the borehole radius during the first hour of loading made on the average for majority of the experiments described about 60% of the total deformation.

The long-term deformation modulus of investigated clay is 2-3 times less than the "instantaneous" deformation modulus.

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SESSION 1

ADDITIONAL CONTRIBUTIONS AND DISCUSSION

CONTRIBUTION

A NEW DEVICE FOR IN SITU MEASUREMENT OF SOIL COMPRESSIBILITY

C. HERGHELEGLU- (*Central Building Research Institute, Bucharest, Romania*)

E. UNCHESL- (*Tehnoton, Jassyn, Romania*)

ABSTRACT

This paper discusses a prototype pressuremeter built in Romania. The unique design of the device enables it to be used in different types of soil (sand, soft clay, stiff clay). The pressuremeter cell is metal and does not use a rubber membrane. This paper is merely intended to draw attention to the design and main features of the device.

DESCRIPTION OF THE DEVICE

The prototype pressuremeter manufactured in Romania is equipped with a cylindrical metal pressuremeter cell, consisting of six sections parallel to the generating line of the probe. Each section is set in motion by a mechanical device actuated by a single hydraulic unit.

The spaces occurring between the sections during radial deformations of the probe are closed by a mechanical system that guarantees the continuity of the probe surface during the tests.

SESSION 1 ADDITIONAL CONTRIBUTIONS AND DISCUSSION

The device, called IMC-120, features the following components (Fig. 1) :

- . pressuremeter cell,
- . control, monitoring and measurement console,
- . pressure supply and deformation measurement tubes,
- . tripod for use above the borehole and probe pullout system.

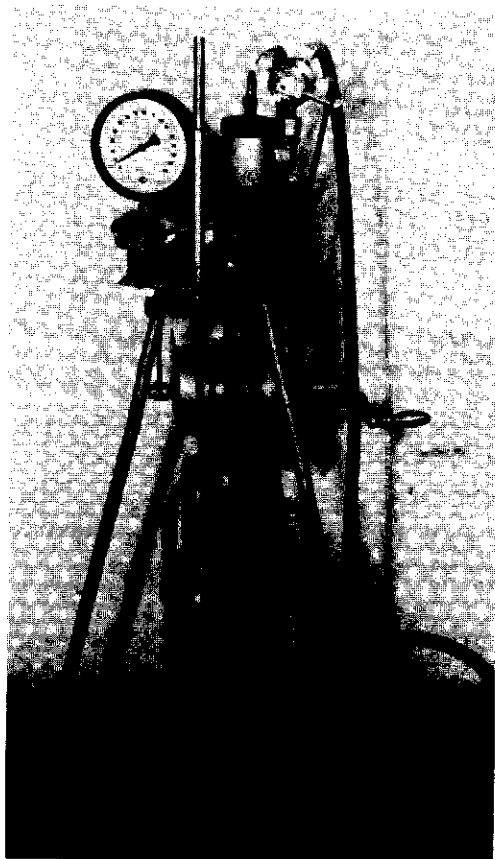


Fig. 1 - General vue of the device

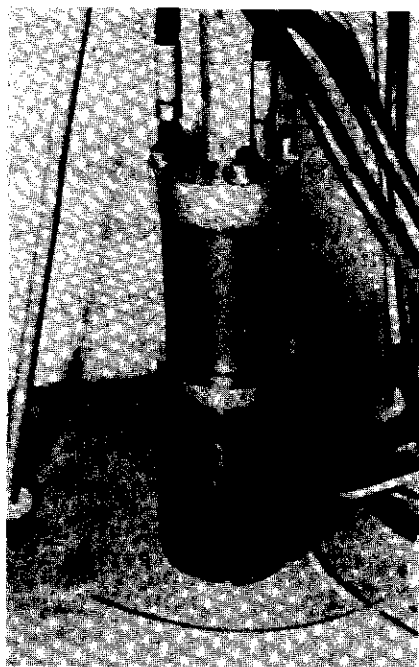


Fig. 2 - The pressure cell

The pressuremeter cell described above is designed so that during expansion its cross-section is inscribed within a polygonal line closely approaching a circle.

Horizontal movements of the shells are monitored by a deformation measurement system located in the console at the surface (Fig. 3).

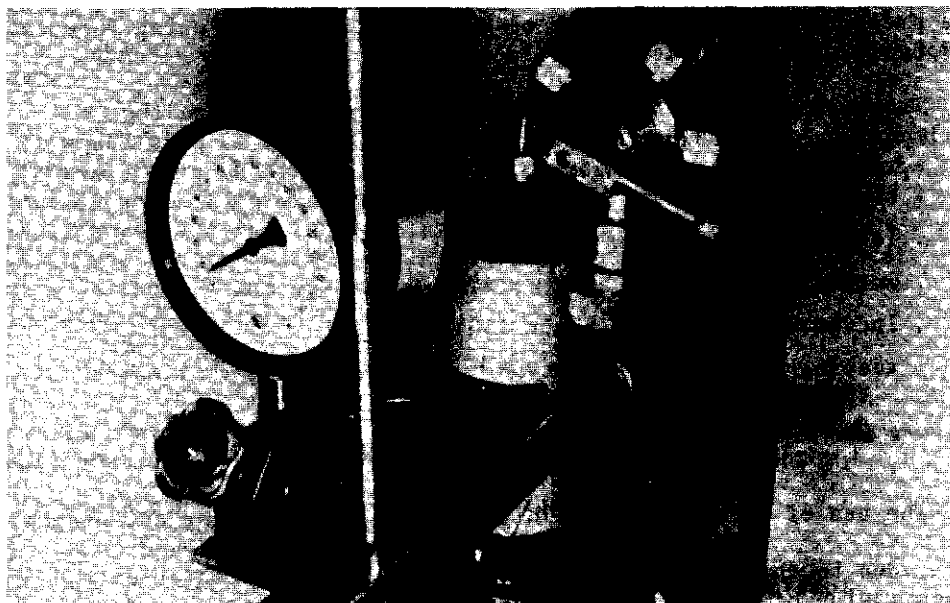


Fig.3 - Control and measurement console (for pressure and deformation)

The probe expansion system is actuated by a hydraulic press equipped with a double-acting piston.

The piston displacement is controlled by a rod fixed to its upper end which travels inside a metal chamber filled with a liquid and is connected to the instrument measurement console. In this way, the piston displacement causes movement of the sections and transmission to the surface of the radial deformation values of the borehole walls resulting from the expansion of the pressuremeter. A special amplification system of these deformations has been developed to improve measurement accuracy.

The pressure applied by the probe against the borehole walls at each loading stage is read on a pressure gauge mounted on the surface console (Fig.3).

The hydraulic pump is started manually and coupled to a small oil tank which serves to power the installation.

The data recorded at each loading stage are :

- . pressure,
- . radial displacement of the cell.

SESSION 1 ADDITIONAL CONTRIBUTIONS AND DISCUSSION

Based on the geometric loading conditions and experimental data, the pressure values during the test can be calculated using formulas, computers, tables or diagrams.

The main technical characteristics of the device are the following :

- . nominal diameter of the probe : 120 mm,
- . length of probe : 200 mm,
- . increase in surface area : 160%,
- . increase in volume : about 250%,
- . testing depth with prototype : 15 m.

ADVANTAGES OF THE DEVICE

The use of this device has highlighted the following advantages :

- . due to the use of flexible tubes for the implementation and transmission of radial deformations of the pressuremeter probe, the device is capable of operating at greater depths than those mentioned above,

- . the loading and probe deformation monitoring systems operate independently,

- . simple and rugged mechanical construction of the device make it very easy to use and to transport.

This prototype device is currently undergoing improvements. A new version is planned for borehole depths up to 50 m, in soft rocky soils. This new instrument is planned with the possibility of automatic data recording during the tests.

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SESSION 1 ADDITIONAL CONTRIBUTIONS AND DISCUSSION

COMMENTS by

S. LACASSE- (*Norwegian Geotechnical Institute*) about D. SHIELD'S paper

I would like to add to Dr. Don Shields' contribution on the use of pressuremeter in North America, by enumerating some of the institutions which he did not mention and which have studied the pressuremeter and its interpretation. The list is certainly not exhaustive, but the work done certainly illustrates that interest exists in the USA and Canada. Dr. Jean-Louis Briaud has helped me in drawing up this list.

In Canada :

- the National Research Council has worked with different pressuremeters ; the work by Eden and Crawford is noteworthy,
- tests run by Laval University (Prof. Tavenas, Roy),
- work done by Prof. Ladanyi (Ecole Polytechnique),
- the reports produced by Univ. of British Columbia (Prof. Campanella).

In the USA :

- different institutions doing research on the self-boring pressuremeter, include : Texas A & M University, University of Texas, Stanford University and soon Virginia Tech (Prof. Clough), University of Florida, the Corps of Engineers,
- Dr Denby, now at Golder and Assoc., still works with different pressuremeters,
- the Department of Transportation, a federal agency, sponsored research at MIT (Prof. C.C. Ladd) on the interpretation of the self-boring pressuremeters test. Prof. M. Jamiolkowski, from the University of Turin, became a consultant to this project. A Symposium was held in Washington in 1978.
- Texas A & M University offers a short course in pressuremeter testing,
- the Power and Light Company uses pressuremeter test results for the design of power lines.

I agree with Dr. Don Shields that in practice, the use of the pressuremeter in North America is limited but the good work done at the above mentioned institutions reflects the interest and the potential of the pressuremeter. It is up to us to work on the interpretation and the test procedure to render the technique attractive to the North American practice.

CONTRIBUTION

ASTM STANDARD ON PRESSUREMETER TEST.
J.L. BRIAUD- (*Texas A & M University*)

The ASTM Committee D. 18.02 on "Sampling and Related Field Testing" has formed subcommittee D. 18.02.07 on "The Pressuremeter". This subcommittee is charged with the responsibility of writing pressuremeter standards. A distinction is made between the self-boring pressuremeter and the preboring pressuremeter.

At the present time a standard is proposed for vote on the Suggested Practice for Preboring Pressuremeter Tests in Soils. Separately a standard is ready to be proposed for vote on the Suggested Practice for the Preparation of a Preboring Pressuremeter Test Bore Hole.

In the future standards related to self-boring pressuremeter testing will be addressed.

Committee D. 18.13 on "Marine Geotechnics" has started work on the possibility of writing is not a standard at least some guidelines for offshore pressuremeter testing.

COMMENT by

A. VAN WAMBEKE- (*Ecole Royale Militaire, Brussels*)

Concerning the Techniques Louis Ménard GE pressuremeter, which interprets test results automatically, it is advisable to consider the risks incurred by automatic interpretation of the pressuremeter test. The parameters E and p_L may thus be determined from an unsatisfactory test. It is essential for the engineer's critical approach to gauge the value of the pressuremeter curve itself.

M. GAMBIN- (*Etudes Pressiométriques Ménard*) in reply to
A. VAN WAMBEKE

The pressuremeter type GE as described by J. Leblanc will help improving the immediate processing of the readings. Presently, most of the time, the pressuremeter operator is not sufficiently qualified and there is no technician of the requested level at the site to perform such a processing. However this primary analysis is often required by the consulting engineer to adapt the scope of the soil investigation without loosing time.

Naturally it will always be possible to store the readings on a suitable material and send them to the offices.

SESSION **2**

Chairman : J. C. PARRIAUD

Laboratoire Central des Ponts et Chaussées, Paris, France

Self-boring pressuremeter

THE SELF-BORING PRESSUREMETER

ABSTRACT

The general principle of the self-boring technique is first given, distinguishing two types of method depending on whether the breaking up of the soil is carried out only by the injection of a fluid under pressure or by the action of a rotary tool.

A description is then given of some particular applications allowing the use of pressuremeters under given soil conditions.

The Model 76 (PAF 76) self-boring pressuremeter is then described in detail with its technology, operating procedure and the characteristics measured during the standard test called the "reference test".

The paper ends with an indication of some new application possibilities for the method in marine soil reconnaissance in shallow to medium water depths.

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THE SELF-BORING PRESSUREMETER

The self-boring technique is a soil reconnaissance method consisting in placing test and measuring instruments in the mass of the soil without disturbing it (reference 1).

Self-boring devices are designed on the principle shown schematically in Figure 1. They consist of thin-wall core samplers (1) carrying the test or measuring instruments on their sides (2). They are driven into the soil under the action of static or dynamic forces (3). As they are driven, the core which has penetrated into the sampler must be broken up and removed towards the surface through the inside of the device (4). The measurement can thus be carried out on a soil which has not undergone any lateral displacement and whose water content has not been modified.

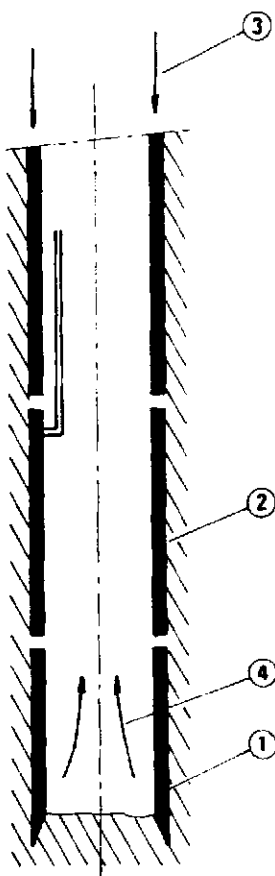


Figure 1 - Schematic diagram of self-boring apparatus

THE SELF-BORING PRESSUREMETER

We shall confine ourselves here to the application of the self-boring method to the pressuremeter test even though, for marine reconnaissance, other types of measurement by self-boring can be of interest : for example, permeability tests (reference 2) or density measurements (reference 3).

OVERVIEW OF THE SELF-BORING METHOD

In order to place a self-boring apparatus correctly in a soil, three actions must be carried out together :

- applying a driving force in order to overcome the resistance of the soil to the penetration of the cutting edge as well as skin friction forces,
- breaking up of the soil which penetrates into the cutting tool,
- remove to the surface the sediment thus created.

If the latter operation is not performed properly, soil clogging will result in the cutting tool. Penetration then continues with lateral displacement of the soil which is then disturbed and the measured properties are no longer those of the virgin soil.

Self-boring apparatus are classified into two families depending on the method used for breaking up the soil, either by "jetting" or by means of a chopping tool.

1. Self-boring by jetting

The soil is broken up only by the injection of fluid under pressure. This fluid can be water, air or both simultaneously.

The simplest apparatus designed according to this principle is the PAF 65, a schematic section of which is given in Figure 2. The cutting shoe (1) is fitted on the inside with adjustable nozzles (5) through which water is injected under pressure flowing through the flexible pipe (4). The sediment moves up through the inside of the probe body (6) and then through the inside of the drill string (3) which also used to apply the static or dynamic driving force.

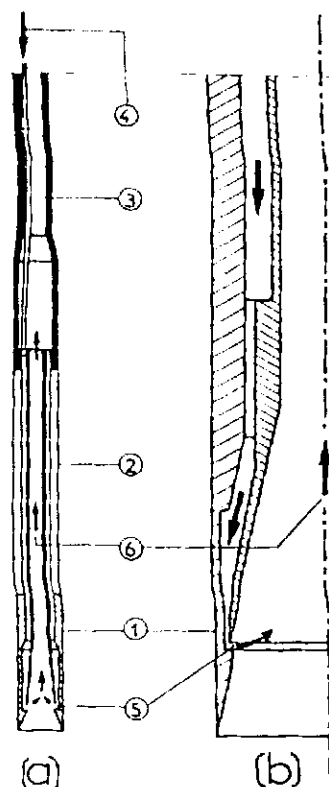


Figure 2 - Schematic diagram of Model 65 self-boring pressuremeter

This pressuremeter exists in three diameters (65 mm, 100 mm, 132 mm). The injection fluid flow which can be applied is limited by the section of the flexible pipe (4). This pressuremeter can consequently be used only in fine sandy or coherent soils of low compactness.

This difficulty has been overcome by means of the device developed by Noret (reference 4) shown schematically in Figure 3. This system is made up essentially of a double string of tubes running continuously from the probe to the surface :

- an exterior tube (1) of 100-mm diameter carrying the pressuremeter cell (2),
- a middle tube (3) with a smooth interior throughout its height.

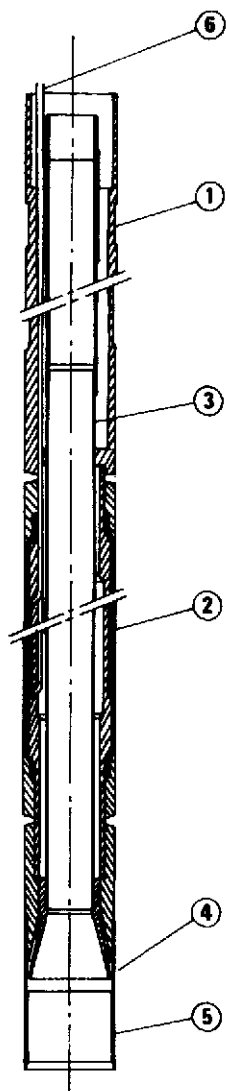


Figure 3 - Schematic diagram of air-injection type self-boring apparatus

The injection fluid is air under pressure. This air flows in the annulus between the two tubes and comes out through the nozzles (4) placed over the cutting edge (5). This annulus is sufficiently large to allow the air flows necessary, namely 8 to 12 m³ per minute under a pressure of 800 kPa.

THE SELF-BORING PRESSUREMETER

The inner diameter of the middle tube is 56 mm, allowing the removal of sediment in which the diameter of the largest elements can reach about 500 mm. The expansion and measurement lines of the pressure cell are also placed in the annulus between the two tubes. In spite of a certain assembly complexity, this system is very effective for the study of sandy soils. Its use for the testing of coherent soils is presently under study.

2. Self-boring with chopping tool

In this case, the soil is broken up by a special tool called a chopping tool rotated by either a drill string driven from the surface or by a motor built into the probe itself.

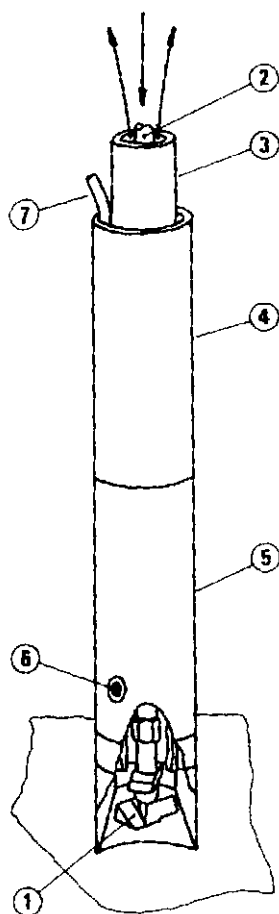


Figure 4 - The Camkometer

THE SELF-BORING PRESSUREMETER

in the apparatus developed by Professor Wroth, the Cankometer (reference 5), the chopping tool is rotated from the surface. A schematic view is given in Figure 4.

The chopping tool (1) is driven by a hollow middle rod (2) through which is injected water under pressure. This rod turns freely inside a tube (3) used for removing the sediment to the surface. There is also an outer tube (4) which carries the pressiometric cell (5) which may be equipped with a pore pressure tap (6). The pressiometric cell supply and measurement lines (7) run through the annulus between the two tubes.

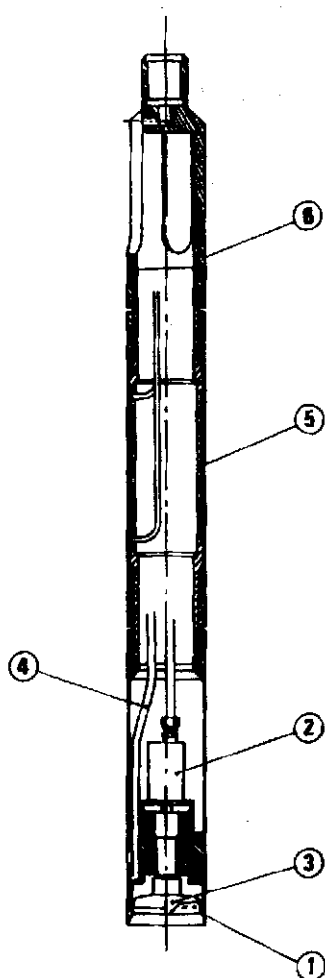


Figure 5 - Model 76 self-boring pressuremeter probe

THE SELF-BORING PRESSUREMETER

In the PAF 76 (reference 6) the chopping tool is rotated by a miniature hydraulic motor placed inside the cutting shoe (Figure 5). This apparatus is made up of an assembly of three modules :

- the self-boring module which includes the cutting shoe (1), the hydraulic motor (2) driving the chopping tool (3) as well as a pipe for the injection of water or drilling mud,
- the measurement module (5) which in the present case is a pressiometric cell,
- the driving module (6) which makes it possible to exert a driving force on the probe ; this module is screwed at its end to a drill pipe string.

This apparatus exists in two diameters : 100 and 132 mm. Its length is 1.25 m and its weight about 60 daN.

It makes it possible to test soft to very stiff coherent soils (cohesion up to 400 to 500 kPa) and fine sandy soils. It can also be used at great depths. The company Rodio of Milan was able to test stiff clays down to a depth of 60 meters.

This is the same principle upon which is based the design of the self-boring marine probe which will be described elsewhere during this symposium.

THE SELF-BORING PRESSUREMETER MODEL 76 (PAF 76)

Like any pressuremeter apparatus, the PAF 76 includes several parts (Figure 6) :

- the probe (1),
- the pressure-volume controller (2) referred to as the CPV,
- the different lines connecting the probe to the surface : the drill string (3), the flexible hydraulic lines supplying the motor (4) and the line (5) connecting the pressiometric cell (6) to the CPV (2).

THE SELF-BORING PRESSUREMETER

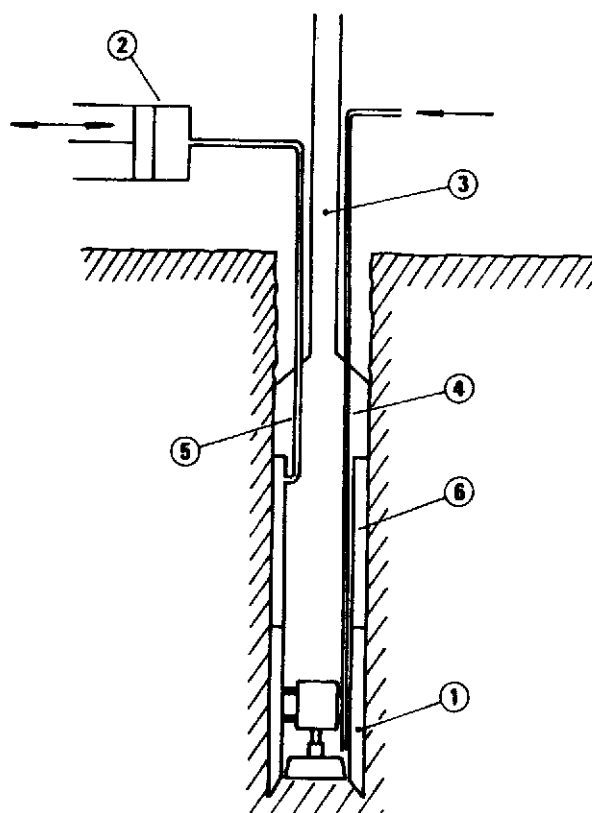


Figure 6 - Model 76 self-boring pressuremeter

The probe of the PAF 76

This is the probe described in the preceding chapter. It includes a single pressiometric cell protected by a lining of metal blades. This cell is expanded radially by the injection of water under pressure delivered by the surface CPV. It can be equipped with two electric pressure transducers : one giving the pressure in the cell (total pressure p), the other giving the pore pressure from a filtering surface placed on the wall of the pressuremeter in contact with the soil.

The pressure-volume controller (CPV)

The CPV is an adjustable-flow positive-displacement pump.

THE SELF-BORING PRESSUREMETER

This pump can be actuated either manually or by means of a low-voltage electric micromotor (24 volts d.c.). This device makes it possible to impart a volume variation rate to the pressiometric cell. The action of the soil is measured either by means of a precision pressure gauge incorporated in the CPV or by means of an electric pressure transducer incorporated in the probe.

The advantages of this type of CPV are significant : the automation of the test is possible simply, thus allowing in particular the completion of complex loading programs such as cyclic loadings. Remote control is also possible, and of particular interest for certain types of marine reconnaissance.

A brief description is given below of the succession of the operations necessary for the proper implementation of the PAF 76.

1. Preparation of equipment

These are operations customarily applied to any pressiometric test : assembly, saturation of circuits, measurement of probe resistance and calibration. The latter operation is particularly important : it consists in placing the wall of the pressuremeter exactly in the projection of the cutting edge.

If such were not the case, the system would not be correctly placed because there would be a lateral displacement of the soil prior to the expansion test.

2. Placing of probe on test level

This operation is thus carried out by self-boring, the measurement cell being kept at its calibration volume V_0 . In order for this operation to be conducted correctly, a certain number of self-boring parameters must be complied with, such as the water injection rate, the driving rate and the "clearance" between the end of the cutting edge and the chopping tool.

3. Relaxation phase

Once the test level has been reached, the self-boring operation is interrupted and the probe is held still. With the measurement kept at constant volume, a drop in pressure is observed with time (Figure 7). The soil relaxes. The pressure tends asymptotically toward a value p_0 which is the total horizontal soil pressure at rest.

THE SELF-BORING PRESSUREMETER

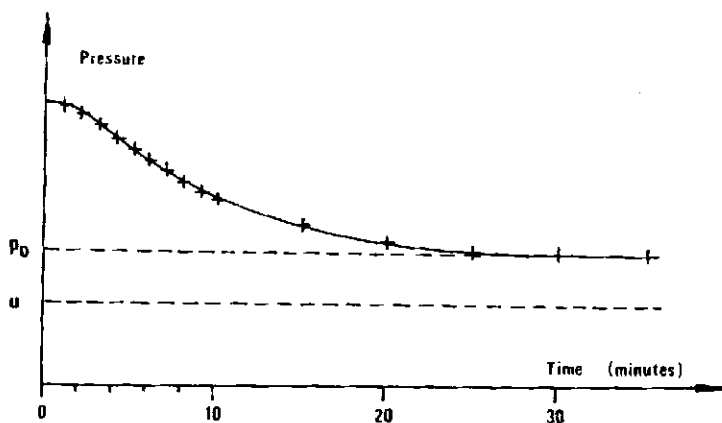


Figure 7 - Relaxation curve with self-boring pressuremeter

The normal relaxation time is of the order of an hour and a half. If the total pressure continues to drop after the period, this means that the probe is not correctly placed. It is then necessary to modify the self-boring parameters or possible to bring the probe back up to the surface for recalibration.

4. Expansion test

Once the speed has been suitably selected, the CPV is actuated and the measurement cell expands at a constant rate. At regular time intervals, the injected volume ΔV and the corresponding pressure p are noted. After the test, the measurement cell is reinflated by reversing the operation of the CPV motor.

The self-boring operation is then resumed until the new test level is reached, and so on.

THE PAF 76 : THE REFERENCE TEST AND THE SOIL PROPERTIES MEASURED

To minimize the scattering of data relative to the measured soil properties, a reference test has been defined. This test is standardized from the viewpoint of the self-boring procedure, probe geometry and in particular the geometry of the self-boring module (shape and surface index of cutting edge).

The loading program is also standardized : continuous expansion of the measurement cell at the deformation rate $\Delta V/V_0$ of 2 % per minute up to a maximum deformation of 25 %.

For this deformation rate, the test may be regarded as undrained in clays and drained in sands.

Figure 8 gives the corresponding pressiometric curve as well as the main properties measured. This curve is shown in the diagram ($\Delta V/V_0$) where ΔV is the volume variation of the measurement cell having an initial volume of V_0 , and p is the total soil contact pressure. It is observed that the curve has a monotonic concavity indicating that there was no decompression of the soil prior to the test.

This curve shows the pressures p_0 , p_2 , p_5 and p_{20} corresponding respectively to the deformations $\Delta V/V_0$ of 0, 2, 5 and 20 %. These pressures will be used for characterizing the soil and for calculating foundation bearing capacity.

One then calculates the shearing moduli G_{p0} (initial tangent modulus), G_{p2} and G_{p5} (secant moduli), for example according to the relationship :

$$G_{p2} = \frac{p_2 - p_0}{2} \times 100$$

These moduli are used for calculating the settlement of foundations. If necessary, Young's moduli can be determined according to the following well-known relationship :

$$E = 1 (1 + \nu) G$$

Where ν is the Poisson's ratio of the soil.

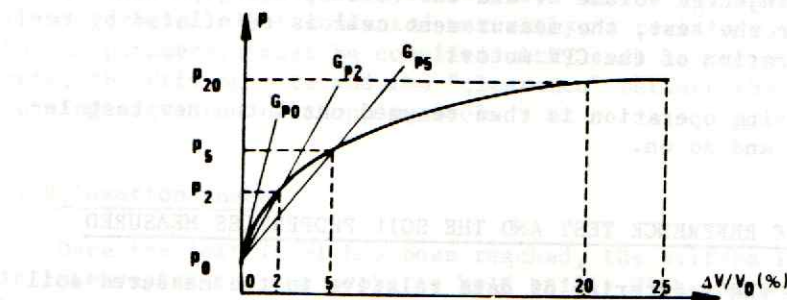


Figure 8 - Expansion curve with self-boring pressuremeter.
Definition of characteristic moduli and pressures.

THE SELF-BORING PRESSUREMETER

It is important to point out here that these properties differ from those measured by the Ménard test ; the softer the soil, the greater is this difference. These properties can consequently not be used in the rules given by Ménard. They have called for the establishment of specific rules which will be dealt with in another paper at this symposium.

The test also makes it possible to determine other characteristics, some of which are original :

- the soil identification coefficient β (Figure 9).

This coefficient is given by the relationship :

$$\beta = \frac{p_{20} - p_5}{p_{20} - p_0}$$

It has been shown (reference 7) that this coefficient depends on the nature of the soils, and it allows their identification by simply examining the expansion curve.

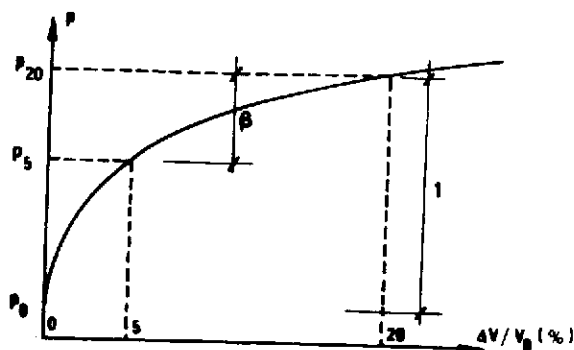


Figure 9 - Soil identification coefficient.

- Shear resistance properties of soils.

It has been demonstrated (reference 8) that in the case of undrained tests, the shearing resistance mobilized in the soil in contact with the probe is given by the relationship :

$$\tau = \frac{\Delta V}{V_0} \times \frac{dp}{d(\Delta V/V_0)} \left(1 + \frac{\Delta V}{V_0}\right)$$

THE SELF-BORING PRESSUREMETER

In this case, the first two terms represent the sub-tangent NT to the pressiometric curve (figure 10). The third term $(1 + V/V_0)$ is a "correcting" term which may be neglected with small deformations.

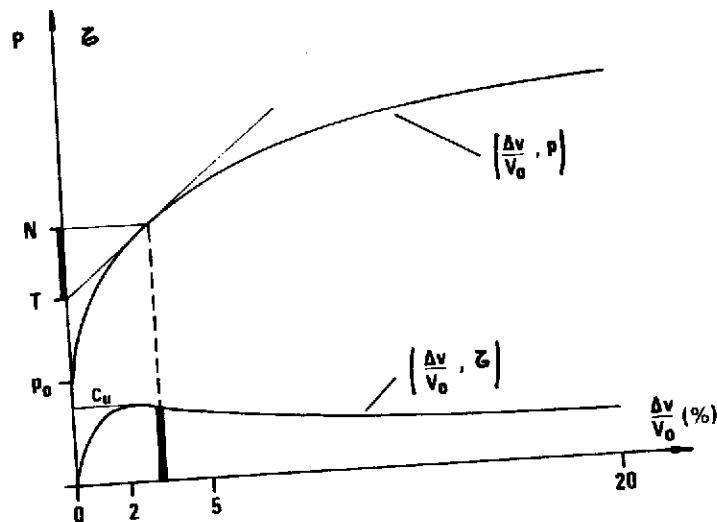


Figure 10 - The self-boring pressuremeter : expansion curve and derived shearing curve.

On the shearing curve it is thus possible to determine the undrained cohesion C_u as well as the tangent or secant shearing modulus G for different deformations.

In the case where the test is not strictly undrained, the preceding relationship leads to an apparent shearing resistance which - for example in the case of dilatant soil - is higher than the shearing resistance actually mobilized in the soil.

CONCLUSIONS

The self-boring method applied to the pressuremeter test makes it possible to measure in situ the properties of undisturbed soils necessary for selecting foundations and their dimensioning : soil identification, total soil pressure at rest, bearing capacity and settlement parameters.

THE SELF-BORING PRESSUREMETER

The method is particularly well suited to soft soils for which conventional methods are not effective, particularly in marine applications. However, owing to the development of new processes (jetting by air injection for example) it is now possible to extend its application to coarse sandy soils which are relatively compact.

For its application to marine reconnaissance, several cases must be distinguished :

- for large water depths, specially designed facilities are required : such is the case of the self-boring marine pressuremeter which will be discussed during this symposium ;
- for small depths (no more than a few tens of meters), equipment such as the PAF 76 can be combined without difficulty with the facilities currently used in this case, provided of course that the drilling and handling equipment is not subjected to any wave action (any heave of the probe would cancel the advantages of the self-boring technique) ;
- for medium water depths, adaptation studies are required (and are moreover being carried out) in order to arrive at a technically and economically acceptable method.

THE SELF-BORING PRESSUREMETER

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F. BRUCY*, J. B. FAÏ*, J. MARIGNIER**

OFFSHORE SELF-BORING PRESSUREMETER : THE LPC/IFP PROBE

ABSTRACT

Soil investigation is an operation of prime importance for the design and engineering of offshore petroleum facilities. This consideration has led the Institut Français du Pétrole to develop the offshore self-boring pressuremeter (PAM) which has the advantage of performing thorough and wellcontrolled in situ geotechnical tests.

The offshore self-boring probe is derived from the self-boring pressuremeter developed by the Laboratoires des Ponts et Chaussées. It has the following dimensions : length of 6 m, nominal diameter of 160 mm. It is made up of the stacking of three modules ; the self-boring module, the pressuremeter itself, and the driving module. The probe is operated from a control cabin where the different functions are remote controlled.

Trials of the probe totalize nearly 300 meters of borehole length and more than 100 pressuremeter tests in different types of soil, i.e. soft clay, stiff and plastic clay, fine sand and sandy silt. The tests have proven the reliability of the probe. All the results and especially the logs of the different self-boring parameters have confirmed the quality of the implementation of the probe. Pressuremeter measurement analyses have led to an evaluation of the influence of the procedural options and have justified them. In return, the data acquired by the offshore self-boring pressuremeter during field experiments are reliable interpretation references for future geotechnical investigations surveys using the PAM.

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OFFSHORE SELF-BORING PRESSUREMETER : THE LPC/IFP PROBE

INTRODUCTION

Soil investigation is an operation of prime importance for the design and engineering of offshore petroleum facilities. It shows specific aspects that are linked to the difficulties involved in working at sea and the depth of the water to be crossed through. Likewise, offshore foundations design features must be taken into consideration. Offshore soil investigation surveys make use of different methods. The most widely used ones are drilling and sampling techniques. The samples are tested in the laboratory aboard the ship and especially ashore. However, in situ tests are getting more and more emphasized. For example, one such test is the cone penetration test. With regard to the pressuremeter test, this has given extensive proof of its viability onshore in France for twenty years. The self-boring process developed by the Laboratoires des Ponts et Chaussées (LPC) has resulted in the highly fruitful development of the method (1).

To help improve offshore soil investigation techniques, the Institut Français du Pétrole, with the assistance of LPC, has undertaken the adaptation of the self-boring pressuremeter for practical offshore use and has developed the offshore self-boring pressuremeter (PAM) (2), (3), (4) and the LPC/IFP self-boring probe. The offshore self-boring pressuremeter meets offshore soil investigation requirements because it has various advantages compared with conventional methods :

- it performs in situ measurements, which is of particular interest offshore ;
- it makes a complete geotechnical test ;
- it gives meaningful characteristics because self-boring provides an opportunity for careful control of the stress and strain fields in the soil.

1. DESCRIPTION OF THE PROBE

The operational criteria of the offshore self-boring pressuremeter (PAM), such as the implementation principle, the depth of the water column to be crossed through, etc., have been used as a basis for choosing the dimensions of the probe, the control principles and the options concerning test procedures. Likewise, the control principles that have been retained make it possible to monitor all boring

OFFSHORE SELF-BORING PRESSUREMETER : THE LPC/IFP PROBE

parameters while the probe penetrates into the ground. The corresponding logs make up a set of data which is of great interest and which add to the pressuremeter results themselves.

1.1. Dimensions and characteristics of the probe

The probe is derived from the PAF 76 self-boring pressuremeter developed by the Laboratoires des Ponts et Chaussées. It is about 6 meters long and 160 mm in diameter. It is made up of the stacking of three modules (fig. 1) : the self-boring module, the measuring or pressuremeter module, and the driving module.

The self-boring module enables the probe to penetrate into the soil without being rotated and especially without either decompressing or displacing the formations crossed through. The soil is cut through by a cutting edge that is beveled toward the inside (fig. 2). A slight oversize of the cutting edge (1 %) beyond the nominal diameter has proven to be of great interest (Cf. § 3.1.1.). A grinder with toothed blades (rotating at 230 rpm) breaks up the core of sediments, and the cuttings are evacuated by water circulation (140 l/mn) inside the probe (fig. 2).

The pressuremeter consists of a single dilatable cylindrical cell with a diameter of 160 mm and a length to diameter ratio of 2. Dilation can attain 25 % of the initial volume.

The driving module exists in two variants :

- drill collar weighing 8 kN which is used in soft soils ;
- a packer combined with a jack capable of supporting 35 kN by dilating inside the borehole (fig. 1). It can be used to jack in the probe by sequences (fig. 3) and is used for stiff clays and dense sands. The rate of jacking is selected by the operator at between 5 and 35 cm/min.

1.2. Specific equipment of the probe

The equipment required for implementing the probe is attached to a seabed frame (fig. 4). It mainly includes a water circulation pump and two constantly-adjustable positive-displacement pumps controlling the dilations of the pressuremeter and packer, i.e. the pressure and volume controls (CPV). The CPV with respective capacities of two and ten liters are equipped with high-precision pulse counters.

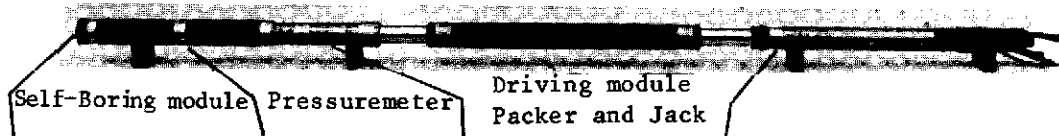


Fig. 1 - Offshore Self-Boring Pressuremeter, the LPC/IFP Probe.

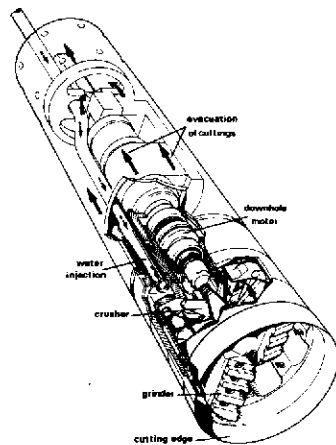


Fig. 2 - Self-Boring Module.

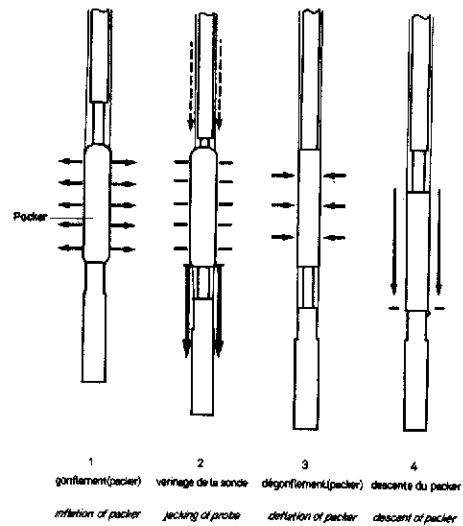


Fig. 3 - Sequences for advance of Probe with Packer and Jack.

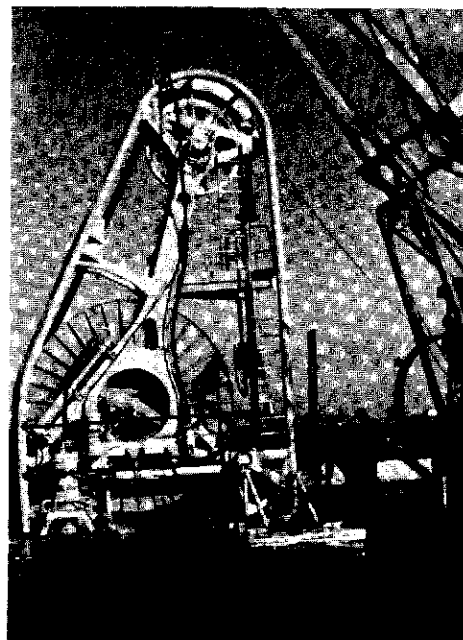


Fig. 4 - Offshore Self-Boring Pressuremeter, the Sea-Bed Frame

OFFSHORE SELF-BORING PRESSUREMETER : THE LPC/IFP PROBE

1.3. Operating and controlling the probe

The probe is operated from a control cabin on the surface where the different hydraulic and mechanical functions are remote controlled, in particular :

- water injection at the level of the cutting edge (Cf. fig. 2) ;
- rotating the grinder (also Cf. fig. 2) ;
- dilating and deflating the pressuremeter ;
- dilating and deflating the packer (Cf. fig. 3) ;
- jacking in the probe and descent of packer (Cf. fig. 3).

The instrumentation of the probe feeds data to a computer so it can control operations by entirely automated sequences. This instrumentation enables the following measurements to be performed :

- position of the probe in the borehole ;
- holdback tension of the probe ;
- resistant torque on the grinder ;
- water injection pressure ;
- volume variation of the packer, and pressure ;
- position of the jack stem ;
- jack pressure ;
- and finally, variation in pressuremeter volume, and pressure.

Two electronic devices, one on the seabed frame and the other in the control cabin, centralize and process the commands and measurements that are transmitted by multiplexing via the electric umbilical cable linking the seabed frame to the surface control cabin.

2. EXPERIMENTING THE PROBE

The offshore self-boring pressuremeter and probe were experimented during different workshop tests as well as during various tests in the field. A great many boreholes have been achieved since 1979 on different sites onshore, and then, at sea.

OFFSHORE SELF-BORING PRESSUREMETER : THE LPC/IFP PROBE

2.1. Trials in soft clay

Field tests carried out at the site of Cran (Brittany, France) were performed to operate the probe on an already known site in order to test its different components during a real operation. The identification properties of Cran clay are given in Figure 5. Two campaigns (1979 and 1981) included a total of ten boreholes providing complete profiles for different configurations of the probe, i.e. regular or 1 % oversized cutting edge, drill collar or jack-packer, totaling 140 meters of length penetration and 65 pressuremeter tests. The role of the different boring parameters, the behavior of the dilated packer and the nature of the pressuremeter results were analyzed. Operating the probe in medium-consistency fine clay sediments proved to be entirely satisfactory.

2.2. Trials in a stiff and plastic clay

Field tests were carried out at Winnezele (northern France) for the purpose of testing the operating of the PAM probe in a stiff and especially plastic clay. Its identification characteristics are given in Figure 6. Two campaigns (October 1979 and April 1980) provided an opportunity to analyze the extremely important problems of the evacuation of cuttings and of packer resistance. The proper design of the grinding system and the sediment return circuit proved to be quite positive. Resistance of a packer having a single reinforced rubber membrane banded at each end proved to be satisfactory.

2.3. Trials in fine sand

Field tests were carried out at Moidrey, near Mont Saint Michel, France (March 1980), to check the performances of the probe in fine sandy soil and in particular to perform various tests of the packer.

2.4. Trials in silt (port of Le Havre)

The tests performed in a dock in the port of Le Havre had the goal of checking the reliability of the equipment underwater. A total of five boreholes were achieved, amounting to 43 meters of length penetration and 30 pressuremeter tests during about 50 hours of subsea operating. These tests served to complete, for silty sediments,

OFFSHORE SELF-BORING PRESSUREMETER : THE LPC/IFP PROBE

the results obtained with the probe during the preceding test campaigns.

3. TEST RESULTS WITH THE PROBE

The field tests with the probe had the dual advantage of enabling the equipment to be further developed and tuned up and of carrying out complete operations providing results that can be construed as reference data for interpreting. A borehole operation carried out with the PAM probe bring back a great many geotechnical data which are of two kinds : (i) self-boring logs, and (ii) pressuremeter logs.

3.1. Self-boring logs

Self boring logs are variations in the main boring parameters during penetration of the PAM probe into the ground. They are :

- the driving force, Q ;
- the pressure inside the undilated pressuremeter, p_i ;
- the resistant torque on the grinder, Cr ;
- the circulation fluid pressure, p_w (seawater).

Measurements are recorded every 10 cm of penetration and are printed out in real time.

3.1.1. Driving force

The force required to drive in the probe is a parameter that was carefully analyzed to determine its variations as a function of experimental conditions because its level governs the feasibility of the test, given the capacities of the two driving modules described above (drill collar and jack-packer). The forces measured were mainly influenced by the geometry of the cutting edge used.

The use of 1 % oversized cutting edge considerably modified forces Q compared to the ones obtained with a regular cutting edge because of the decrease in lateral friction. Tests effectively showed a reduction of about 50 % in Cran clay and 30 % in Winnezelee clay. A comparison with test results involving the PAF 76 developed by LPC also shows that the soil reaction (i.e. Q) depends on the

OFFSHORE SELF-BORINGPRESSUREMETER : THE LPC/IFP PROBE

depth	soil	W	W _L	I _p	Cu vane
0 to 3.5 m	soft plastic clay	110%	120%	80%	60 to 15 kPa
3.5 to 18 m		50 to 80%	60 to 100%	30 to 60%	15 to 60 kPa

Fig. 5 - Basic geotechnical data for Cran site.

depth	soil	W	I _p	C (triaxial)
0 to 10m	highly plastic clay	40 to 30%	60%	50 to 90 kPa

Fig. 6 - Basic geotechnical data for Winnezeele site.

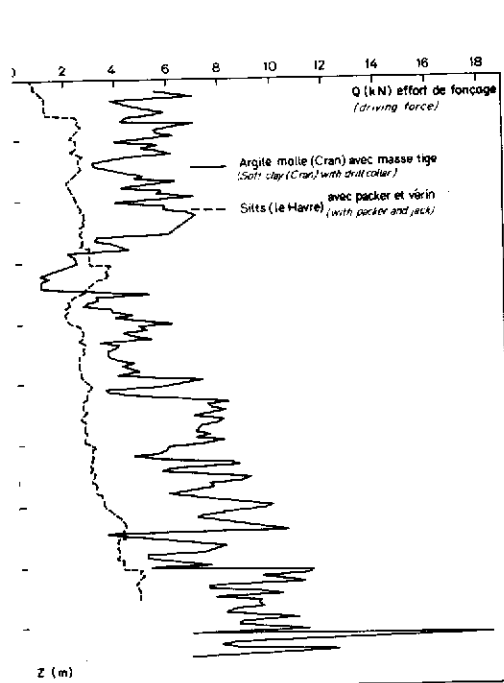


Fig. 7 - Driving force logs.

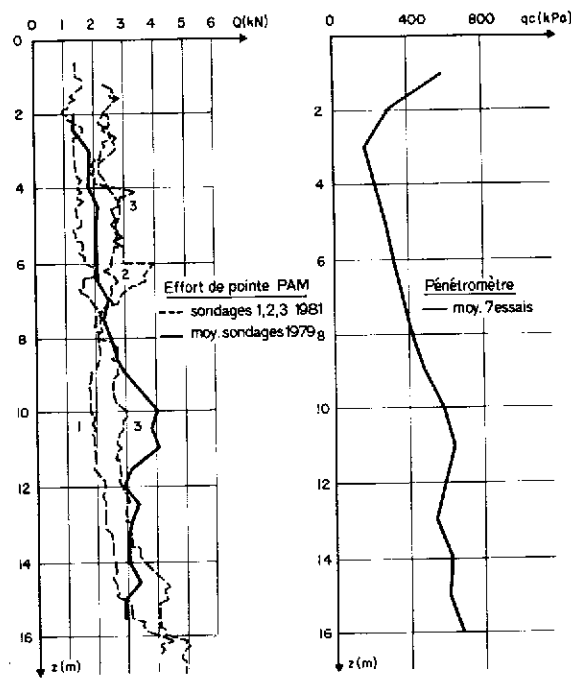


Fig. 8 - Comparison of driving forces and cone resistance at Cran site.

OFFSHORE SELF-BORING

overall geometry :
surface coefficient
driving forces. It
causes an increase

- an increase
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OFFSHORE SELF-BORING PRESSUREMETER : THE LPC/IFP PROBE

overall geometry of the cutting edge, i.e. overall cross-section and surface coefficient. The self-boring quality governs the level of driving forces. Indeed, the accidental displacement of the sediment causes an increase in the effort on account of :

- an increase in the tip cross-section because of a sediment plug or a trapped pebble ;
- an increase in lateral friction on the probe (despite the use of an oversized cutting edge) ;
- eventually, an increase in water circulation pressure so as to produce a piston effect.

On the other hand, a decompression of the soil underneath the tool (e.g. caving in loose sand) reduces the tip force. Sediment grinding and evacuation systems thus have to be carefully designed for providing a good quality of self-boring. The use of a longer cutting edge is planned in granular soil.

Under normal operating conditions, boreholes give access to driving force profiles which are directly representative of the soil (fig. 7) and are useful for effective interpreting. The driving forces vary according to the nature and stiffness of the formations crossed through and can be used to detect any eventual heterogeneity of the subsoil. Table I sums up various data.

TABLE I

Driving forces obtained with the PAM probe

Soil	Driving force (1% oversized cutting edge)
soft clay Cu = 35 kPa	3 kN
stiff clay Cu = 70 kPa	6 kN
clayey silt	2 kN
sandy silt	6-12 kN
fine sand	10-20 kN

OFFSHORE SELF-BORING PRESSUREMETER : THE LPC/LFP PROBE

It is interesting to compare the driving force profiles of the probe with other geotechnical measurements also linked to the failure properties of the soil :

- cone resistance R_p obtained with a static penetrometer ;
- pressure p_{20} of the pressuremeter characterizing the limit state of the soil.

A direct comparison of R_p and Q (fig. 8) requires a fictional tip area to be determined for the probe which takes into consideration the overall geometry of the cutting edge. Figure 9 shows the variations of Q as a function of $p_{20}-p_0$, for all of the tests performed to date with the probe. This result could be made more specific by measurements on other sites, in particular on ones having high pressures p_{20} .

3.1.2. Pressure in the undilated pressuremeter

The pressure existing inside the pressuremeter cell, p_i , is a very important parameter which can be used to judge the quality of the self-boring being done because it depends directly on the state of the borehole walls. Hydrostatic pressure can be compared with the horizontal stress of soils at rest (calculated). The influence of boring conditions on pressure p_i were investigated. The use of an oversized cutting edge produces pressures p_i very similar to the hydrostatic pressure because of the lateral decompression of the soil around the pressuremeter. On the other hand, the accidental displacement of the sediment outside the cutting edge produces a pressure rise inside the pressuremeter. This phenomenon is generally accompanied by an increase in the driving force.

Under normal operating conditions with a regular cutting edge, measurements of p_i provide a profile that gives a good indication of the total horizontal stress in the formation (fig. 10).

3.1.3. Resistant torque on the grinder

The resistant torque on the grinder is a parameter which enables the operator to be constantly aware of the operating conditions of the tool in the self-boring module. Under normal conditions, measurements are extremely stable. An increase in the torque indicates the blocking of the tool by a pebble (larger than 2 cm) that has not been broken up. An increase in the driving-in force then confirms that the borehole has reached the bedrock.

OFFSHORE SELF-BORING PRESSUREMETER : THE LPC/IFP PROBE

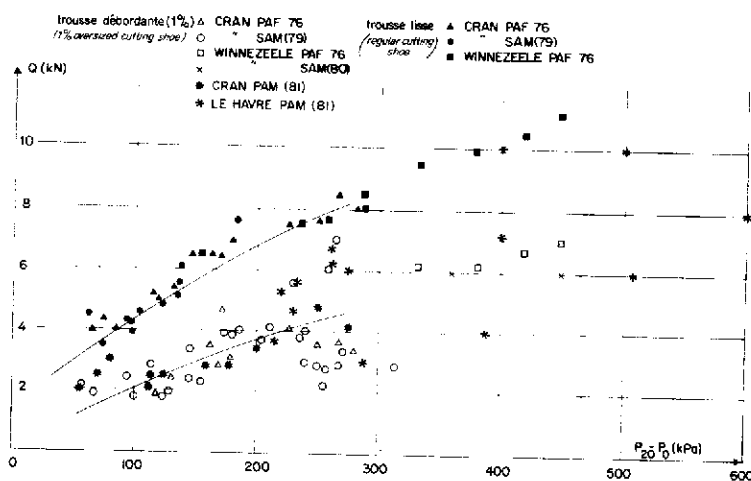


Fig. 9 - Driving force variations versus dilation pressure $P_{20} - P_0$.

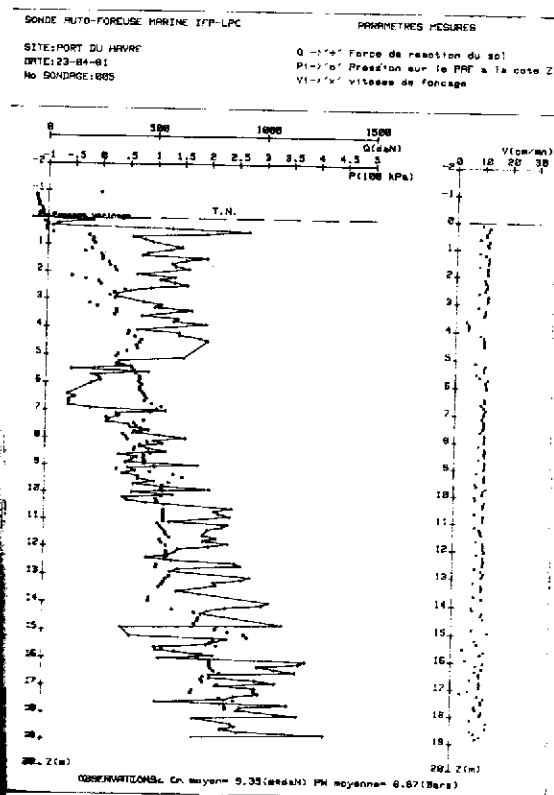


Fig. 10 - Example of Self-Boring parameter log.

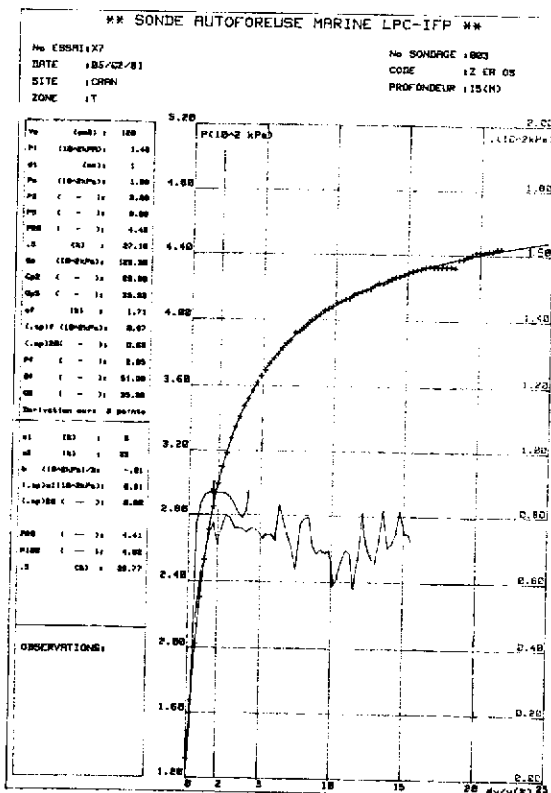


Fig. 11 Typical interpreted Pressuremeter log.

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3.1.4. Circulation Pressure

The water circulation pressure constantly indicates the sediment evacuation conditions via the inside of the self-boring probe. These conditions mainly influence the proper operating of the probe. They govern the level of driving forces to be developed and, what is essential, the quality of the self-boring achieved. Indeed, deficient evacuation may result in both the formation of a sediment plug at the end of the probe and the consequent increase in the peak force as well as the displacement of sediments outside the cutting edge which causes compression and remolding of the soil. Such incidents result in an appreciable increase in circulation pressure, and this may deteriorate even further the quality of self-boring as a result of a piston effect on the tip (Cf. driving force), or fracturing of the formation as observed by Clough and Denby (6). The need to guarantee the quality of self-boring required that particular care be taken in developing the sediment evacuation function inside the probe. Under normal operating conditions, circulation pressures thus remain extremely stable.

3.2. Pressuremeter logs

The borehole schedule includes a certain number of pressuremeter tests, generally every meter or every two meters. The criteria for implementing the probe offshore justified various options concerning the operational procedures :

- possible use of 1 % oversized cutting edge ;
- limiting the relaxation time of the cell to one or two minutes before its expansion ;
- volumic dilation of the pressuremeter by 5 % per minute.

The pressuremeter logs obtained by the probe can be interpreted to determine the pressuremeter parameters as well as the shear curve of the soil which interest the geotechnical engineers (fig. 11).

3.2.1. Initial pressure p_0

The pressure p_0 existing inside the pressuremeter cell at the time when it just comes into contact with the soil is an important measurement because it characterizes the at rest state of the soil. It is read on the pressuremeter log. For a probe using a regular cutting edge, it is the departure pressure for the test. For a probe using 1 % oversized cutting edge, the curve shows an inflection point giving pressure p_0 and the volume deformation $(\Delta V/V)_0$ for achieving contact with the soil.

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Two types of behavior have been observed. In clay, p_0 represents a good estimate of the horizontal stress of soils at rest. Indeed, $(\Delta V/V)_0$ is evaluated correctly in penetration performed with an oversized cutting edge since the borehole walls are selfstanding. In sands and silts, p_0 measures a certain decompression of the soil after the passage of an oversized cutting edge because the borehole tends to close in. A relaxation time of a few minutes serves to obtain a p_0 which represents a good estimate by defect of the horizontal at rest stress of soils.

3.2.2. Dilatation pressures p_2 , p_5 , p_{20}

The pressures measured for, 2, 5 and 20 % are noted p_2 , p_5 and p_{20} . Some scattering of measurements is observed for p_2 and even for p_5 , probably due to the influence of the initial state of the borehole (e.g. effect of an oversized cutting edge). Measurements of p_{20} are generally quite homogeneous (fig. 12). Pressure p_{20} is of particular importance because it is a parameter which characterizes the limit state of the soil, for any self-boring probe. Measurements performed with the offshore self-boring pressuremeter and the PAF 76 are exactly the same (fig. 12), so that the eventual use of corresponding designing methods can be considered (1).

3.2.3. Moduli G_0 and G_{p_2}

The most interesting pressuremeter modules are the initially tangent module G_0 and the secant module G_{p_2} defined by $G_{p_2} = (p_2 - p_0)/0.02$. The use of an oversized cutting edge in soft clay brings about a 50 % reduction in G_0 , whereas G_{p_2} remains the same. Furthermore, the results are homogeneous and are comparable to those obtained with the PAF 76. The problems of borehole behaviour nonetheless explain a certain dispersion in the case of granular soil (fig. 13).

3.2.4. Soil identification coefficient β

The coefficient β calculated from pressuremeter curves can be used to identify the nature of soils according to Table II below (5).

Figure 14 gives examples of results obtained with the PAM probe. They effectively correspond to the lithological description that was already known for the test sites.

Fig. 12 - Profile of the dilation pressure p_{20} at Cran site.

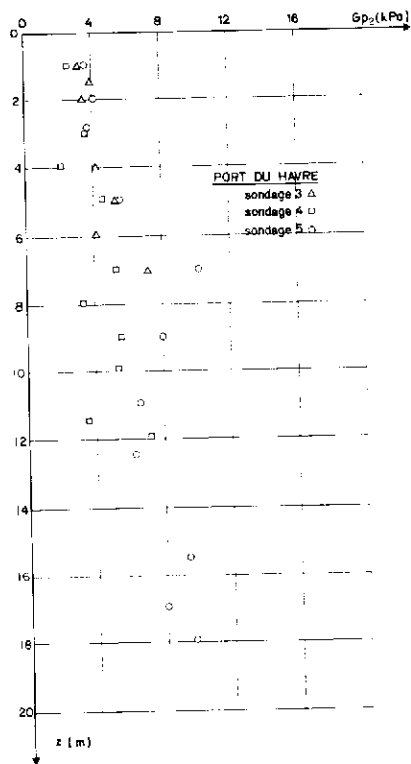
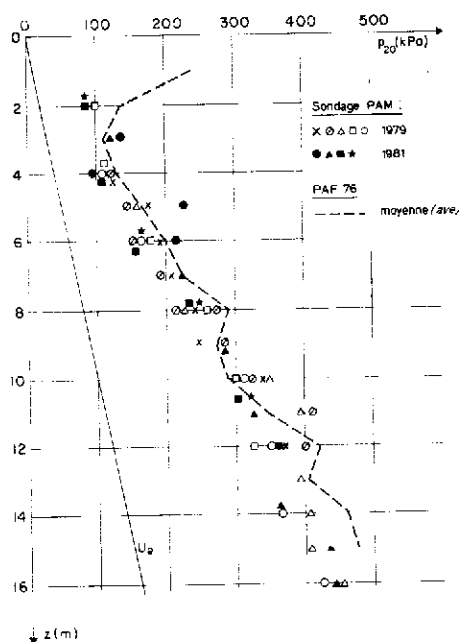


Fig. 13 - Example of modulus G_{p2} value profile.

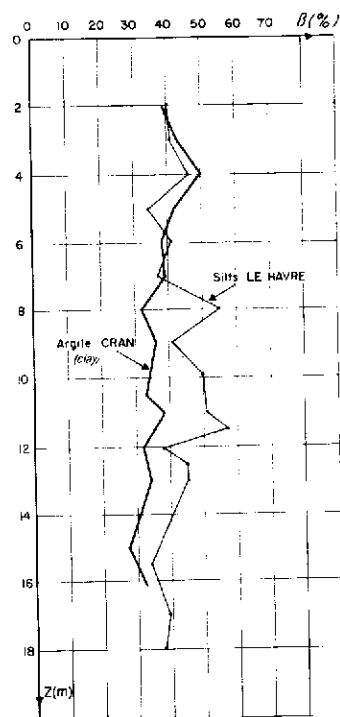


Fig. 14 - Identification coefficient β logs.

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3.2.5. Shear

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OFFSHORE SELF-BORING PRESSUREMETER : THE LPC/IFP PROBE

TABLE II

Soil identification by β

$\beta = (p_{20} - p_5)/(p_{20} - p_0)$	Nature of soil
$\beta < 25$	very sensitive clays
$25 < \beta < 40$	clays
$35 < \beta < 45$	compressible silts and sands
$45 < \beta < 60$	dense sands
$\beta > 60$	very dense sands

3.2.5. Shear strength C_u

The shear curve for an undrained cohesive soil under stress is analytically deduced from the pressuremeter curve. Computing is extrapolated to the case of silts and sands. The choice of the shear-strength value C_u is guided by a curve-smoothing method which must be as objective as possible. Measurements performed with the probe have proven to be very faithful and generally comparable to those obtained with the PAF 76. However, the results must be interpreted as a function of the geometry of the cutting edge because the existence of a remolded zone resulting from the use of an oversized cutting edge leads to an over estimation of shear strength.

CONCLUSIONS

The offshore self-boring pressuremeter has been designed to meet the needs of offshore soil investigation. Tests of the PAM probe amount to nearly 300 meters of borehole length and more than 100 pressuremeter tests which have given proof of its perfect reliability.

The overall results and especially the logs of the self-boring parameters have confirmed the qualities of the implementation of the PAM probe. An analysis of the pressuremeter measurements has served to evaluate the influence of procedural options and to justify them.

OFFSHORE SELF-BORING PRESSUREMETER : THE LPC/IFP PROBE

In return, the data acquired by the offshore self-boring pressuremeter during field trials provide reliable interpretation references for future geotechnical investigation surveys.

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BRITISH EXPERIENCE WITH THE SELF-BORING PRESSUREMETER

ABSTRACT

Research and development of the self-boring pressuremeter began in the UK at Cambridge in 1970. Since 1978 the instrument has been used on a regular basis in commercial engineering practice as an instrument for the in situ measurement of basic soil properties. This paper describes briefly some recent findings regarding the optimum setting of the cutter in order to minimise disturbance, and reviews the experience gained by British engineers in the use of the instrument in design in geotechnical engineering. Examples of pressuremeter tests in clay and sand at one typical site at Zeebrugge are presented. Emphasis is placed on the use of the pressuremeter for the measurement of the shear modulus of the ground.

I. INTRODUCTION

The purpose of this paper is to provide a brief review of the experiences to date of the development and use of the Self-Boring Pressuremeter (SBP) in the United Kingdom. The technique of self-boring as a means of inserting a cylindrical instrument into the ground was first used in Britain in 1971, by the Soil Mechanics Group of the University of Cambridge and initiated by Hughes (1973). The need for minimising the disturbance which is inevitably caused to the surrounding soil as any instrument is inserted into the ground, arose from efforts to measure as accurately as possible the in situ lateral stresses, with particular application to the design of earth retaining

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structures. The initial research work was confined to the design and development of a stiff load cell for direct measurement of the total horizontal stress, σ_{h_0} (reported verbally at the 5th European Conference on Soil Mechanics and Foundation Engineering at Madrid, 1972); but as the work progressed it became clear that the use of a self-boring pressuremeter would prove to be as effective, with the great advantage of allowing other soil properties to be measured in addition to σ_{h_0} .

The research work on the SBP developed into a major programme at Cambridge, generously funded by the Department of the Environment through the Geotechnics Division of the Building Research Station. The financial support consisted of a sequence of three 3-year research contracts. The work has been fully reported in the Ph.D. theses of four research students, Hughes (1973), Windle (1976), Clarke (1981) and Fahey (1980), and in many other publications.

In 1972 a patent was taken out on the SBP in the UK for the purposes of protecting and controlling its development and use. As a result of its early promise and success, and interest shown by research organisations in other countries, arrangements were made in 1974 for manufacture of the Cambridge self-boring pressuremeter under licence by a new company, Cambridge Insitu, established at Little Eversden, near Cambridge.

Demand was also growing at that time from British firms of consulting engineers for the SBP to be used, or at least tried out at sites where there were special problems. Because this demand could not be met by a research team working within a university environment, a new firm, PM Insitu Techniques Ltd., was created in 1978 to provide a service of pressuremeter testing on a commercial basis.

II. DEVELOPMENTS IN THE USE OF THE PRESSUREMETER

The pressuremeter was first conceived by Ménard in 1954, as a tool which was inserted into a preformed borehole and used for the measurement of the strength and stiffness of soils and rocks. Since Ménard's pioneering work there have been many major developments in the pressuremeter, especially in the last 15 years, which can be conveniently divided into four categories: (i) techniques of measurement, (ii) techniques of insertion, (iii) new analyses of the expansion test, and (iv) new types of test.

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2.1 Techniques of measurement

In the basic expansion test of the pressuremeter, measurements are made of the pressure and of the increase in volume of the inflatable membrane. In the early equipment used by Ménard these observations were made at the ground surface by monitoring the pressure and volume of the fluid used for inflation. With the development of accurate and reliable electrical transducers placed within the pressuremeter it has been possible to improve the accuracy of these measurements by an order of magnitude or more. For example the feeler gauges incorporated in the Cambridge SBP are sensitive to movements of the order of 0.005 mm, which is equivalent to less than 0.02% volumetric strain for an instrument with a diameter of 83 mm (which is the case of the latest version).

2.2 Techniques of insertion

It is presumed that the purpose and details of the self-boring technique are well understood. In order to minimise the disturbance caused by insertion, various studies have been carried out of the degree of disturbance created by different modes of operation and design of equipment. It turns out that the geometrical arrangement of the cutting shoe and cutting tool is crucial. Recent work by Clarke (1981) is discussed later in this paper, and suggests why the push-in-pressuremeter may prove to be a successful compromise for offshore use between the complex SBP and the cruder Ménard instrument.

2.3 New analyses of the expansion test

Initially, the results of pressuremeter tests were interpreted by means of empirical expressions to give parameters for design such as allowable bearing capacity factors and moduli for allowable settlement. The first fundamental interpretation of an expansion test was supplied by Gibson and Anderson (1961) in which the pressuremeter was considered to be infinitely long so that the deformation of the surrounding soil was assumed to be in conditions of axial symmetry and plane strain. One analysis was applicable to undrained expansion tests in clay in which the soil is assumed to undergo zero volume change and to behave as an elastic/perfectly plastic material characterised by a shear modulus G and an undrained shear strength c_u . A separate analysis was achieved for expansion tests in cohesionless soils for which the soil is assumed to behave elastically until failure occurs at a constant effective stress ratio (governed by the Mohr-Coulomb criterion in terms of a friction angle ϕ') and with no volume change.

Both of these analyses have been improved. The undrained expansion test can now be interpreted with no prerequisite assumption of stress-strain properties of the soil being tested as a result of work independently done by Baguelin et al (1972), Ladanyi (1972) and Palmer (1972). Unfortunately the interpretation is very sensitive to the disturbance of the soil caused by insertion, and to the datum selected for the strain.

The second case, the drained expansion test in a cohesionless material, has been modified by Hughes, Wroth and Windle (1977) to take account of the volume change that occurs after failure has been initiated. The relevance of this new analysis has been confirmed by Fahey (1980) by a special series of laboratory tests under carefully controlled conditions.

2.4 New types of test

To date most pressuremeter tests have consisted of a rapid expansion test either conducted by means of stress increments applied at regular intervals, or under constant rates of strain. With the advent of microprocessors and computer controlled experiments, as well as additional observations, e.g. pore-pressure measurements, it is now possible to carry out new types of test. One such possibility is a "holding" test for the purpose of measuring in situ the consolidation characteristics of the soil being tested. Clarke, Carter and Wroth (1979) report the results of such holding tests in which a quick undrained expansion test in a clay to, say, 10% strain, is followed by a phase when the membrane is kept automatically at its (enlarged) radius by adjustment of applied pressure. During this phase the clay undergoes consolidation with a reduction in the total radial stress and excess pore pressure, both of which are continuously recorded. From the results the coefficient of consolidation can be deduced.

III. OPTIMUM POSITION OF THE CUTTER

Recent work, which has an important bearing on the success of SBP tests, has been a careful investigation by Clarke (1981) of the influence of the cutter position on the amount of disturbance that is caused. The problem is best described in qualitative terms by means of Fig.1.

Fig.1(a) is a schematic section through a thick-walled cylindrical instrument, with sharp cutting edge, at a stage during insertion into the ground by a steady jacking process. Soil will enter the

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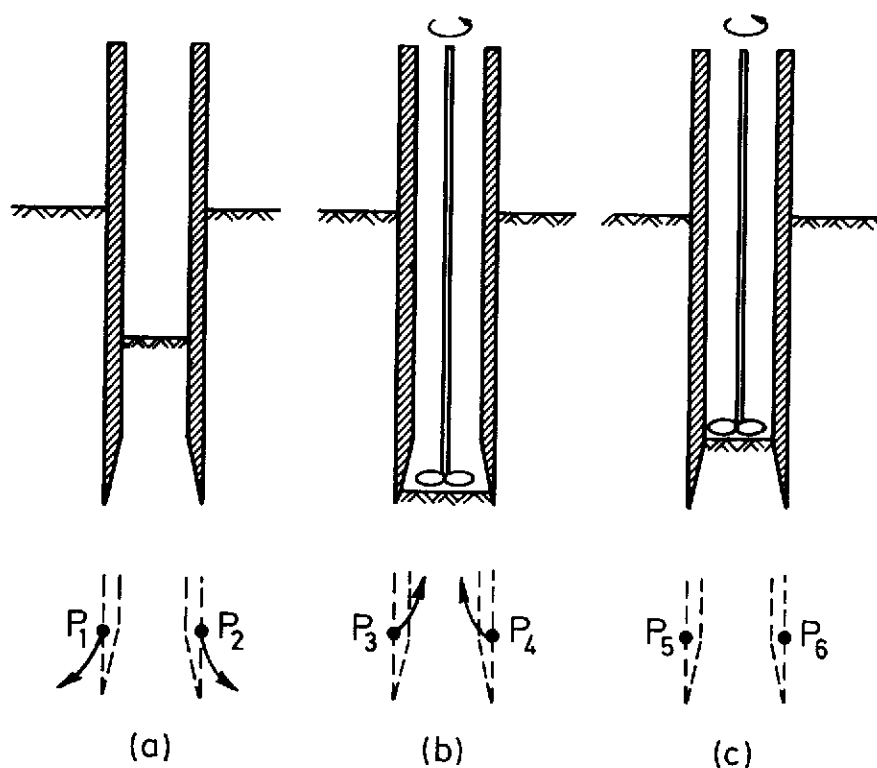


Fig.1 Effects of cutter position on degree of disturbance

instrument until a plug is formed, so that in effect it becomes a closed-ended cylinder. The influence on elements of the ground P_1 and P_2 which are ahead of the cutting shoe is an increase in the level of stress, with the movements of the elements being radially outwards and downwards as indicated.

In contrast, if an internal cutting device is introduced into the instrument right at the bottom, Fig.1(b), so that it drills a hole as the instrument is advanced, the overall effect is a reduction in stress level. Soil elements such as P_3 and P_4 will move radially inwards and upwards as indicated.

By varying the position of the cutter relative to the cutting shoe, Fig.1(c), it is possible that a balance can be struck between the two opposing effects and minimal disturbance caused; the elements P_5 and P_6 experience negligible movement as the instrument approaches and advances beyond them.

In an ideal situation, the soil that enters the instrument will be exclusively that volume of soil that is directly ahead of the instrument but within the space that it will occupy as it advances. In such a case the distortion of the volume of soil ABCD in Fig.2(a) as it passes through the cutting shoe will be the same as that being extruded through a die of similar dimensions ABCD in Fig.2(b).

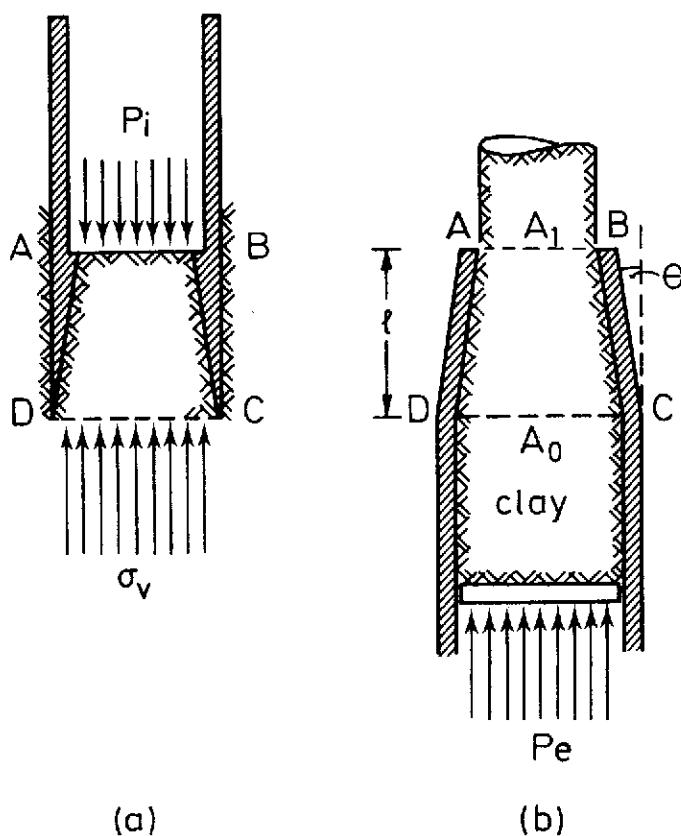


Fig.2 Extrusion of clay through the cutting shoe

In a series of experiments of extrusion of clay, Clarke (1981) showed that the extrusion pressure p_e (making allowance for the friction between the clay and the parallel sided section of the clay before entering the die) is a function of the undrained shear strength of the clay c_u and the geometry of the die represented by

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BRITISH EXPERIENCE WITH THE SELF-BORING PRESSUREMETER

the area ratio A_0/A_1 and either the length l or the angle θ of the die.

This relationship can then be used to predict the performance of the clay entering the instrument in the ideal case. At a particular site and test depth, the strength of the soil will almost certainly be known approximately from previous site investigation, and the total vertical stress σ_v (at the horizon CD) and the total stress due to drilling fluid etc. within the instrument (at the horizon AB) can be estimated. Using these values the optimum dimensions of the cutting shoe can be selected. In practice, the easiest parameter to vary is the effective length of the die by adjustment of the position of the cutter.

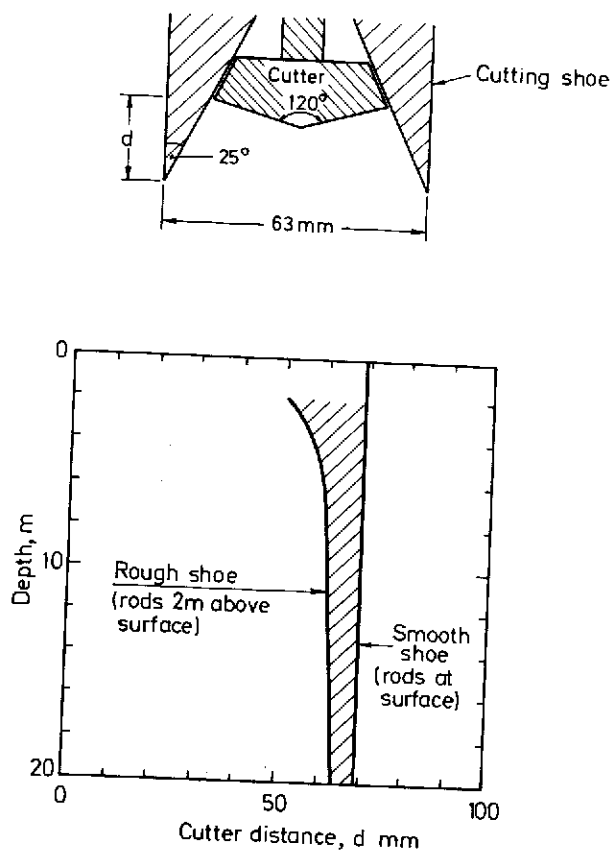


Fig.3 Optimum setting of cutter for a site in London clay

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An example of the outcome of such a calculation is given in Fig.3 for an investigation carried out in London clay. Fig.3(a) shows a section through the cutting shoe and cutter used on that occasion; Fig.3(b) indicates how the optimum setting of the cutter (in terms of the dimension d) should vary with depth below the ground surface. The shaded zone is considered to be the acceptable range of values bounded by the two extremes of (i) a rough cutting shoe with the drill rods projecting 2 m above the surface, and (ii) a smooth shoe with the top of the drill rods level with the ground surface. For this case a single value of d of about 65 mm would be suitable for the complete profile to a depth of 20 m.

These findings also apply to the push-in-pressuremeter developed at the Building Research Station for offshore use and operated as the Stressprobe (Reid et al, 1982). This instrument is designed to have relief behind the cutting shoe (level AB in Fig.2(a)) so that, in the absence of a cutter, as the soil enters the body of the instrument during insertion the frictional forces are confined to the shoe only. With appropriate design of the cutting shoe, the requirements of minimal disturbance can be met.

IV. A GOOD QUALITY PRESSUREMETER TEST IN CLAY

In this and the next two sections, examples are given from one site of (i) a good quality pressuremeter test in clay, (ii) a good quality test in sand and (iii) a poor quality test in sand. The site is part of the outer harbour at Zeebrugge where it is planned to construct large storage tanks for liquefied natural gas. As part of the overall site investigation PM Insitu Techniques Ltd. were commissioned by Distrigas N.V. to carry out a profile of self-boring pressuremeter tests; the work was supervised by Tractebel Z acting as engineers.

The soil profile at this site consists of hydraulic pumped fill to a depth of 15 m, quaternary formations consisting mainly of sands from 15 m to 33 m, Bartoon clay from 33 m to 46 m and below this Pamielian deposits being predominantly sandy soils.

The self-boring pressuremeter cannot be drilled through soil containing hard inclusions, such as sandstone layers and gravel which occur at this site. It cannot be drilled continuously through sand deposits as it leaves an unsupported borehole which may collapse and make extraction of the instrument difficult if not impossible.

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On this site it was therefore necessary to use a drilling rig to provide a borehole which would not collapse and to cope with hard inclusions. The borehole was drilled and cased using a shell and auger rig to a depth of 1 m above the level of a proposed test position. The SBP was lowered down the borehole, and drilled beyond the bottom of the casing to at least 1 m into material presumed to be undisturbed. A pressuremeter test was carried out. In certain instances it was possible to advance the SBP a further metre and carry out a second test. The SBP was withdrawn from the borehole so that the hole could be drilled further by the shell and auger rig. The one borehole was terminated at a depth of 88 m and included 24 pressuremeter tests.

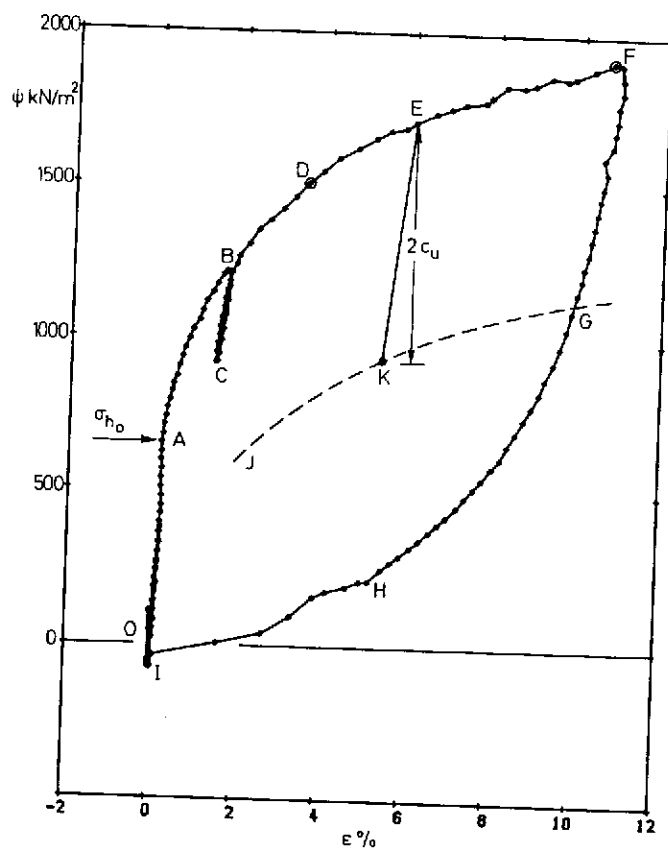


Fig.4 Results of a pressuremeter test in Bartoon clay, Zeebrugge

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The results of a pressuremeter test in clay at a depth of 43.4 m are reproduced in Fig.4. The readings are automatically recorded on site both on magnetic disc and as a hard copy on paper, and subsequently processed and plotted by computer. The pressure ψ applied to the inside of the membrane is plotted against the strain ϵ which is the observed expansion of the membrane divided by its initial radius a_0 . The pressure ψ has been corrected for the (small) strength of the membrane itself, represented by the difference between the ordinates of points 0 and 1.

4.1 Measurement of in situ lateral stress

At the start of the expansion test the membrane is fitting tightly over the body of the instrument and has the same diameter as the cutting shoe. In theory no expansion of the membrane should be detected until the applied pressure ψ is equal to the in situ total lateral stress in the ground in contact with the pressuremeter. In reality there will be some small compliance of the instrument itself, until at point A (the lift-off pressure), on the expansion curve, the soil starts to deform under increasing lateral stress.

It is usually assumed (both for convenience and lack of better information) that the lateral stress in the ground is the same in all horizontal directions; this is unlikely to be the case except in very homogeneous deposits subject solely to one-dimensional consolidation during their geological history.

The Cambridge SBP has three separate feeler arms for measurement of the movement of the membrane, which are symmetrically disposed at 120° spacing around the section of the instrument. By careful scrutiny at suitably large scale of separate plots of ψ against each of the values of ϵ , three separate estimates of the σ_{h_0} can be

obtained (on the presumption that there has been negligible disturbance of the ground during insertion of the pressuremeter). The results in Fig.4 are for one of the three feeler arms, and indicate a value of σ_{h_0} of 646 kN/m^2 . The measurement of in situ lateral

stress by the SBP is discussed in detail by Ghionna, Jamiolkowski and Lancellotta (1982) and by Lacasse and Lunne (1982).

4.2 Measurement of undrained shear strength

In the original analysis of Gibson and Anderson (1961) for the interpretation of an undrained pressuremeter test in clay, it is assumed that the clay behaves as a perfectly elastic/perfectly plastic material characterised by a shear modulus G , (a Poisson's ratio

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$v = 0.5$) and an undrained shear strength c_u . The analysis leads to the result that, after failure has been initiated in the clay, the applied pressure ψ is linearly related to the logarithm of the *current* volumetric strain $\Delta V/V$. Moreover, if natural logarithms are used the gradient of the line will be equal to c_u .

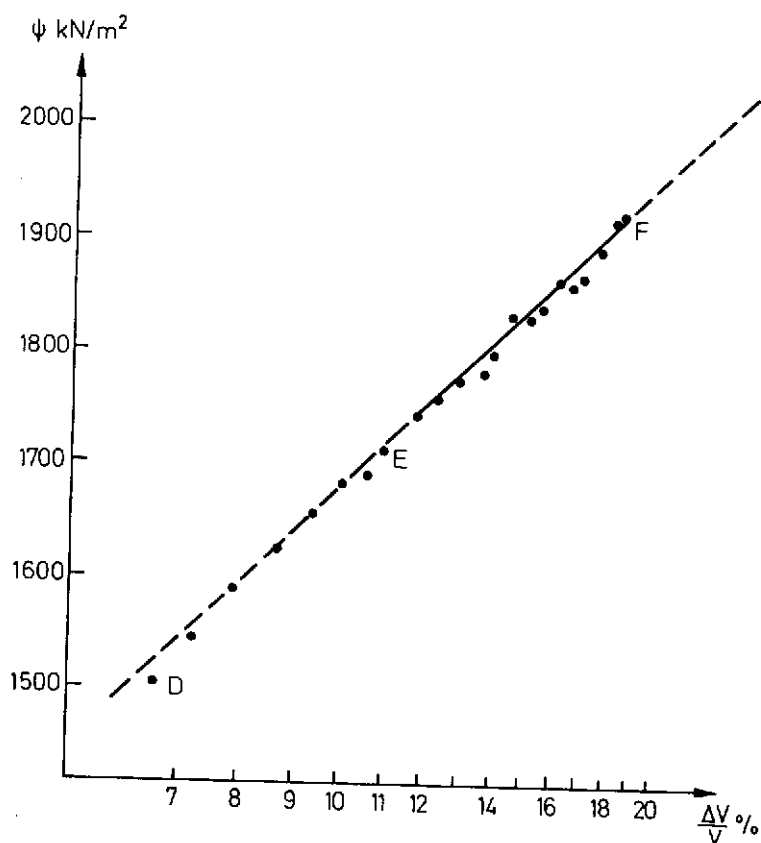


Fig.5 Processed results of a pressuremeter test in Bartoon clay

The field results of the pressuremeter test of Fig.4 have been replotted in Fig.5 in the above manner as ψ against $\log (\Delta V/V)$. Note that the volumetric strain is directly related to the observed strain by the relationship

$$\Delta V/V = 1 - (1 + \epsilon)^{-2}. \quad (1)$$

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The points D, E and F correspond with those selected in Fig.4. The results lie remarkably close to a straight line, and that selected has a gradient $c_u = 386 \text{ kN/m}^2$. The newer and superior analysis of Baguelin et al (1972), Ladanyi (1972) and Palmer (1972) does not require an assumption to be made about the stress-strain behaviour of the soil. It leads to the result that the local gradient of the curve drawn through the points in Fig.5 is the shear stress corresponding to the shear strain being experienced at that stage of the test by the clay in contact with the pressuremeter. The fact that the curve is so close to a straight line vindicates the assumption implicit in the Gibson and Anderson analysis, that the clay behaves as a perfectly elastic/perfectly plastic material.

From experience gained at a large number of sites it is believed that the Gibson and Anderson analysis is entirely satisfactory for design purposes in geotechnical engineering and that the extra sophistication of the 1972 analysis is not warranted, particularly because it is so sensitive to the choice of datum.

4.3 Measurement of shear modulus

The shear modulus of the clay can be measured by arranging for an unloading-reloading cycle such as BCB to be included in the expansion test. If the clay behaves as a perfectly elastic material in unloading then BC will be a straight line of gradient $2G$ (using small strain theory). It should be noted that the cycle BCB is markedly linear with very small hysteresis, and gives a value of the shear modulus $G = 47 \text{ MN/m}^2$.

In carrying out such an exercise care must be taken not to exceed the elastic limit of the clay during the unloading phase; this restricts the amplitude of the stress cycle that can be applied. For the ideal elastic/plastic material the allowable amplitude is $2 c_u$ as illustrated in Fig.6. The top diagram, Fig.6(a), is a horizontal section through the pressuremeter membrane at the start of the unloading cycle, with three typical elements of soil, P, Q and R, at different radial distances. The shear stress τ and (engineering) shear strain γ experienced by the three elements are shown qualitatively in Fig.6(b), with the corresponding points on the applied pressure-strain curve of Fig.6(c). On unloading, the element of soil P in contact with the membrane can sustain a change in shear stress of $2 c_u$ before it fails at state X (when the radial stress becomes the minor principal stress at failure).

It is therefore possible to construct a curve such as JKG in Fig.4, which serves as a theoretical boundary to the elastic behaviour in unloading. It should be noted that this boundary fits well with the extent of the linear portion of the final unloading part of the test FGHI.

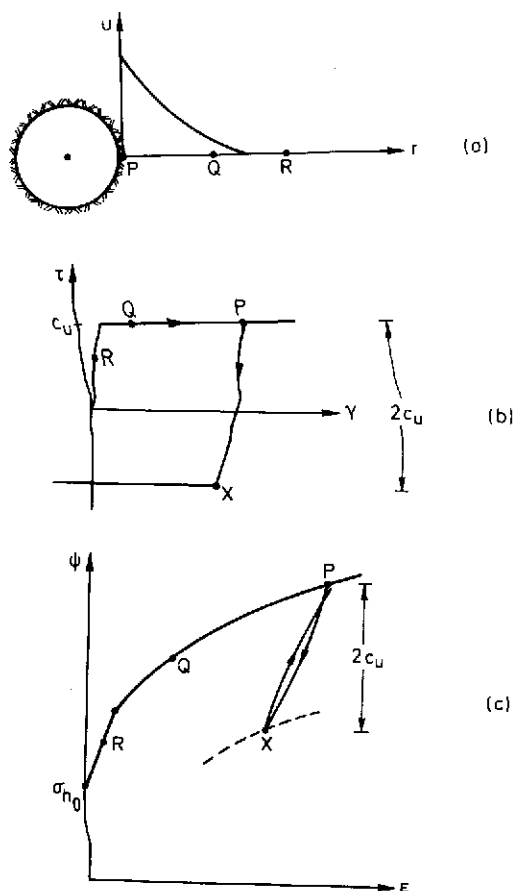


Fig.6 Theoretical limit of elastic behaviour in pressuremeter test in clay

It is well established that for granular materials the value of elastic moduli are dependent on the value of the mean effective stress p' , usually proportional to $(p')^n$ where n is typically 0.5 to 0.6. The question arises as to how this phenomenon will affect the interpretation of unloading-reloading cycles.

For a perfectly elastic material, an expansion test (or an unloading test) of an infinitely long pressuremeter is such that there is no change of p' , so that the unloading response BC should be unaffected and remain straight.

If the unloading cycle is conducted at a different stage of the test, such as at E, the mean *total* stress p is greater than at B. However, once at failure, the clay, while it remains undrained, will

not experience any change in *effective* stresses so that the deduced shear modulus should remain the same and be independent of the stage of the test at which it is measured.

In reality some drainage will take place during the test. In Fig.6(a) the likely radial distribution of excess pore-pressure u is sketched; this gives rise to substantial hydraulic gradients in the radial direction. Some consolidation will occur, with an increase in p' near the pressuremeter, and with the consequence that the observed values of G would be expected to increase marginally with the value of the strain at which the unloading cycle is commenced.

V. A GOOD QUALITY PRESSUREMETER TEST IN SAND

The results of a pressuremeter test in sand (in the same borehole as that for the test in clay) at a depth of 10.6 m are reproduced in Fig.7. The shape of the curve is different from that of the test in clay of Fig.4, both in the loading portion MPQRT and the final unloading portion TUVW. This difference is always observed in good quality tests and can be used as an indicator of the material tested. It can be represented numerically by the factor β defined as (Baguelin, 1982):

$$\beta = \frac{p_{20} - p_5}{p_{20} - p_0} \quad (2)$$

where p_0 , p_5 and p_{20} are respectively the applied pressure at zero, 5% and 20% volumetric strain. Noting that volumetric strain is approximately twice the plotted strain ϵ , the value of β for the clay in Fig.4 is 0.45, whereas that for the sand in Fig.7 is 0.51.

5.1 Measurement of in situ lateral stress and pore pressure

The same method as that outlined in section 4.1 can be used for an assessment of the in situ lateral total stress. When the results of Fig.7 are plotted to a larger scale, the lift-off pressure is given by point L, i.e. a stress of 107 kN/m².

In the case of a test on a cohesionless material which is sufficiently permeable, the final part of the unloading curve VW provides valuable information about ground conditions. As the membrane is deflated, ultra-loose sand is collapsing back into the enlarged borehole under very small effective stresses. In the extreme condition at W, the membrane is being pushed back to its initial size by the

BRITISH EXPERIENCE WITH THE SELF-BORING PRESSUREMETER

action of the ground water, and the pressure recorded inside the membrane is a measure of the ambient pore pressure u_0 given by the horizontal tangent WVZ. (Note that point W is the last one automatically recorded during the test, and the computer when producing the plot of Fig.7 has inappropriately drawn the chord WO, instead of continuing the line VW back to zero strain.)

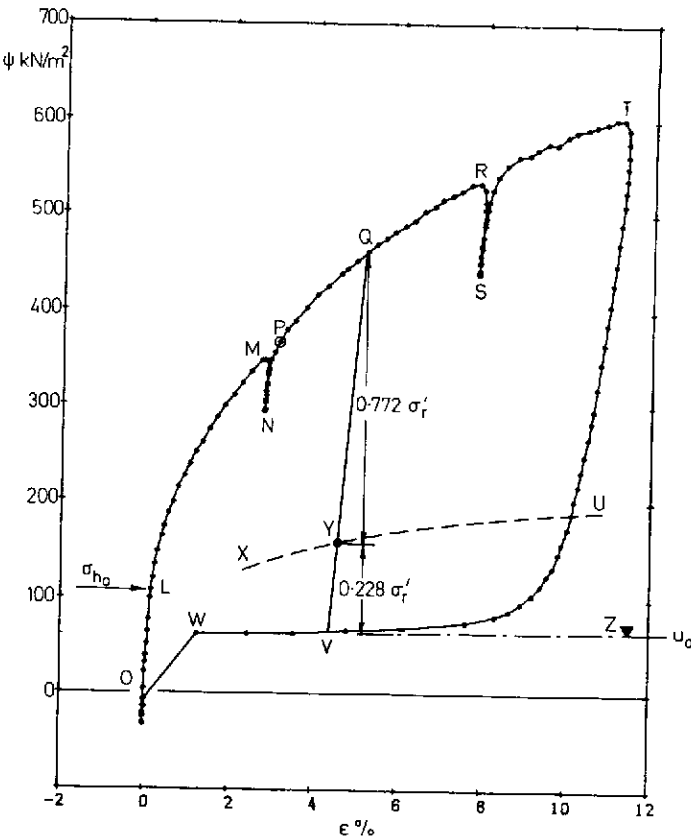


Fig.7 Results of a pressuremeter test in sand at Zeebrugge

The value of u_0 is 62 kN/m^2 corresponding to a water table at a depth of 4.3 m. The water table in the hydraulic fill was observed to fluctuate between wide limits as the tide level varied. Estimating the total vertical stress, at the test depth of 10.6 m, to be 160 kN/m^2 , this would suggest a value of the coefficient of earth pressure at rest K_0 to be $(107-62)/(160-62) = 0.46$.

5.2 Measurement of friction angle

In the analysis of Hughes, Wroth and Windle (1977) for the interpretation of a drained pressuremeter test in sand, it is assumed that the sand behaves elastically before failure, and fails at a constant ratio of effective stresses and at a constant rate of dilation. The analysis leads to the result that, after failure has been initiated in the sand, the logarithm of the *effective* radial stress $(\psi - u_0)$ is linearly related to the logarithm of the strain ϵ ; the gradient of the line s is given by the expression

$$s = \frac{\sin \phi' (1 + \sin \nu)}{1 + \sin \phi'} \quad (3)$$

where ϕ' is the angle of internal friction and ν is the angle of dilation.

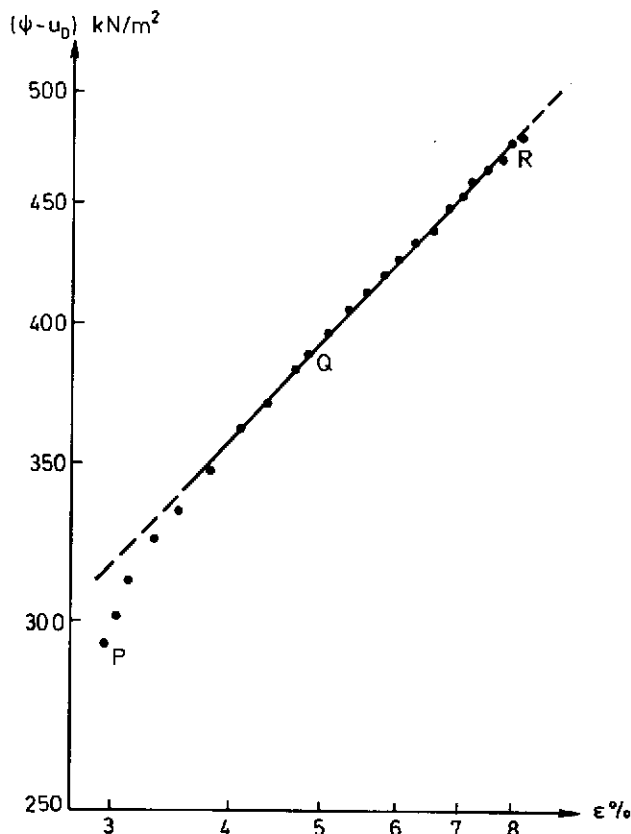


Fig.8 Processed results of a pressuremeter test in sand at Zeebrugge

BRITISH EXPERIENCE WITH THE SELF-BORING PRESSUREMETER

The field results of the pressuremeter test of Fig.7 for the range PQR have been plotted in this manner in Fig.8. They approximate to a linear relationship whose gradient $s = 0.425$. Adopting a value for the critical state angle of friction at constant volume of 35° , gives a value of $\phi' = 39^\circ$ and $v = 9.5^\circ$ (see Hughes et al, 1977).

5.3 Measurement of shear modulus

The shear modulus of the sand can be measured in the same way as that for the clay, by carrying out small unloading-reloading cycles such as MN and RS in Fig.7. The corresponding value of the shear modulus for cycle MN is 31 MN/m^2 .

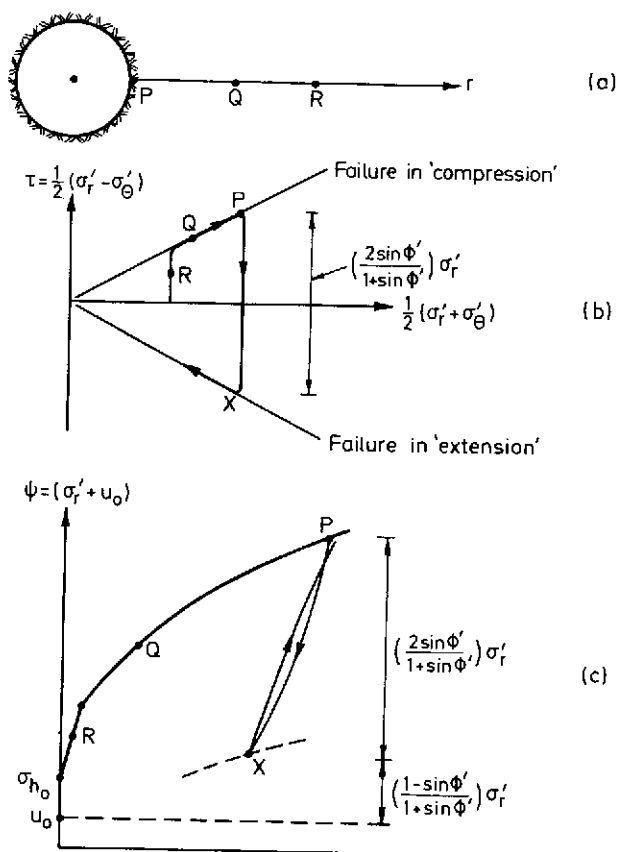


Fig.9 Theoretical limit of elastic behaviour in pressuremeter test in sand

As for the case of the clay there is a limit to the elastic range of behaviour during unloading. For the idealised sand behaviour used in the analysis, the allowable amplitude of the stress cycle PX is

$\left(\frac{2 \sin \phi'}{1 + \sin \phi'} \right) \sigma_r'$, as illustrated in Fig.9. The behaviour of the sand is represented in Fig.9(b) in terms of the shear stress $\tau = \frac{1}{2}(\sigma_r' - \sigma_\theta')$ and a mean effective stress $\frac{1}{2}(\sigma_r' + \sigma_\theta')$. The effective stress path during elastic unloading is PX (at constant mean effective stress).

It is therefore possible to construct a curve, such as XYU in Fig.7, which serves as a theoretical boundary to the elastic behaviour in unloading. This has been based on the observed value of $\phi' = 39^\circ$ to give the ratios of 0.772 and 0.228 used in the plot. It should be noted that σ_r' varies with the stage of the test so that the distance QY would not be the same if Q were to be chosen elsewhere on the virgin loading curve; care must be taken to subtract the ambient pore-pressure u_0 from the applied pressure ψ to give the relevant value of σ_r' acting on the sand which is in contact with the membrane. The theoretical boundary fits well with the extent of the linear portion of the final unloading part of the test TUVW.

Unlike the clay there should be no excess pore-pressures generated during the expansion test. It follows that the mean effective stress p' increases as the test progresses, and it would be expected that the shear modulus would increase as the position of the unloading-reloading cycle moves further from the origin of the test. This increase is expected to be proportional approximately to $\sqrt{p'}$ which at failure will be proportional to $\sqrt{\sigma_r'}$, i.e. to $\sqrt{(\psi - u_0)}$ at the start of the unloading cycle. Hence, in Fig.7, for the two unloading cycles MN and RS the modulus measured at RS should be of the order of $\sqrt{480/280} \approx 1.3$ times that measured at MN.

VI. A PRESSUREMETER TEST IN DISTURBED SAND

It is not always possible to drill the self-boring pressuremeter into the ground without causing some disturbance, particularly in sand. Caution must then be adopted in interpreting a pressuremeter test in such circumstances.

An example of a test in disturbed sand is given in Fig.10, conducted in the same borehole at Zeebrugge. It should be abundantly clear from inspection of the curve that the sand is badly disturbed locally around the instrument, and is in a very loose condition. The relevant evidence consists of (i) the very low gradient of the unloading-reloading cycle BCDE (with no proper loop), (ii) the portion DE of expansion with no increase of applied pressure

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which merely indicates the ambient pore-pressure and (iii) the reverse curvature between E and F. Beyond F sufficient expansion has occurred to start to test the sand at some radial distance from the instrument. A value of the shear modulus deduced from the cycle FGH is bound to be an underestimate of the in situ value for the undisturbed sand.

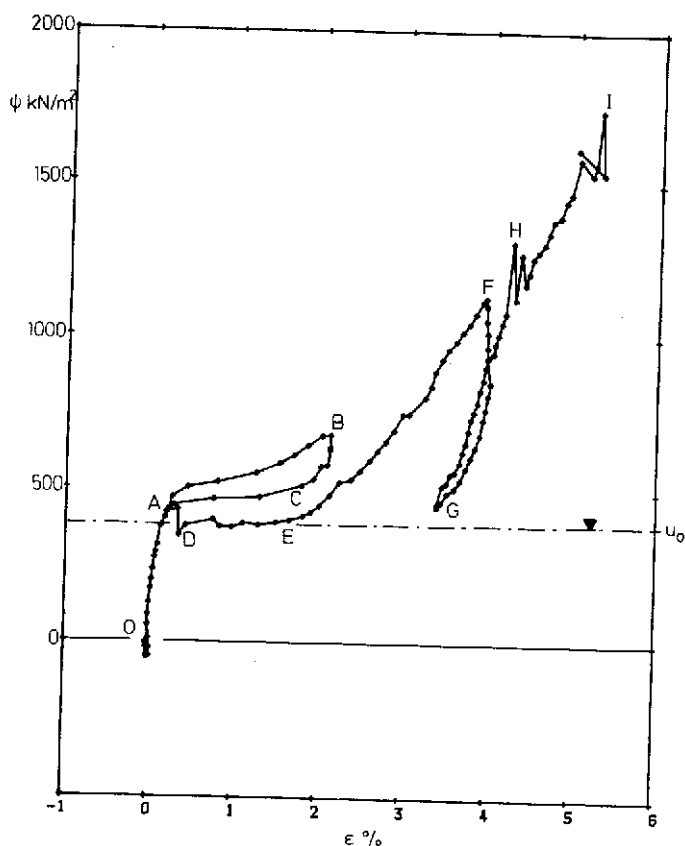


Fig.10 Results of pressuremeter test in disturbed sand at Zeebrugge

VII. CONCLUSIONS

The extent of the experience gained in the practical use of the self-boring pressuremeter in the UK in the period May 1978 to September 1981 is presented in Table 1. From this it will be seen

BRITISH EXPERIENCE WITH THE SELF-BORING PRESSUREMETER

that the soil property most frequently measured is the shear modulus particularly in the case of sands. In situ measurements of the shear modulus together with the value of the effective lateral stress are essential if deformation analyses - such as finite element computations - are to be carried out using realistic models of soil behaviour which incorporate non-linearity, plasticity or failure.

TABLE 1

Self-boring pressuremeter tests carried out by PM Insitu Techniques Ltd during period May 1978 - September 1981

CLAYS (including silts)	No.
Tests in normally consolidated and lightly over-consolidated clays	219
Tests in overconsolidated clays	281
Undrained tests	485
Drained tests	12
Holding tests	4
Cyclic loading tests	3
Measurements of lateral stress	485
Measurements of shear modulus	569
Measurements of undrained shear strength	467
Measurements of coefficient of consolidation	4
Measurements of angle of friction	8
Measurements of lateral stress with K_0 cell	104
Maximum depth drilled and tested	80.3 m
SANDS	
Tests in normally consolidated and lightly overconsolidated sands	43
Tests in overconsolidated sands	32
Measurements of lateral stress	32
Measurements of shear modulus	118
Measurements of angle of friction	53
Maximum depth drilled and tested	88.0 m

BRITISH EXPERIENCE WITH THE SELF-BORING PRESSUREMETER

This accumulated experience is sufficient to have established that the self-boring pressuremeter can be used successfully for the in situ measurement of soil properties for use in geotechnical engineering. Under most circumstances the instrument can be inserted with minimal disturbance to provide good quality data. Care must be taken both in operating the instrument and in interpreting the results.

VIII. ACKNOWLEDGMENTS

The results included in this paper are quoted by permission of PM Insitu Techniques Ltd and their clients for the job at Zeebrugge, Distrigas N.V. and Tractebel Z.

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CHARACTERISTICS OF SATURATED CLAYS AS OBTAINED FROM SBP TESTS

A summary is given in the following of the six-years experience gained by the Geotechnical Engineering Group of the Politecnico di Torino (PDT) in using the Self-Boring Pressuremeter (SBP) in saturated cohesive deposits and interpreting the test results.

The SBP, a highly improved version of the well-known Ménard Pressuremeter (MP), has been developed simultaneously in France [Baguelin et al. (1972)] and England [Wroth and Hughes (1972)]. More recently, other SBP equipment has also been developed in the USA [Clough and Denby (1980), Steussy (1980)] and in Japan [Mori (1981)].

In 1977, the PDT undertook a co-operative research program with MIT under the supervision of Prof. C.C. Ladd, receiving financial support from the Federal Highway Administration (FHWA) of the U.S. Department of Transportation. The aim of the program was to investigate the SBP performance in saturated clays, focusing attention on the following main objectives:

- a) To investigate the influence of installation and testing procedure on test results, including the examination of test repeatability, and the effects of probe geometry (length-to-diameter ratio, L/D).
- b) To evaluate the influence of using various methods of interpretation on estimated values of initial total in situ horizontal stress (σ_{ho}) and derived undrained stress-strain-strength characteristics (e.g. undrained modulus, E_u ; peak and average values of undrained shear strength, c_u ; limiting pressure p_{lim}).
- c) To compare the above parameters with values obtained from K_0 -consolidated undrained (CK_0U) triaxial (TX) and direct simple shear (DSS) tests and other in situ devices such as the field vane (FV).

SBP tests have been carried out at seven sites [Table I] using two French probes of the PAFSOR type with ratios of L/D equal to 2 and 4,

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CHARACTERISTICS OF SATURATED CLAYS FROM SBP TESTS

TABLE I

Site	Year	Type of SBP	L/D	OCR (-)	W_n (%)	LL (%)	PI (%)	No. of tests
PORTO TOLLE (Italy)	1975	PAFSOR	2	1	36.4±14	52.3±2.2	30.5±16	62
	1976	PAFSOR	2					
	1979	PAFSOR	2					
	1979	PAFSOR	4					12
TRIESTE (Italy)	1979	PAFSOR	4	≤ 1	49.9±4	70.7±11.5	47.4±10	11
	1979	PAFSOR	2					11
GUASTICCE (Italy)	1977	PAFSOR	2	1 to 1.1	67.6±6.5	88±11	62.7±8.9	6
MONTALTO DI CASTRO (Italy)	1976	PAFSOR	2	2.5 to 4	25.9±2	52±5.5	34±5.5	21
BANDAR ABBAS (Iran)	1977	PAFSOR	2	1.5 to 2.5	24.1±2.7 32.9±5.8	34.8±2.7 52.8±7.4	16.2±2.8 27.8±4.4	47
ONSOY (Norway)	1979	CAMKOMETER	6	1 to 2	60-70	55-75	20-35	12
DRAMMEN (Norway)	1979	CAMKOMETER	6	1.1 to 1.5	30-35 55-65	50-70	10-15 25-30	6

respectively, and the English probe Camkometer with L/D = 6.

The tests carried out with the Camkometer at the Norwegian sites have been conducted in co-operation with the Norwegian Geotechnical Institute [see Lacasse et al. (1981)].

The following sections summarize the principal findings of the research program with respect to points b) and c) above. The detailed results are presented in the paper of Ghionna et al. (1981).

Reference is also made to the work carried out at MIT in the framework of the same research program. This part has been summarized by Ladd et al. (1979).

A. INTERPRETATION OF SBP TESTS

In principle, SBP tests make it possible to obtain the following important information on saturated cohesive deposits:

- in situ total horizontal stress, σ_{ho} ;

CHARACTERISTICS OF SATURATED CLAYS FROM SBP TESTS

- undrained strength (c_u) and deformation (E_u) properties and/or the complete undrained stress-strain curve;
- limiting pressure (p_{lim}).

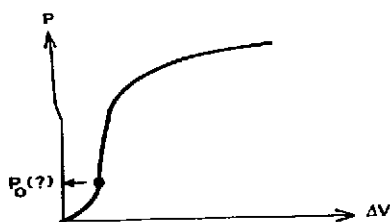
However, serious questions arise as soon as one tries to find out to what extent the significant potential of the SBP test has been put to good use so far through practical applications.

In fact, a review of available applications of the SBP in geotechnically well-documented cohesive soils leads to the conclusion that not withstanding its great potential the test leads very frequently to results which are not in agreement with those of other in situ and high-quality laboratory tests. Leaving the detailed illustration and discussion of this phenomenon to subsequent sections of this paper, it must be pointed out that most of the uncertainties regarding the reliability and true meaning of SBP data are related to the following unsolved questions of experimental and practical nature:

- How is the disturbance caused by the self-boring process to be quantified, and how can it be controlled?
- What kind of criteria should be used in order to establish the optimum time interval (= relaxation time) which must occur between probe insertion and the start of the expansion test? [Baguelin et al. (1972), (1974), (1978), Wroth and Hughes (1974), Denby (1978), Clough and Denby (1980), Denby et al. (1981), Mori (1981)].
- How is the expansion rate to be selected in order to achieve "acceptable" results? Two opposite needs must be kept in mind in this respect: the rate must be quick enough in order to prevent partial drainage of the soil, on the other hand it should be not too rapid because the stress-strain relationship of many natural clays is sensitive to strain-rate effects [see for example Prevost (1976), Ladd et al. (1977), Ladd et al. (1979)].
- What is the influence of the L/D ratio on strength and deformation properties obtained from SBP tests? In this respect one has to make a distinction between pressuremeters measuring the radial displacement, u_r , at midheight of the probe, like the Camkometer, and pressuremeters in which u_r is detected on the basis of the measured volume change of the probe.
- Which are the effects of the flexibility of the measuring system and the drift of the electronic equipment on results of tests with the presently used SBP probes?

The detailed discussion of the above points is beyond the scope of the present paper, but it is believed that before proceeding with the discussion of experimental results it may be useful to define what according to the writers' experience may tentatively be judged as an excellent expansion curve. Such a curve (Fig. 1.b), in contrast to the one which one obtains with the MP (Fig. 1.a), must conform to each of the following requirements:

CHARACTERISTICS OF SATURATED CLAYS FROM SBP TESTS



a) MENARD'S PRESSUREMETER CURVE

FIG. 1

Examples of pressuremeter curves



b) SELF-BORING PRESSUREMETER CURVE

- there must be no inflection point at the beginning of the expansion curve; this means the computed pressuremeter-modulus, E_{up} , must decrease monotonically as the applied cavity pressure (p) increases;
- the measured initial pressure, P_0 , must be very close to the best estimate of σ_{ho} ;
- the pore pressure, u , at the interface between probe and soil, measured before the start of the expansion test, must be close to the hydrostatic pore pressure in the surrounding soil mass, u_o .

A-1. EVALUATION OF IN SITU HORIZONTAL STRESS

The possibility of evaluating the existing in situ horizontal stress certainly represents one of the most promising and interesting features of the SBP test.

The reliability of σ_{ho} obtained from SBP tests is generally assessed by comparing it with values determined by means of other in situ and/or laboratory tests. However, the validity of the laboratory procedures used to assess σ_{ho} must equally be subject to some reservations [see Wroth (1975), Ladd et al. (1977), Jamiolkowski et al. (1979)]. The comparison between σ_{ho} (SBP) and the "best estimate" of σ_{ho} obtained from other tests is therefore but a relative procedure.

CHARACTERISTICS OF SATURATED CLAYS FROM SBP TESTS

Keeping in mind the above mentioned limitations, the experience gained in determining σ_{ho} from SBP tests may be summarized as follows:

- From good or excellent expansion tests which show no inflection point it may reasonably be expected that p_o equals approximately σ_{ho} .
- It is of great importance for a correct determination of σ_{ho} from SBP tests that during the self-boring process the probe maintains its cylindrical shape as perfectly as possible in order to minimize lateral soil strains due to installation, see for example Law and Eden (1981).
- When using the Camkometer probe it is essential to properly evaluate the so-called "lift-off" pressure [Wroth and Hughes (1973), Denby (1978), Ladd et al. (1979), Denby and Clough (1980)]. The lift-off pressure corresponds to the early outward movement of the strain arms which overcomes, according to Denby (1978), the small but unavoidable disturbance caused even by a nearly perfect insertion procedure. An example of the lift-off pressure is shown in Figure 2; generally, the applied internal pressure does not cause any rise of the pore pressure in the soil until it reaches the lift-off pressure, p_{o1} . Once the values of p_{o1} and ϵ_{o1} have been established, it is assumed that the beginning of the expansion curve occurs at a point which corresponds to these values, and the strain ϵ_{o1} which has occurred between p_o and p_{o1} is neglected in all further considerations.
- Recently it was discovered [Dalton (1981)] that the so-called lift-off pressure phenomenon is at least partially due to the mechanical deformability of the three feeler arms which measure the radial displacement. The example of lift-off pressure shown in Figure 2 refers to non-modified feelers as far as their set-up and construction is concerned, which, however, were operated by a modified electronic equipment allowing the separate display of the readings of each individual feeler. Figure 3 shows an example of the lateral displacements measured after the modification of the feelers. As a result of the modification the lift-off phenomenon is appreciably reduced, but one discovers that when the radial displacement measurements of the three feelers are made separately, each arm leads to a slightly different value of lift-off pressure. This fact cannot be explained presently; it may be due to a slight eccentricity in the stress and strain field induced in the soil by the self-boring process and further research is needed.
- The relaxation time plays an important part in the correct assessment of σ_{ho} from SBP tests as can be seen from the example shown in Figure 4. This fact which is now widely recognized [Baguelin et al. (1972), (1974), (1978), Wroth and Hughes (1974), Windle and Wroth (1977), Dalton (1981), Clough and Denby (1980), Mori (1981), Denby et al. (1981), Ladd et al. (1979), Ghionna et al. (1981)] requires further research, however.

CHARACTERISTICS OF SATURATED CLAYS FROM SBP TESTS

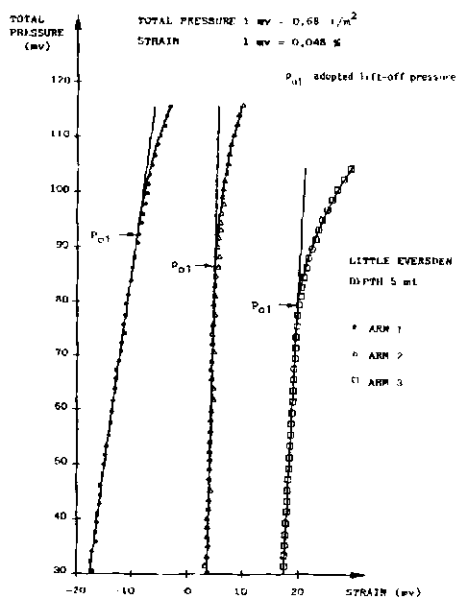


FIG. 2: Determination of the lift-off pressure

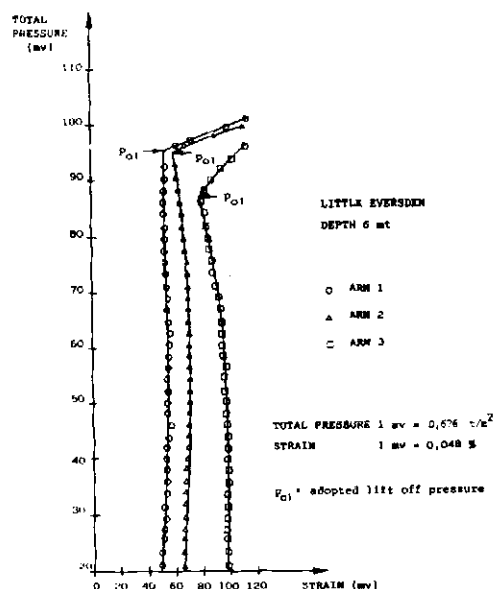


FIG. 3: Lateral displacements after the modification of the feelers.

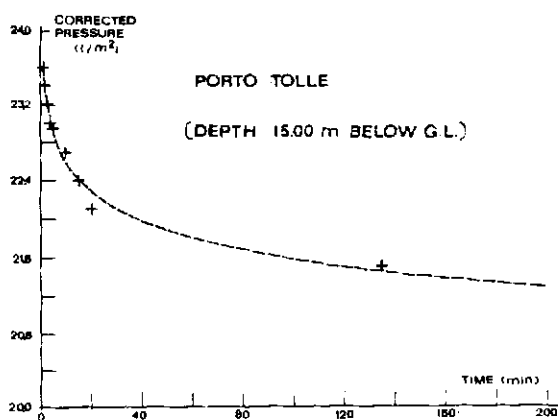


FIG. 4: Example of relaxation curve

- There are different procedures available in the existing literature for the correction of the experimental expansion curves for soil disturbance in order to improve the reliability of the predicted σ_{ho} values [see Marsland and Randolph (1977), Denby (1978), Ladd et al. (1979), Clough and Denby (1980), Prevost (1979), Arnold (1981), Ghionna et al. (1981)]. The relative merits of some of these approaches will be discussed later.

CHARACTERISTICS OF SATURATED CLAYS FROM SBP TESTS

A-2. STRESS-STRAIN AND STRENGTH CHARACTERISTICS

The available interpretation approaches may be split into three categories:

A-2-a. Elasto-plastic approach.

It is assumed that the undrained soil response may be approximated by an elastic ideally-plastic stress-strain relationship. To this category belongs the theory worked out by Gibson and Anderson (1961) for the interpretation of tests carried out with Ménard's pressuremeter, modified and adapted to the SBP by Windle and Wroth (1977).

A-2-b. Approach of Baguelin-Ladanyi-Palmer.

Without making any a priori assumptions regarding the soil stress-strain relationship, Baguelin et al. (1972), Ladanyi (1972) and Palmer (1972) showed that the following equation holds for the shear stress at the cylindrical cavity wall under undrained plane-strain conditions:

$$\tau = \frac{1}{2} (\sigma_r - \sigma_\theta) = \frac{1}{2} \epsilon_\theta (1 + \epsilon_\theta) \cdot (2 + \epsilon_\theta) \frac{dp}{d\epsilon_\theta} \quad \dots(1)$$

where:

σ_r, σ_θ = radial and circumferential stress, respectively

ϵ_θ = circumferential strain = $(1 - \frac{\Delta V}{V + \Delta V})^{\frac{1}{2}} - 1$

At small strains this equations reduces to:

$$\tau = \epsilon_\theta \cdot \frac{dp}{d\epsilon_\theta} \quad \dots(2)$$

The complete stress-strain curve can be obtained from the above equation either by graphical or analytical procedures leading to values of both c_u and the so-called derived undrained modulus, E_u , which may conveniently be expressed as a function of the induced shear stress level [see for example Baguelin et al. (1972), Palmer (1972), Wroth and Hughes (1973)].

A-2-c. Approach of Prevost - Höeg and Denby.

Experience gained with the Baguelin - Ladanyi - Palmer approach showed that in many cases the derived $\tau - \gamma$ curve has an irregular shape. Because of this, a smoothing of the experimental data is necessary before the stress-strain relationship can be worked out [Baguelin et al. (1972), Jamiolkowski and Lancellotta (1977), Denby (1978), Ladd et al. (1979)]. Generally, this smoothing operation consists in adopting mathematical functions for fitting the pressuremeter expansion curves, which in most cases implies that a stress-strain curve of some assumed shape is incorporated in the interpretation procedure.

CHARACTERISTICS OF SATURATED CLAYS FROM SBP TESTS

The approach based on the theoretical formulation of Prevost and Höeg (1975) has been used by Jamiolkowski and Lancellotta (1977), Denby (1978) and Ladd et al. (1979) for the interpretation of SBP tests.

In particular, Ladd et al. (1979) proposed to modify the relationship in order to account for the possibility of having less than 1% strain at failure.

A.3. LIMIT PRESSURE

The theoretical limit pressure, p_{lim} , as defined by Bishop et al. (1945), is the internal pressure existing at an infinite expansion.

Because of the physical impossibility of expanding the probe to such big a volume, p_{lim} must be determined by an appropriate extrapolation method. Reminding the readers that a complete review of these extrapolation methods can be found in the works of Baguelin et al. (1978) and Ladd et al. (1979), the writers have adopted the following two methods:

- the already mentioned approach of Windle and Wroth (1977);
- the approach of Al-Awkati (1975) which consists in extrapolating the p vs $1/\epsilon_\theta$ plot in order to find p_{lim} at $1/\epsilon_\theta = 0$;
see also Jamiolkowski and Lancellotta (1977) and Ghionna et al. (1981).

B. SITE CHARACTERISTICS

Table II shows the relevant geotechnical characteristics of the sites on which the SBP tests have been performed.

More details may be found in the work of Ghionna et al. (1981).

In addition to the data shown in the table, the following points should also be noted:

- a) For the Porto Tolle site there is clear geological evidence that the silty clay layer encountered some 6 to 8 m below G.L. is N.C., see Jamiolkowski et al. (1980).
- b) At the Trieste site the clay layer is still undergoing some consolidation under the weight of an approximately 5 m high fill placed in 1970; see Hansbo et al. (1981).
- c) The silty clay deposits of the Montalto di Castro site are encountered at rather large depths (36 to 66 m below G.L.); most of the performed SBP tests exhibited expansion curves of the Ménard type because of soil disturbance caused by the self-boring process.
- d) The causes of the preconsolidation phenomena at the Bandar Abbas site are not known. In this work, the adopted working hypothesis is that the silty clay stratum has been subjected to mechanical overconsolidation and that its *in situ* K_0 value may therefore be related to OCR.

TABLE II: SITE CHARACTERISTICS

SITE	SOIL TYPE	GWL m b.G.L.	PI %	NORMALIZED STRENGTH AND DEFORMABILITY						SENSI- TIV- ITY FROM FIELD VANE	OCR	NOTES
				TX-CK ₀ -U-CL		DSS-CK ₀ -U		FV				
				$\frac{c_u}{\sigma_{vo}}$	$\frac{E_{u50}}{\sigma_{vo}}$	$\frac{c_u}{\sigma_{vo}}$	$\frac{E_{u50}}{\sigma_{vo}}$	$\frac{c_u}{\sigma_{vo}}$	$\frac{E_{u50}}{\sigma_{vo}}$			
Porto Tolle (ITALY)	Light brown soft silty clay	1.1	30.5	0.31	89±37	0.26	82±23	0.29		2 to 3	1	
Trieste (ITALY)	From grey to dark grey organic clay with shell fragments	1.0	47.4	0.32	90±34	0.28	144±32	0.35		2 to 4	≤1	Still subject to consoli- dation under the weight of fill placed in 1970.
Montalto di Castro (ITALY)	Dark grey from hard to very hard silty clay	8.0	34.0	0.54	250±350	NA	NA	NA	NA	NA	2.5 to 4	Top of layer at a depth of ≈ 37 m below G.L.
Bandar Abbas (IRAN)	Stiff silty clay with inter- bedded layers of clayey silt	2.0	16 to 28	0.54	400	NA	NA	0.56		3 to 4	1.5 to 2.5	Refers mainly to the silty clay
Onsøy (NORWAY)	Grey soft to firm silty clay	0.1	20 to 36	0.33	141±38	0.26	≈135	0.23		5 to 6	1 to 1.2	In NC range ≈ 5 ÷ 6 m below G.L.
Onsøy (NORWAY)	Grey silty clay	0.1	15 to 30	0.45	122	0.28	129	0.48		6 to 9	≈2	Upper 5 m of the deposit; data refer to a depth of 3 to 4 m
Drammen (NORWAY)	Soft lean silty clay	1.0	10 to 15	0.34	260	0.22	285	0.25		7 to 8	≈1.15	
Drammen (NORWAY)	Soft plastic clay	1.0	25 to 30	0.40	235	0.32	195	0.34		7 to 8	1.5	Upper 7 m of the deposit
Guasticce (ITALY)	Grey from very soft to soft clay	0.1	63	0.32	220 to 280	NA	NA	NA		0.30	1 to 1.1	In NC range ≈5 m below G.L.

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- e) As far as the two Norwegian sites are concerned, which are extremely well documented from a geotechnical point of view, reference is made to the work of Lacasse and Lunne (1979) for all further information.

C. EXPERIMENTAL RESULTS AND RELATED COMMENTS

C-1. UNDRAINED SHEAR STRENGTH

The determination of the undrained shear strength from the results of the SBP tests was done by means of the following approaches:

C-1-a. Gibson and Anderson (1961), using:

- p_{lim} determined according to Windle and Wroth (1977);
- p_0 corresponding to the starting pressure of the expansion test;
- E_{up} determined directly from the expansion curve at a stress-strain level ϵ_0 equal to ϵ_{050} , deduced from the stress-strain curve derived from Prevost and Höeg's (1975) approach with the modifications proposed by Ladd et al. (1979).

A so called "average" undrained strength, c_u^{GA} , is obtained by the procedure.

C-1-b. This "average" strength has also been evaluated using the graphical procedure suggested by Windle and Wroth (1977) which yields c_u^{WW} from the slope of the p vs $\ln \Delta V/V$ plot in the post-yield range of the expansion curve.

C-1-c. The undrained peak strength, c_u^{PH} , has been computed from the stress strain curves that result from Prevost and Höeg's (1975) approach (modified by Ladd et al. (1979)) which is valid for strain softening material.

C-1-d. For the tests performed in 1979 at Porto Tolle and those carried out at the Norwegian sites the investigation method proposed by Denby (1978) has been tried for the assessment of the undrained peak strength, c_u^D .

The values of c_u^{WW} , c_u^{PH} and c_u^D obtained from the outlined reduction methods and normalized with respect to $\bar{\sigma}_{v0}$ are given in Table III.

C-2. The data shown call for the following remarks:

C-2.a. When applied to good-quality expansion curves all presently available interpretation methods for the calculation of c_u from SBP tests lead to substantially similar results. This is self-evident from the data shown in Table IV which summarizes c_u resulting from different available interpretation approaches applied to six expansion

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TABLE III: SUMMARY OF NORMALIZED UNDRAINED STRENGTH

S i t e	Year	Probe	SBP TESTS		
			$(c_u/\bar{\sigma}_{vo})_{PH}$	$(c_u/\bar{\sigma}_{vo})_{WW}$	$(c_u/\bar{\sigma}_{vo})_D$
PORTO TOLLE	1975	L/D = 2	0.56±0.13	0.51±0.16	NC
PORTO TOLLE	1976	L/D = 2	0.47±0.07	0.45±0.08	NC
PORTO TOLLE	1979	L/D = 2	0.43±0.04	0.47±0.13	0.42±0.04
PORTO TOLLE	1979	L/D = 4	0.28±0.03	0.29±0.04	0.28±0.04
MONTALTO DI CASTRO	1976	L/D = 2	0.84±0.05	0.64±0.11	NC
BANDAR ABBAS	1977	L/D = 2	0.58±0.23	0.67±0.30	NC
GUASTICCE	1977	L/D = 2	NC	0.46	NC
TRIESTE	1979	L/D = 2	0.52±0.15	0.58±0.19	NC
TRIESTE	1979	L/D = 4	0.26±0.07	0.33±0.11	NC
DRAMMEN PLASTIC CLAY	1979	L/D = 6	0.65±0.08	0.65±0.08	0.62±0.07
ONSOY OC CLAY UPPER 5 M	1979	L/D = 6	0.70 to 0.85	1.0 to 1.4	≈0.63
ONSOY NC CLAY BELOW 5 M FROM G.L.	1979	L/D = 6	0.44±0.02	0.48±0.08	0.46±0.06
NC = not computed					

curves of excellent quality. The c_u values resulting from the graphical procedure of Baguelin et al. (1972) and Palmer (1972) have been omitted because the method was frequently observed to lead to stress-strain curves of irregular shape.

C-2-b. In most examined cases, excluding the Trieste and Porto Tolle tests carried out with the Pafisor probe which had a length to diameter ratio of 4, c_u derived from good-quality SBP tests is substantially higher than the undrained strength obtained from CK_U laboratory tests and the field vane.

This finding gets support from other researchers around the world. Ac

TABLE IV: EXAMPLES OF NORMALIZED STRENGTH AND STIFFNESS

AS OBTAINED FROM HIGH-QUALITY SBP TESTS

Site	Porto Tolle	Porto Tolle	Trieste	Trieste	Onsøy (NC)	Drammen
Depth (m)	15	15	11	18	13	10
Probe	L/D=2	L/D=4	L/D=2	L/D=4	L/D=6	L/D=6
p_o/σ_{ho}	0.98	1.02	0.99	1.19	0.99	1.00
$(c_u/\bar{\sigma}_{vo})_{GA}$	0.36	0.20	0.54	0.17	0.41	0.50
$(c_u/\bar{\sigma}_{vo})_{WW}$	0.49	0.24	0.80	0.23	0.41	0.51
$(c_u/\bar{\sigma}_{vo})_{PH}$	0.44	0.25	0.68	0.22	0.43	0.51
$(c_u/\bar{\sigma}_{vo})_D$	0.48	0.23	0.71	0.24	0.42	0.52
$(c_u/\bar{\sigma}_{vo})_{TX}$	0.31	0.31	0.32	0.32	0.32	0.40
$E_{u50}/\bar{\sigma}_{vo}$	145	330	216	69	536	397
$E_{up}/\bar{\sigma}_{vo}$	172	397	294	83	555	505
$(E_{u50}/\bar{\sigma}_{vo})_{TX}$	89±37	89±37	90±34	90±34	118	235
$(E_{u50}/\bar{\sigma}_{vo})_{DSS}$	87	87	144±52	144±52	32	65
$\epsilon_{\theta f}$ (%)	1.6	0.3	1.7	1.1	0.4	0.5
ϵ_{θ} = circumferential strain at failure from stress-strain curves obtained by means of the Prevost and Höeg (1975) approach, modified by Ladd et al. (1979)						

cording to Ladd et al. (1979), c_u derived from SBP tests should theoretically lie between the values resulting from CK_oU plane-strain compression and CK_oU DSS tests. According to Prevost (1979) and Levadoux and Baligh (1980), it should coincide with c_u from CK_oU DSS tests.

It appears that undrained strength anisotropy is not sufficient to explain the observed differences in the c_u values.

C-2-c. The stress-strain curves obtained from the SBP tests show strains to failure which are much lower than those observed in laboratory tests. This phenomenon, which is only partly documented here, appears very clearly from the experimental data presented by Ghionna et al. (1981).

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C-2-d. The above mentioned over-estimate of c_u may primarily be due to a combination of factors:

- Disturbance caused by probe installation which has two effects:
 - . p_o at the beginning of the expansion test is lower than σ_{ho} ;
 - . a cylinder of disturbed soil is created around the probe which leads to the so-called "strength paradox" [See Baguelin et al. (1972), Clough and Denby (1980), Law and Eden (1980)].

The latter point has not been explored in the research presented here. The whole concept of the so-called "strength paradox" worked out by Baguelin et al. (1972) is referring to a situation in which an annulus of disturbed soil of some thickness exists around the expanding cylindrical cavity. According to Baguelin et al. (1972), the larger the thickness of this annulus, the higher the derived strength.

The concept, however, has so far always been used with the unrealistic assumption that within the disturbed zone the total horizontal stress is still equal to σ_{ho} , and the whole question requires further numerical research.

As far as the first point is concerned, experimental evidence has been reached which shows that the condition $p_o \neq \sigma_{ho}$ has a direct and important influence on c_u derived from the SBP tests. Figure 5 clearly shows that the derived normalized undrained strength becomes larger as the value of the ratio p_o/σ_{ho} decreases.

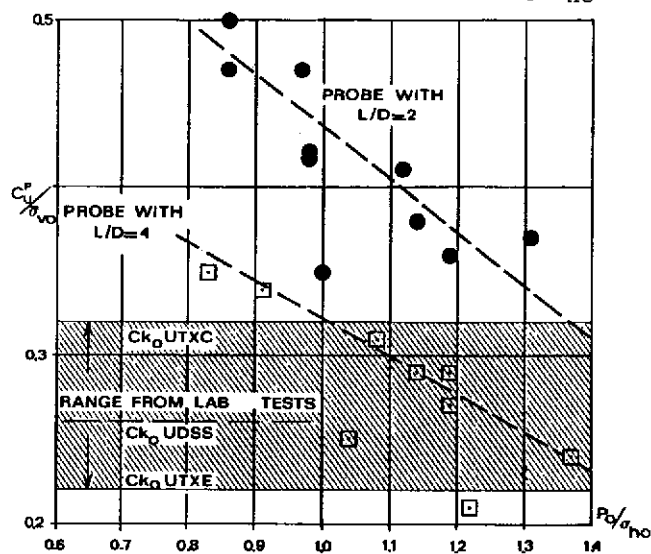


FIG. 5:
Influence of the
ratio p_o/σ_{ho} on the
normalized shear
strength.
Porto Tolle Site

c_u^p = c_u ACCORDING TO MODIFIED PREVOST AND HOEG APPROACH

p_o = HORIZONTAL PRESSURE ACCORDING TO MARSLAND AND RANDOLPH (1977)

σ_{ho} = ESTIMATED ACTUAL HORIZONTAL PRESSURE

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However, it can be seen from the data shown in Table IV that even for "perfect" tests, in which $p_0 \approx \sigma_{ho}$, the c_u (SBP) values tend to be higher than those obtained from other in situ and laboratory tests. Therefore, the above discussed effect of the adopted p_0 on the c_u values derived from SBP tests is not sufficient to completely explain the observed disagreement.

- Non-cylindrical expansion of the probe due to the finite value of the L/D ratio.
Theoretically [Vesic (1972)], the over-estimate made when assessing c_u from an expansion curve based on a spherical cavity, but using interpretation approaches based on a cylindrical cavity, will be of the order of 30%. This, however, holds for initially isotropic stress conditions; the figure applying to more realistic anisotropic initial stress conditions is not known to the writers' knowledge.
- Possibility of partial drainage occurring during the expansion test.
- Strain rate effects; see for example Prevost (1976).
- Inadequacy of the soil models and/or interpretation techniques used when computing c_u ; see for example Prevost (1979).
- Small experimental errors in the measurement of p and ΔV causing large errors in the derived stress-strain curves and c_u values.

C-3. UNDRAINED STIFFNESS

Table V shows the average of the normalized undrained Young's modulus, E_u , evaluated from SBP tests carried out at the examined sites (*). Two values of E_u have been taken into consideration:

C-3-a. E_{u50} at a stress level equal to one half of the peak shear stress, determined from the stress-strain curve obtained by Prevost and Höeg's (1975) approach, with the modification of Ladd et al. (1979).

C-3-b. The undrained pressuremeter modulus, E_{up} , evaluated directly from the expansion curve for $\epsilon_0 = \epsilon_{050}$, where ϵ_{050} denotes the circumferential strain at a shear stress equal to one half of that at failure (i.e. the stress level at which E_{u50} has been computed).

The analysis of the data shown in Table V and their comparison with undrained stiffness values measured in the laboratory permits the following comments:

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- (*) The results obtained at the Guasticce site are omitted because of the limited number of tests performed.

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TABLE V: SUMMARY OF THE MEASURED UNDRAINED STIFFNESS

Site	Year	Probe	SBP Tests	
			$(E_{u50}/\sqrt{\sigma_{vo}})_{PH}$	$E_{up}/\sqrt{\sigma_{vo}}$
Porto Tolle	1975	L/D=2	165 ± 33	NC
Porto Tolle	1976	L/D=2	234 ± 83	NC
Porto Tolle	1979	L/D=2	239 ± 72	303 ± 70
Porto Tolle	1979	L/D=4	253 ± 85	319 ± 136
Montalto di Castro	1976	L/D=2	≈ 336	≈ 322
Bandar Abbas	1977	L/D=2	231 ± 90	312 ± 120
Trieste	1979	L/D=2	104 ± 65	NC
Trieste	1979	L/D=4	61 ± 22	NC
Drammen Plastic Clay	1979	L/D=6	≈ 302	≈ 420
Onsøy OC Clay	1979	L/D=6	347 to 600	435 to 894
Onsøy NC Clay	1979	L/D=6	481 ± 73	542 to 112
NC = Not Computed				

- The E_{u50} values determined from stress-strain curves of good-quality SBP tests at the Porto Tolle, Drammen and Onsøy sites significantly exceed those which result from laboratory tests carried out on high-quality undisturbed specimens. The same applies to the undrained stiffness E_{up} computed directly from the expansion curve, although in this case the comparison with laboratory test values is more difficult since it is impossible to assess the relevant stress level for E_{up} .
- The reasons for this phenomenon are not yet clear. Since it occurs even with good-quality tests in which $p_o/\sigma_{ho} \approx 1$, see Table IV, the explanation may be found among the following reasons:
 - . the high sensitivity of E_{u50} to even a small disturbance which must cast some doubt on all laboratory determined moduli;

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- . the difference between strain rates of SBP and laboratory tests;
 - . the influence of the very different effective stress paths followed by the soil in the laboratory and the field.
- A further complication with the undrained stiffness determined from SBP tests arises because of the fact that the derived E_{u50} differs from E_{up} evaluated directly from the expansion curve for $\epsilon_0 = \epsilon_{050}$.

C-4. LIMIT PRESSURE

The limiting expansion pressure p_{lim} has been extrapolated from the expansion curves using both previously mentioned approaches [Windle and Wroth (1977) and Al-Awkati (1975)].

The experience gained in this respect may be summarized as follows:

C-4-a. The first method leads to p_{lim} values which are always 6 to 13% higher than those derived from the p vs $1/\epsilon_0$ plot.

C-4-b. p_{lim} appears to be insensitive to the varying amounts of disturbance caused by the self-boring process (causing variations in p_0).

C-4-c. At the sites of Porto Tolle and Trieste, the p_{lim} values from tests carried out with the short Pafisor probe ($L/D=2$) are very large if compared with those obtained from tests run with the longer sound ($L/D=4$), see example in Figure 6.

C-4-d. The p_{lim} values obtained from slightly or even highly disturbed tests have been found to be substantially equal to those resulting from good-quality expansion curves, so that the influence of an erroneous estimate of p_0 , and, more generally, the disturbance of the surrounding soil seem to have negligible effects on p_{lim} .

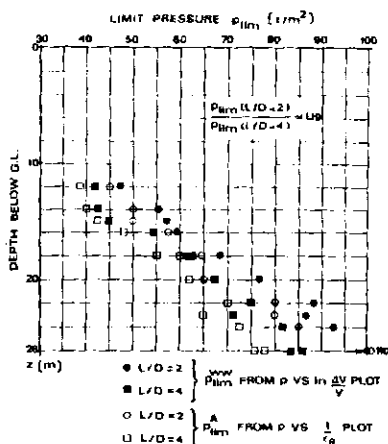


FIG. 6: Influence of the probe geometry on the p_{lim} values. Porto Tolle Site.

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C-5. IN SITU HORIZONTAL STRESS

The possibility of evaluating the existing in situ horizontal stress is certainly one of the most interesting features of the SBP tests.

The experience gained, which is summarized in Table VI, permits the following comments:

C-5-a. SBP tests without inflection point, run in soft NC and slightly OC cohesive deposits, lead to average p_o/σ_{ho} ratios ranging from 0.96 to 1.15, with σ_{ho} denoting the best estimate of the in situ total horizontal stress, whereas p_o denotes the starting pressure in tests run with the Pafsor probe and the "lift-off pressure" in tests run with the Camkometer.

The results from the Trieste site, where the p_o/σ_{ho} ratio was appreciably higher (≈ 1.3), are an exception, but this is probably due to the fact that this deposit is underconsolidated with respect to the weight of the superimposed fill and the values estimated for σ_{ho} are therefore uncertain.

C-5-b. As far as OC clays are concerned, the experience gained at the Montalto di Castro site was much less favourable since the average value of p_o/σ_{ho} was as low as ≈ 0.55 . This is due to the fact that most of the tests exhibited an inflection point. The estimate of p_o was therefore rather difficult and, moreover, the testing depths were large in this case (39 to 65 m below G.L.) which meant that the insertion of the probe and the execution of the tests caused some additional problems.

C-5-c. In nearly all available tests the adjustment of the p_o values followed the procedures suggested by Marsland and Randolph (1977). The results obtained show that this procedure does not improve the reliability of the predicted σ_{ho} when applied to good-quality tests without inflection point. However, the tests run at the Montalto di Castro site show that Marsland and Randolph's (1977) correction procedures do lead to some improvements when applied to Ménard type expansion curves with an inflection point, but that the obtained increase of the lateral total stress is not sufficient to produce agreement between p_o^{MR} and the best estimate of σ_{ho} .

At the two sites where Denby's (1978) procedure for correcting the measured p_o value was applied to Camkometer test data, the results did not allow definite conclusions.

C-5-d. The above observations show that the usefulness of the SBP test for estimating σ_{ho} , particularly in NC and slightly OC clay deposits, is supported by experimental evidence. However, in order to obtain a confirmation of this application of the probe and its extension to

TABLE VI: TOTAL IN SITU HORIZONTAL STRESS

Site	Year	Probe	L/D	p_o / σ_{ho}	p_o^{MR} / σ_{ho}	p_o^D / σ_{ho}
PORTO TOLLE	1975	PFS	2	0.90 ± 0.12	0.19 ± 0.19	NC
PORTO TOLLE	1976	PFS	2	0.80 ± 0.12	0.81 ± 0.17	NC
PORTO TOLLE	1979	PFS	2	1.05 ± 0.12	1.04 ± 0.15	0.97 ± 0.16
PORTO TOLLE	1979	PFS	4	1.12 ± 0.15	1.15 ± 0.18	1.07 ± 0.14
MONTALTO DI CASTRO	1976	PFS	2	0.55 ± 0.10	0.71 ± 0.18	NC
BANDAR ABBAS	1977	PFS	2	0.97 ± 0.15	1.07 ± 0.26	NC
GUASTICCE	1977	PFS	2	0.82 ± 0.14	0.79 ± 0.15	NC
TRIESTE	1979	PFS	2	1.29 ± 0.18	1.29 ± 0.18	NC
DRAMMEN (*)	1979	CMK	6	0.99 ± 0.06	0.98 ± 0.08	1.08 ± 0.10
ONSOY (*)	1979	CMK	6	1.00 ± 0.04	1.00 ± 0.02	1.15 ± 0.08

(*) p_o corrected for lift-off pressure

p_o = initial pressure as measured in tests run with the CMK probe corrected for lift-off pressure

p_o^{MR} = initial pressure corrected according to Marsland and Randolph (1977)

p_o^D = initial pressure corrected according to Denby's (1978) procedure; in tests run with the PFS probe this corresponds to the apparent p_o according to Denby (1978)

σ_{ho} = best estimate of the existing in situ total horizontal stress; for details see Ghionna et al. (1981)

PFS = Pafisor probe; CMK = Camkometer probe; NC = Not Computed

heavily OC deposits one will have to work on the following points:

- The improvement of the installation procedure with emphasis to be placed on a better understanding of the process in view of a further reduction of the disturbance of the surrounding soil.
- The investigation of the role of the relaxation time between insertion of the probe and the start of the test.
- The reduction of the flexibility of the measuring system and the drift of the electronic components.

Recent tests, carried out when the writers' Camkometer probes were modified, showed that the so-called lift-off pressure phenomenon may be attributed almost entirely to equipment performance. It must also be pointed out that in the above test series, in which the radial strain of the three arms was measured separately (see Figs. 2 and 3),

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each arm led to slightly different p_0 values. Even if this fact cannot be explained presently, it shows very clearly that further research is needed for a better understanding of the influence of the self-boring process on the measured p_0 .

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S. LACASSE, T. LUNNE*

IN SITU HORIZONTAL STRESS ¹ FROM PRESSUREMETER TESTS

ABSTRACT

The self-boring pressuremeter test represents a promising method to evaluate the in situ horizontal stress, provided one carefully selects and controls the installation procedures appropriate to the soil type investigated.

The paper presents the interpretation of the in situ horizontal stress from 21 self-boring pressuremeter tests in two Norwegian marine clay deposits. Eight different methods of data interpretation were compared. "Good quality" pressuremeter curves (i.e. monotonically increasing pressure, without inflection point) led to a prediction of horizontal stress within 5 to 10% of the stress believed the true in situ stress.

The satisfactory prediction of in situ horizontal stress is especially significant since a reliable "initial pressure" (P_0) is absolutely essential for derivation of undrained shear strength and shear modulus from the pressuremeter expansion curve.

I. INTRODUCTION

The evaluation of the in situ horizontal stress (σ_{ho}) represents an important but very difficult task in geotechnical engineering. Procedures such as hydraulic fracturing, earth pressure cells and laboratory testing can prove expensive, time-consuming and often subject to considerable uncertainty (Wroth, 1976; Tavenas et al., 1975;

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Ladd et al.,1977; Jamiolkowski et al.,1980). The self-boring pressuremeter offers an attractive alternative for the measurement of in situ horizontal stress, since the device also provides a complete undrained stress-strain curve in situ, with measurements of pore pressure during the test. The self-boring method is especially promising for determination of the in situ horizontal stress because, if properly performed, it tunnels itself into the ground with a minimum of disturbance.

The stress acting on the pressuremeter measurement cell after installation and the strain history of the surrounding soil influence significantly the stress-strain-strength characteristics derived from the recorded expansion curve. The achievement of the real potential of the pressuremeter in practice depends on the extent to which the installation technique affects the values of in situ horizontal stress, shear modulus and undrained shear strength and how well one can establish the installation procedures most appropriate for the material under investigation.

The paper presents the interpretation of the in situ horizontal stress from self-boring pressuremeter tests, as derived from eight methods with varying levels of complexity. The suitability and reliability of each method are discussed. The pressuremeter tests, carried out in two Norwegian soft clay deposits, are compared in terms of installation techniques, shape of field expansion curves and methods of interpretation.

II. TEST SITES AND PREVIOUS MEASUREMENTS

A. Onsøy (near Fredrikstad)

Thirteen self-boring pressuremeter tests were carried out in Onsøy clay. The very homogeneous Onsøy deposit consists of a weathered crust less than one-metre thick underlain by 8 m of soft clay with iron spots, organic matter and shell fragments and by 36 m of homogeneous soft medium-plastic clay over bedrock (Lunne et al.,1976). The natural water content varied between 60 and 65%, with an average plasticity index of 23% in the top 9 metres and 36% below 9 m. Figure 1 presents the profile, a typical result from a piezocone test (Lacasse and Lunne,1982), and the field vane undrained shear strength. The vane strength falls very near the strength obtained from constant volume simple shear tests and approximately on the average of the results from consolidated-undrained triaxial compression and extension tests (Lacasse et al.,1981). Based on field vane tests, the average sensitivity of 7 in the top 9 m decreased to 5 below this depth.

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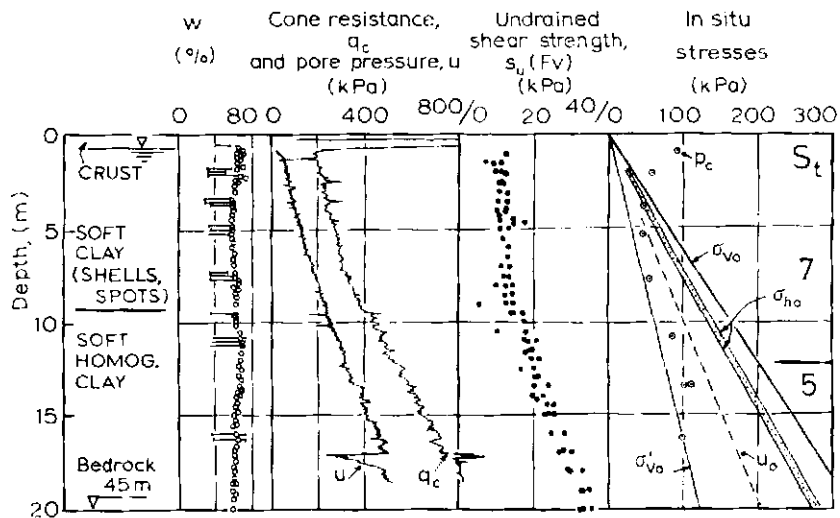


Figure 1. Onsfjord test site.

Figure 1 also presents the stress history at the site. The in situ pore pressure was measured as five percent larger than hydrostatic at a depth of 20 m. Values of preconsolidation stresses (p_c) based on one-hour incremental loading oedometer tests indicated an overconsolidation ratio decreasing from 15 to 1.3 between 2 and 6 m and constant at 1.3 below 6 m.

The figure also shows the most plausible in situ horizontal stress, based on previous hydraulic fracturing tests in situ (Berre, 1974; 1982) and on the following empirical correlations:

- 1) Brooker and Ireland's (1965) relationship among the coefficient of earth pressure at rest (K_0), plasticity index (I_p) and overconsolidation ratio (OCR).
- 2) Schmidt's (1966) relationship supplemented by the results presented by Ladd et al. (1977), who defined the K_0 -value at a given OCR (K_0^{OC}) as a function of the K_0 for normally consolidated clay (K_0^{NC}), the overconsolidation ratio and exponent m . The values of K_0^{NC} and m depend on the plasticity index. The relationship states (unloading conditions only):

$$K_0^{OC} = K_0^{NC} \cdot OCR^m \quad (\text{Eq. 1})$$

- 3) Massarsch's (1979) relationship for normally consolidated clays with plasticity index between 20 and 80%.

For Onsfjord clay, the normally consolidated K_0 -value was taken as 0.55 ± 0.01 and 0.60 ± 0.2 for plasticities of 23 and 36%. The m -

factors were of 0.37 and 0.40 respectively. At an OCR of 3, the reference K_0 -values ranged between 0.8 and 0.9. For an OCR of 1.3, K_0 -values ranged between 0.56 ± 0.02 ($I_p = 23\%$) and 0.65 ± 0.03 ($I_p = 36\%$).

The K_0 -values from hydraulic fracturing tests tended to yield the lower bound of σ_{ho} (in Fig.1), whereas the K_0 -values based on Schmidt's relationship gave the higher σ_{ho} . The uncertainty in the reference σ_{ho} is only $\pm 3\%$ about the average.

B. Drammen (Danvikgate site)

The eight self-boring pressuremeter tests carried out in Drammen investigated a 5-m thick plastic clay layer lying directly above a 35-m thick layer of lean clay. Sand and silty clay are encountered in the top 5 m (Lunne et al., 1976). The natural water content varied between 50 and 55% with an average plasticity index of 28%. The sensitivity, measured by the field vane, averaged 7. Figure 2 presents the profile, the results from a typical piezocone test (Lacasse and Lunne, 1982), and the undrained shear strength from field vane tests. The average field vane strength approached the triaxial compression strength at 5 m, but came nearer to the simple shear strength at 10 m.

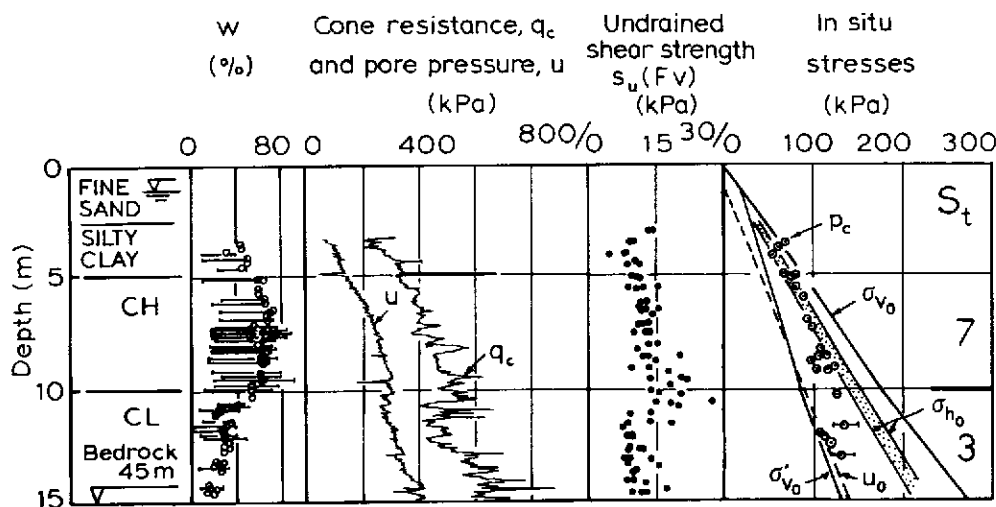


Figure 2. Drammen test site.

The in situ pore pressure was approximately hydrostatic, with the water table one meter below ground surface. The preconsolidation stresses from incremental loading oedometer tests, plotted at the end of primary consolidation, indicated an overconsolidation ratio of 1.5. Figure 2 also shows the most plausible in situ horizontal stress based on previous field and laboratory measurements and the empirical corre-

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lations mentioned before. The in situ measurements included hydraulic fracturing tests and total stress cell measurements (Bjerrum and Andersen, 1972; Berre, 1973). The laboratory measurements were made in a K_0 -triaxial cell.

For Drammen clay ($I_p = 28\%$), the value of K_0 for normally consolidated clay was taken as 0.57 ± 0.02 , with an m -value of 0.39. The K_0 -values determined by the above methods for an overconsolidation ratio of 1.5 ranged between 0.61 and 0.75. The uncertainty in the expected in situ horizontal stress (see Fig. 2) is $\pm 5\%$ about the average. The measurements from the in situ stress cells led to the lower σ_{ho} in Figure 2, whereas hydraulic fracturing (down to 9 m) and Schmidt's correlation (between 9 and 10 m) suggested K_0 -values yielding the higher reference horizontal stress.

III. PRESSUREMETER INSTALLATION PROCEDURES

The tests were carried out with the Cambridge self-boring pressuremeter Mark IV unit, with an 8-cm diameter and 48-cm long measurement cell (Wroth and Hughes, 1973; 1974).

The cutting tool configuration and its location within the cutting module, the volume of wash water and location of the jet holes, and the force applied to the central drill rod to control the rate of advance can be varied to achieve minimal soil displacement around the probe during the self-boring process. Considerable experience is required at each new site before appropriate drilling procedures can be established. Since soil displacements cannot be measured during self-boring, one must evaluate trial tests to ascertain which methods appear to give the best results (Baguelin et al., 1978).

The position of the chopping bit should prevent flow of soil into the shoe, which otherwise could cause disturbance. An advanced position of the bit also facilitates drilling in stiff soil. The rate of advance of the probe has to be regulated to prevent clogging, and flushing rates should properly balance the advance rate, cutter speed and soil conditions. Proper operation of the pressuremeter system during installation results in smooth advance of the drilling tool, and approximately constant flushing pressures and flow rate. The flow rate decreases when clogging occurs and flushing pressures must be increased to break up the material clogging the hole. High pressures can disturb or even hydraulically fracture the soil.

In Onsøy and Drammen clays, the pressuremeter installation technique involved the following procedures. During probe insertion, the rubber membrane was held firmly against a thick-walled tube via a

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partial vacuum such that the outside diameter of the membrane was the same as the diameter of the cutting head. Table 1 lists the drilling parameters used and compares them to the procedure adopted by Clough and Denby (1980) in their investigation San Francisco Bay Mud, a soft, nearly normally consolidated clay with natural water content 90%, plasticity index 50% and liquidity index unity (see also Denby,1978).

Table 1. Self-boring procedures

Drilling characteristics	Present test program	San Francisco Bay Mud (Clough and Denby,1980)
Retraction of chopper inside of cutting shoe	3 cm	2.5 cm
Rate of advance, last meter above test elevation	Onsøy 2 cm/min Drammen 1.5 cm/min	2.5 cm/min
Hydraulic pressure	300-500 kPa	140-560 kPa
Water flow	10-12 l/min	18 l/min
Rotation of surface motor	Onsøy 120 rpm Drammen 60 rpm	50-60 rpm

The cuttings recovered from the self-boring operation provided an indication of the material cut. Invariably, the cuttings in the first 20 - 25 cm below testing depth were soft and without well defined contours. They represented the material disturbed by the expansion test performed immediately above. Below this depth, the cuttings from the "virgin" clay were small, firm and round, indicating a reduced remoulding.

When the instrument reached the proposed test elevation, the field operator released the vacuum, and monitored the pore pressure with time. The expansion test should begin only after stabilization of the pore pressure. The "relaxation" times varied between 90 and 1300 minutes. Several hours are often required for stabilization, although shorter relaxation periods are commonly used in practice.

The pressure-displacement data points were initially recorded manually. Subsequently, a multiple channel stripchart recorder was coupled into the pressure and displacement sensors. The continuous records provided a much better definition of the initial portion of the expansion curve.

IV. RECORDED EXPANSION CURVES

The proportion of successful completions of the pressuremeter expansion tests depends on the presence of coarse sand, gravel and shells. In the homogeneous Onsøy deposit, 12 out of 13 tests were run to completion. In the Drammen deposit, 6 out of 8 tests were successful. Figure 3 summarizes the field pressuremeter curves* obtained. Lacasse et al. (1981) analysed the pressuremeter results in terms of undrained shear strength and shear modulus. The expansion tests gave derived undrained strength and Young's modulus values about twice those determined from laboratory tests and cone and vane tests. The results varied with the initial pressure assumption.

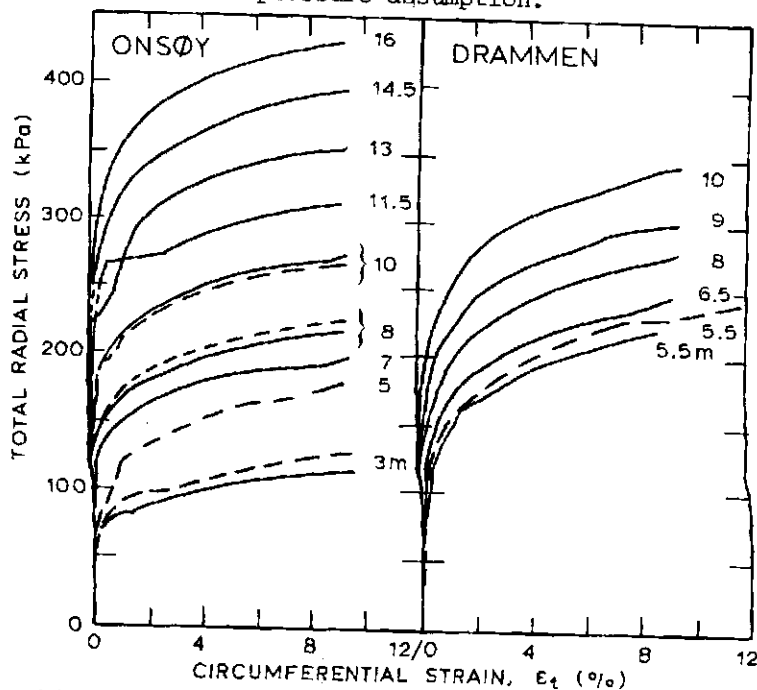


Figure 3. Expansion curves from self-boring pressuremeter tests.

A number of the Onsøy expansion curves (Fig.3) exhibited slightly anomalous behaviour. Sections V and VI discuss predicted horizontal stress as a function of the quality of the expansion curve.

V. HORIZONTAL STRESS INTERPRETATION METHODS

Several methods are currently available for evaluating the in situ

* The tests were carried out without the aid of the membrane-protective "Chinese lantern".

horizontal stress. Some were developed for interpretation of Ménard pressuremeter tests, and not for self-boring results. They are, however, commonly used for the latter. Three classes of interpretation methods exist: (A) approaches related to the direct measurement of the "initial" or "lift-off" pressure; (B) empirical method; and (C) approaches that use the complete pressuremeter curve to determine the initial pressure. The following paragraphs describe the different methods available and Figure 4 illustrates the application of the methods to the test at a 10 m depth in Drammen clay.

A. Lift-off related methods (direct measurement of P_0)

1. INSPECTION OF EXPANSION CURVE

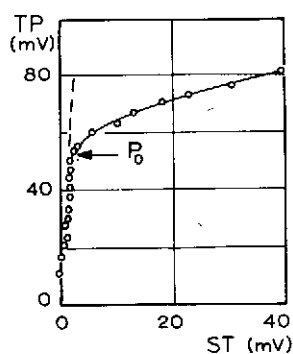
One defines the "lift-off" pressure either by visual inspection of the field data and selection of the initial pressure, P_0 , at the first movement of the membrane, or by plotting expansion pressure versus displacement and taking the initial pressure at the point in the curve with a marked change in slope (Fig.4a). The initial pressure method assumes that the radial or lift-off pressure at the beginning of the expansion test after an adequate relaxation period corresponds to the in situ horizontal stress. The analyst's subjectivity, the shape of the expansion curve, system compliance, non-uniformity of the radial strains, varying degrees of sample disturbance and uncertainties in the shear stresses along the measurement cell circumference may however affect the reliability of the method.

2. INITIAL DEVELOPMENT OF EXCESS PORE PRESSURE

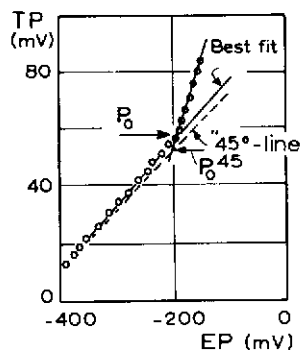
Wroth and Hughes (1974), Hughes et al. (1975) and Wroth (1980) mentioned this method, which assumes that the horizontal stress corresponds to the inflation pressure at which excess pore pressures first develop. The method however reflects the pore pressure generation in a very limited area of the stressed soil.

Wroth (1980), in a very detailed example, showed that one should expect a one-to-one (45°) relationship between expansion pressure and the effective stress cell pressure until excess pore pressures start to develop. At "lift-off", the effective stress cell pressure line diverges from the expansion pressure line. It is thus possible, by plotting expansion pressure and the effective cell pressure readings, to define an initial pressure, P_0 , at the point of significant change in slope (Fig.4b). The method involves some subjectivity. The reliability of the measured pore pressures can be assessed in some cases from the shape of the effective stress path.

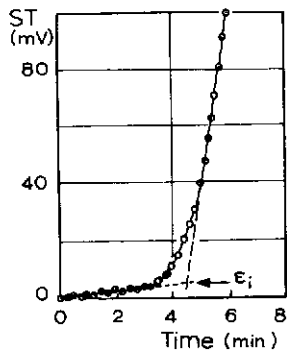
IN SITU HORIZONTAL STRESS FROM PRESSUREMETER TESTS



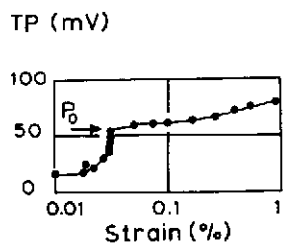
(a) Inspection



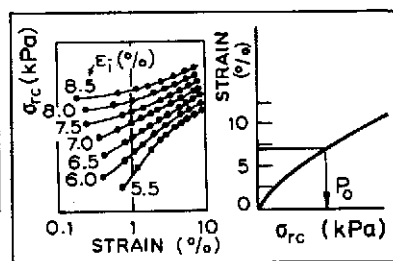
(b) Initial excess pore pressure (Wroth, 1980)



(c) Modified lift-off (Lacasse et al, 1981)



(d) Log (strain) (Law and Eden, '82)



(e) Origin adjustment (Denby & Hughes, 1982)

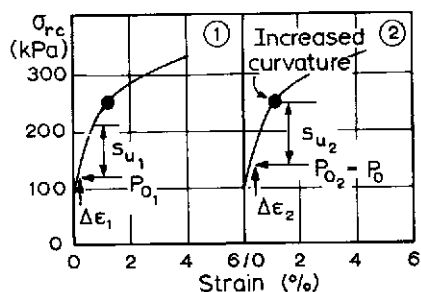
LEGEND

TP: Total pressure (mV)

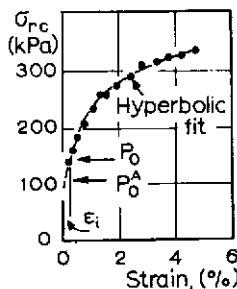
EP: Effective pressure (mV)

ST: Displacement (mV)

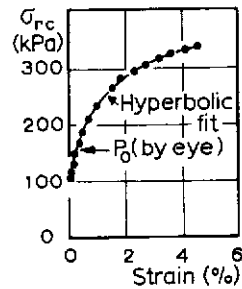
σ_{rc} : Corr. inflation pressure (kPa)



(f) Graphical iteration (Marsland & Randolph, '77)



(g) Hyperbolic-plastic model (Denby, '78)



(h) Double zero shift (Arnold, '81)

Figure 4. Interpretation of horizontal stress in self-boring pressuremeter test (Drammen, test at 10m).

Errors (or changes) in the sensor calibration factors and delays in pore pressure response due to incomplete saturation of the filter(s) can lead to deviation in the predicted lift-off pressure if one tries to use a 45° -relationship between expansion pressure and effective stress cell pressure. Wroth rightly stated that the 45° -line is theoretically correct, but it holds true only for a perfectly responding system. Figure 4b compares the initial pressures obtained by fitting a 45° -line* and obtained directly from the data. The difference can prove significant in some tests. It is thus recommended, when using the initial excess pore pressure approach, to plot directly the total pressure reading versus the effective stress cell pressure reading, and to define lift-off at the intersection of the two straight lines, without attempting to fit a 45° -line between expansion pressure and effective stress cell pressure.

3. MODIFIED LIFT-OFF PRESSURE

The very early movement of the membrane does not necessarily correspond to a reliable estimate of σ_{ho} . Denby (1978) first noted this possibility and developed an extrapolation method for evaluating σ_{ho} (see part C below). Lacasse et al. (1981) defined a modified "lift-off" pressure as the pressure at the start of a linear variation of radial strain with time (Fig.4c). The method corrects in this manner for an initial (false) strain on the circumference of the hole. The early movement of the membrane may be due to slight overdrilling of the hole and/or deformability of the feelers (Dalton and Hawkins, 1982). It is however believed that the averaging of the displacement of the three feelers, as done with older versions of the Cambridge pressuremeter, can mask the precise location of the break in the strain-time curve.

4. LOGARITHM OF STRAIN METHOD

Law and Eden (1982) defined the initial pressure, P_0 , corresponding to the in situ horizontal stress at the break in the expansion pressure versus log strain curve (Fig.4d). This method in fact comes very close to the inspection method as it defines P_0 as the pressure required to just lift the membrane off the probe.

Two additional methods, related to the "lift-off" point are available. They have been developed especially for the interpretation of tests in predrilled holes or self-boring pressuremeter expansion curves with inflection point. The following describes briefly the methods, but they were not applied to the self-boring tests in Onsøy and Drammen.

* The different calibration factors of the total and effective pressure cells have been accounted for.

5. INFLECTION POINT

The inflection point is defined as the break in the expansion curve where reloading ends and loading starts. The method generally applies to test results with significant disturbance due to the drilling procedure (Ladd et al., 1979; Ghionna et al., 1981; Denby and Hughes, 1982).

6. INVERSE VOLUME

Van Wambeke and d'Hemricourt (1975) proposed the inverse volume method where one plots pressure versus the reciprocal of the volume change. The initial pressure is taken at the start of the first linear segment of the inverse curve. The method, little used, and developed mainly for Ménard-type expansion curves, overpredicted significantly the in situ horizontal stress from Cambridge pressuremeter tests in Boston Blue clay with overconsolidation ratio between 1 and 3 (Ladd et al., 1979).

B. Empirical method

For an elasto-plastic soil, the horizontal stress, σ_h , is obtained from the limit pressure, P_L , the undrained shear strength, s_u , and an empirical constant, N_p (Gibson and Anderson, 1961):

$$\sigma_h = P_L - N_p s_u \quad (\text{Eq. 2})$$

The constant N_p depends on the soil type and the methods used to determine N_p and s_u . The constant N_p is expressed as $(1 + \ln \frac{E_u}{3s_u})$, where E_u is the undrained Young's modulus. Since the ratio E_u/s_u for most clays falls between 100 and 2000 (Ladd et al., 1977), N_p should fall within the range 4.5 to 7.5. According to Ladd et al. (1979), the undrained shear strength derived from self-boring pressuremeter tests should theoretically lie between the values from undrained plane strain compression and constant volume simple shear tests. Prévost (1979) and Levadoux and Baligh (1980) believed that the pressuremeter s_u should coincide with simple shear results. Since no plane strain tests were available for Onsøy and Drammen clays, the N_p -value for each deposit relied on results from K_0 -consolidated triaxial compression (CK_0UC) and simple shear (DSS) tests. Table 2 summarizes the resulting N_p -values. However, as described by Lacasse et al. (1981), the interpretation of the complete expansion curve via Eq. (2) with an N_p -factor of 6 resulted in strengths much higher than all existing reference strengths for these two deposits.

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Table 2.

Empirical pressuremeter factor N_p

Site/ Test type	Depth (m)	N_p	
		CK ₀ UC	DSS
Onsøy	2-6	5.3	6.2
	7-17	6.1	6.1
Drammen	5-10	6.3	6.3

The empirical approach suffers from a number of drawbacks. The horizontal stress derived in this manner is very sensitive to the values of limit pressure, factor N_p and undrained shear strength. The limit pressure generally exhibits a wide variability. Factor N_p may not vary appreciably for a given soil, but it may be problematic to assign a reliable value to it. The appropriate reference strength is also rather uncertain.

C. Methods using pressuremeter expansion curve

1. GRAPHICAL ITERATION

Marsland and Randolph (1977) developed a graphical iterative method, which uses the Gibson and Anderson elasto-plastic soil idealization. The procedure, first developed for application to Ménard pressuremeter data in stiff clays, derives the undrained shear strength, s_u , from an assumed value of P_0 and iterates until the value of $P_0 + s_u$ corresponds to the pressure where initial yield occur (Fig.4f). Initial yield is defined at the end of the linear portion of the expansion curve. The Marsland-Randolph method uses the s_u -value derived from the cavity expansion approach proposed simultaneously by Baguelin et al.(1972),Ladanyi (1972) and Palmer (1972).

2. HYPERBOLIC-PLASTIC MODELING OF EXPANSION CURVE

Denby (1978) concluded that the determination of the in situ horizontal stress as corresponding to the very early movement of the membrane was unsatisfactory. He then proposed two definitions of the initial pressure, P_0 (refer also to Denby and Clough,1980). Figure 4g illustrates these two initial pressure definitions.

- "Apparent" initial pressure, P_0^A , taken by extrapolation to zero strain of the pressuremeter curve modeled with a hyperbolic-plastic relationship.
- "Corrected" initial pressure, P_0 , accounting for early membrane movement. Denby introduced an initial strain to account for compliance of the feelers and any disturbance due to the probe installation process. The pressure at this initial strain, calcu-

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lated from the fitted hyperbola equation, corresponds to the in situ horizontal stress. The method obtains explicitly the corrected initial pressure by solving the hyperbolic expression of the pressuremeter curve. It has the advantage of standardizing the procedure by eliminating most of the subjectivity involved in other methods. Denby however did not clarify how one should select the initial strain to account for early membrane movement.

3. DOUBLE ZERO SHIFT OF HYPERBOLIC MODEL

Arnold (1981) proposed a simplified hyperbolic modeling of the expansion curve. The method corrects for a zero shift on both the expansion pressure- and the strain-axis. The test data is first fitted to a hyperbolic expression via three points on a smooth curve (Fig.4h). The experimental starting point of the hyperbolic curve, defined as P_0 , is then determined by inspection. The strain at pressure P_0 corresponds to the zero shift on the strain axis. The method involves some level of subjectivity and depends to a large extent on how well the hyperbolic method fits the data.

4. ORIGIN ADJUSTMENT

Denby and Hughes (1982) explained the origin adjustment method, based on the theoretical linearity of the plot of inflation pressure versus the logarithm of radial strain for an elastoplastic material (Windle and Wroth, 1977). The approach adjusts the strain origin until a best elastoplastic fit of the pressuremeter curve is achieved. If a family of expansion pressure versus radial strain curves are plotted, and corrected for different initial strains, the corrected initial strain for the most linear curve corresponds to the horizontal stress and the slope corresponds to the shear strength (see Fig. 4e). The method generally applies to pressuremeter tests in predrilled holes. In the case of the tests in Onsøy and Drammen clays, the displacements occurring before the linear portion of the expansion curve were too small to enable use of the origin adjustment interpretation method.

VI. DERIVED HORIZONTAL STRESS FROM SELF-BORING PRESSUREMETER TESTS

A. Comparison of results from the different interpretation methods

The Onsøy and Drammen self-boring pressuremeter tests were interpreted according to eight of the methods described in the previous section.

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Figures 5 and 6 compare the reference in situ horizontal stress to the values predicted by the self-boring pressuremeter tests. The σ_{ho} profiles denoted "in situ" represent the best estimate of horizontal stress predicted prior to the start of the test program. The measurements and correlations described in Section II enabled this estimate, considered to be realistic.

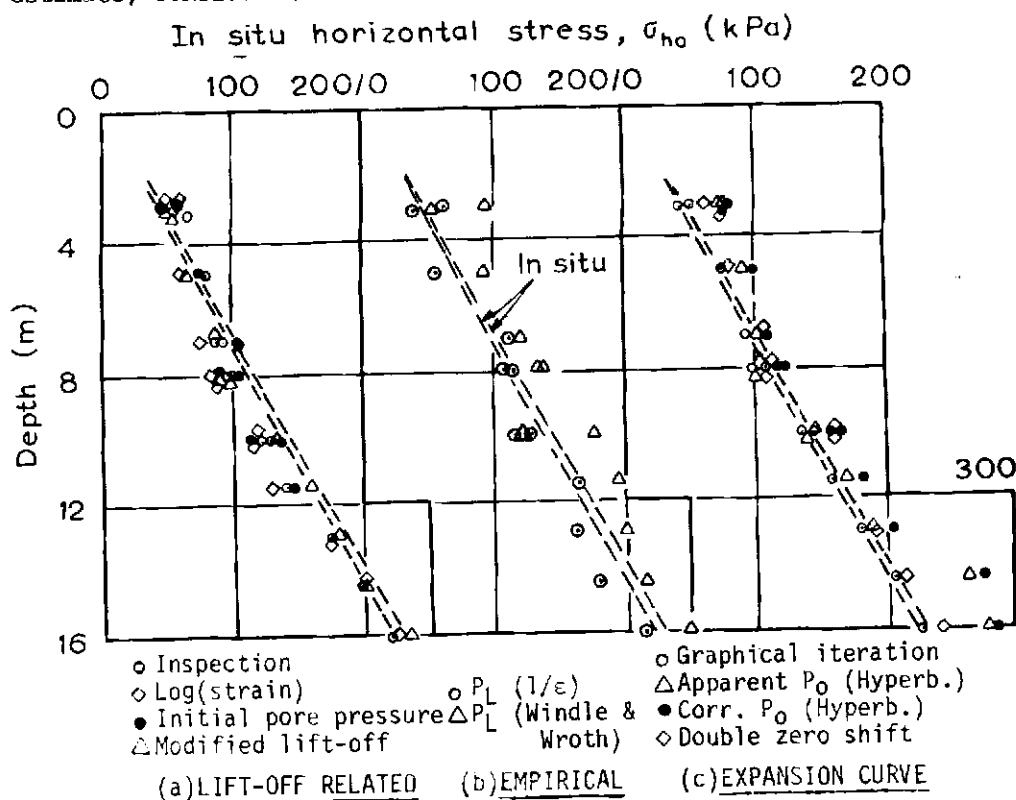


Figure 5. Predicted horizontal stress in Onsøy.

Table 3 lists for each interpretation method the deviations of the predicted stress from the reference in situ horizontal stress at each site. The table indicates approximately equal reliability for several methods, but several comments need to accompany the results shown. Moreover, the interpretation of the pressuremeter tests at the two different sites led to different conclusions with respect to the reliability of some of the methods.

The four lift-off pressure related approaches (Figs. 5a and 6a) generally underpredicted the reference in situ stress, except for the excess pore pressure method in Drammen and the modified lift-off method at both sites.

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Table 3. Reliability of horizontal stress from self-boring pressuremeter tests in two soft clays.

Interpretation method		Onsøy P_o/σ_{ho}		Drammen P_o/σ_{ho}	
		Ave	Std Dev	Ave	Std Dev
Lift-off related	Inspection	0.97	0.13	0.90	0.09
	Initial excess pore pressure	0.99*	0.11	1.01	0.06
	Modified lift-off	0.96	0.08	0.96	0.05
	Log (strain)	0.85	0.12	0.81	0.14
Em-pirical	Empirical (P_L from Windle & Wroth)	1.25	0.30	0.88	0.15
Derived pressuremeter curve	Graphical iteration	0.98	0.04	1.01	0.05
	Hyperbolic-plastic model				
	Apparent P_o	1.04	0.11	0.81	0.02
	Corrected for initial strain	1.13	0.23	1.00	0.04
	Hyperbolic model				
	Double zero shift	1.05	0.11	0.99	0.06

P_o = Horizontal stress derived from interpretation method

σ_{ho} = Previous best estimate of in situ horizontal stress

* Average does not consider three deeper tests which gave absolutely unreasonable results.

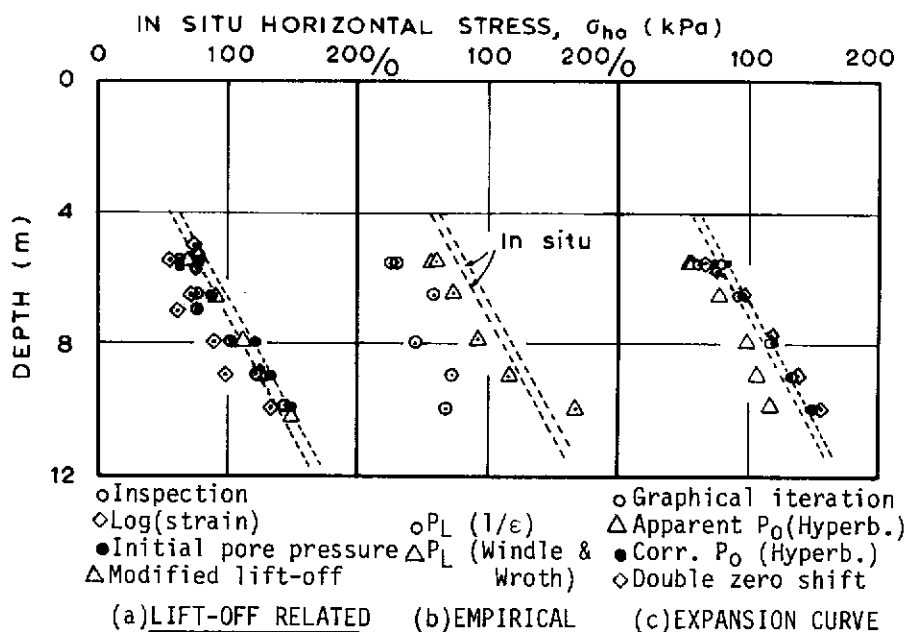


Figure 6. Predicted horizontal stress in Drammen.

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The inspection of the pressuremeter curve generally yielded too low stresses, except for the tests between 5 and 13 m in Onsøy.

The log of strain method plots the early movement of the membrane in a different manner and predicts even lower stresses than the inspection method. Law and Eden (1982) noted that the break in the pressure versus log(strain) curve may not appear well defined. However, for most of the tests in Oslo and Drammen, the break in the curve was sharp.

The fact that the inspection and log(strain) methods gave satisfactory results for some tests probably reflects the minimum of disturbance that occurred during installation.

The excess pore pressure method predicted satisfactorily the in situ horizontal stress at both sites (slightly increased scatter in Onsøy). The method was however inapplicable in the three deeper tests at Onsøy as no linear variation of effective and total pressures was obtained. If one attempted to estimate P_0 from these tests, the resulting values were about 50% too low. For the other tests, the possible uncertainty due to subjectivity, based on experience at these two sites, is rather minor, averaging perhaps 5% on P_0 for all tests. Differences in predicted P_0 between a "45°-line" and a "tangent-line" interpretation, as shown in Figure 4b, can prove more important. While the difference between the two slopes in Figure 4b appears very small, it could lead to decreases in the predicted horizontal stresses of about 10% in Onsøy and 15% in Drammen.

The modified lift-off method, an alternative to the excess pore pressure method, led to consistent σ_{ho} values, plotting generally near the average or the lower bound of the expected in situ reference value. The method is subjective, and a range of values was obtained to illustrate the possible change in predicted σ_{ho} due to this subjectivity. The predicted σ_{ho} covered a range of about $\pm 5\%$ about the average of both sites. The method has the advantage of defining an initial (false) strain which can later be used in the Denby and Clough (1980) modeling. In Onsøy, the modified lift-off pressure approach indicated initial strains between 0.03 and 0.08%. In Drammen, the initial strains ranged between 0.06 and 0.09%. The magnitude of the initial (false) strain reflects not only the compliance of the feelers and soil type, but also the quality of the probe installation technique.

The empirical approach led to an inconsistent evaluation of horizontal stress. The calculations in Onsøy and Drammen used the limit pressures from the inverse strain plot (resulting in the lower σ_{ho}) and from the Windle and Wroth (1977) approach (resulting in the upper σ_{ho} shown in Figs. 5b and 6b). The N_p -factors in Table 2 and the Prévost and Hæg (1975) derived strength (with the Marsland and Randolph σ_{ho}) were used. If a strength other than that derived from

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pressuremeter tests had been used in Eq.1 (for example the simple shear strength), the horizontal stress would have been overpredicted by a factor of 1.5 to 3.0.

The sensitivity of the evaluated σ_{ho} to limit pressure, factor N_p , and reference undrained shear strength makes the method unreliable. It is thus believed unsuitable for prediction of horizontal stress. Denby and Hughes (1982) arrived at the same conclusion.

The pressuremeter expansion curve methods of Marsland and Randolph (1977), Denby (1978) and Arnold (1981) led to good results. The first approach agreed amazingly well with the reference in situ stress, the standard deviation of the predicted σ_{ho} was within the magnitude of uncertainty at each site. One test (5.5 m in Drammen) yielded a predicted σ_{ho} 15% too low, but the discrepancy is due to an unreliably high strength derived from this test.

The graphical iteration method was developed to improve the prediction of horizontal stress from expansion tests of reduced quality (i.e. curves with inflection point). The method does appear to also slightly improve the prediction of σ_{ho} from good quality expansion curves (compare with the modified lift-off and excess pore pressure results).

The hyperbolic-plastic method by Denby yielded higher horizontal stresses than the iterative approach. In Onsøy, the "apparent" initial pressure, slightly overestimating the σ_{ho} , gave a more satisfactory prediction than the "corrected" initial, whereas the opposite holds true in Drammen: the "apparent" initial pressure is consistently 20% too low. Initial strains of the magnitude of those provided by the modified "lift-off" method were used in the Denby method.

For the averages presented in Table 3, the test results at a depth of 3 m in Onsøy were disregarded in the case of hyperbolic modeling methods. Neither the Denby nor the Arnold double shift methods seemed to be able to handle these tests.

The hyperbolic double shift method (Arnold, 1981) predicted horizontal stresses near the reference values at both sites. The results depended however on the hyperbolic constants chosen in the fitting process. If one modeled closely the beginning of the pressuremeter curve (up to strains of 2%), one obtained the results shown in Figures 5c and 6c. If on the other hand, one modeled more closely the pressuremeter curve at large strains, the resulting P_0 was much higher (approximately equal to Denby's corrected values).

The above test results established that for the two soft marine clays, several methods were suitable for deriving the in situ horizontal stress from pressuremeter tests, but that their suitability may well be site-dependent and a function of the type of expansion curve.

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B. Considerations in horizontal stress prediction

Four factors are briefly discussed here: shape of expansion curve, relaxation time, equilibrium pore pressure, and rate of advance cutting tool. Law and Eden (1980;1982) presented the effect of cutting shoe size on measured initial pressure.

The Marsland and Randolph (1977) graphical iteration procedure for determining the initial pressure improved slightly on the predicted horizontal stress in the case of "non-perfect" shape of expansion curves (for example tests at depths of 3, 5 and 11 m in Onsøy). In the case of self-boring pressuremeter tests with good quality expansion curves (no inflection point), the iterative procedure may not lead to results different from the "lift-off" related procedures.

Figure 7 presents an example of the predicted horizontal stress as a function of the relaxation time used in Drammen. The clay has a low permeability and shorter relaxation times may well affect the predicted P_0 , especially in the case of the initial excess pore pressure method. The data in Figure 7 indicate however that relaxation time appears to have little effect on the predicted horizontal stress. A relaxation time of over four hours was required for agreement of the measured pore pressure and in situ pore pressure within $\pm 5\%$.

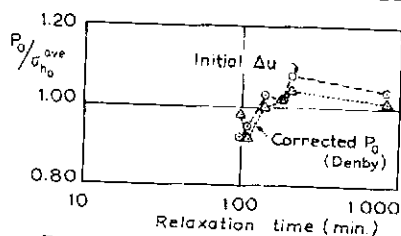


Figure 7. Example of predicted horizontal stress and relaxation time in Drammen.

The rate of advance of the cutting tool in the last meter above test depth was doubled in the case of a few isolated cases (for examples tests at 14.5 and 16 m in Onsøy, test at 8 m in Drammen). This slight increase in cutting rate does not appear to have affected significantly either initial strains or predicted horizontal stress. It may however affect the pore pressure developed during the expansion test if relaxation times are not sufficiently long.

VII. CONCLUSIONS

Since several uncertainties still exist with respect to the interpretation of the in situ horizontal stress from pressuremeter tests, it is recommended to compare the results from different methods of interpretation. The use of several interpretation methods proves advantageous as it provides insight in the reliability of the individual

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parameters (total pressure, pore pressure, displacement) in each test. The interpretation should include results based on direct measurements and results derived from the complete pressuremeter curve.

The following five approaches provided satisfactory estimates of the horizontal stress from self-boring pressuremeter tests: the initial excess pressure method (Wroth and Hughes, 1974; Wroth, 1980); the modified lift-off pressure method (Lacasse et al., 1981); the graphical iterative method (Marsland and Randolph, 1977); and the hyperbolic approaches of Denby (1978) and Arnold (1981). These methods predicted horizontal stress values deviating by no more than $\pm 10\%$ from the average plausible in situ horizontal stress.

The modified lift-off pressure method defined an initial strain to account for system compliance and inevitable disturbance. The magnitude of the strain can constitute an approximate quantitative control of the quality of the drilling technique. Further research should attempt to define an acceptable upper bound for the initial strain. The present results indicated that an initial circumferential strain of 0.1% may be acceptable. Earlier tests by Baguelin et al. (1972) showed that a circumferential strain of 0.25% was probably too large for a reliable determination of in situ horizontal stress.

For the clays investigated, neither relaxation times as short as 90 minutes nor advance rates as large as 4 cm/min appeared detrimental to the reliability of the predicted horizontal stress.

The compliance of the electrical feelers in the Cambridge self-boring pressuremeter equipment is of significance with respect to prediction of horizontal stress (Dalton and Hawkins, 1981; Denby and Hughes, 1982; Ghionna et al., 1982). Independent measurements of stress from each of the three feelers instead of an internally-averaged measurement should improve the reliability of the horizontal stress derived from the self-boring pressuremeter.

The results shown and the conclusions drawn are based on test results in two soft clay deposits and may not apply to stiff overconsolidated clays.

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CYCLIC TESTS WITH THE SELF-BORING PRESSUREMETER

ABSTRACT

In the paper entitled the "The Self-Boring Pressuremeter" presented at this symposium a detailed description is given of the Self-Boring Pressuremeter Model 76 (PAF 76) along with the soil properties that this apparatus makes it possible to measure during a monotonic expansion referred to as the "reference test".

The present paper presents the results of more complex loading tests carried out with the same apparatus and involving cyclic loads of several types. The test device is first described. The measured parameters are the defined, after which the most characteristic experimental data are given.

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It may be said that up to the present time the cyclic loading of soils has been carried out almost exclusively in the laboratory on undisturbed or remoulded soils. Few investigations have been carried out in the field in this respect because existing conventional equipment does not lend itself to this type of test, be it the vane test apparatus or penetrometers of the static or dynamic type. For the Menard pressiometric test the problem is somewhat different. The cylindrical expansion test lends itself well in its principle to cyclic loading. Unfortunately, the technology of the Menard pressuremeter is not very well suited to this type of test which requires good control of the deformation of the measurement cell, good knowledge of the

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The Self-Boring Pressuremeter Model 76 (PAF 76), and especially its expansion system called the "controlled deformation pressure-volume controller" (CPV), make it possible to overcome these difficulties, thus making the apparatus ideal for in-situ investigations of the behaviour of soils under cyclic loading.

I - ADAPTATION OF PAF 76 TO CYCLIC LOADING

Figure 1 shows the measurement setup necessary for investigating cyclic loads. This measurement system is required by the need to carry out (at least in the present phase of research) very precise measurements involving relatively long testing periods ruling out any "manual" control of operations. The operator (after having selected a loading mode) confines himself to checking the proper execution of the test, which is controlled by a programmable computer.

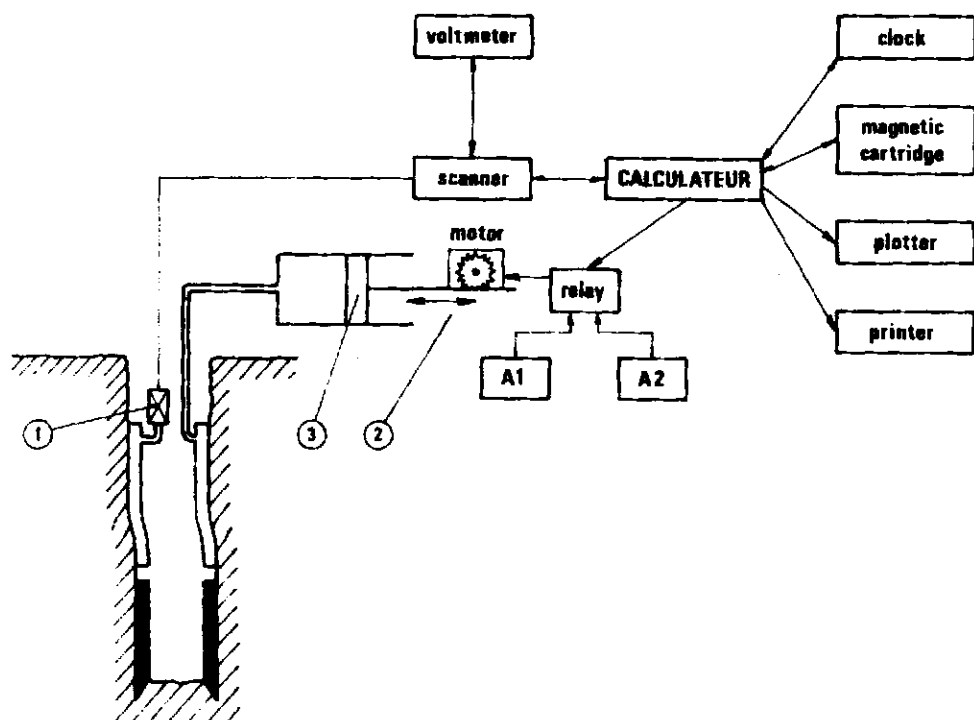


Figure 1 - The Self-Boring Pressuremeter Model 76
Schematic diagram of measurement setup for cyclic loading.

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The computer is linked, firstly, with output peripherals (plotter, printer, etc.) thereby allowing real-time plotting of the test curve and, secondly, with the CPV of the pressuremeter that it controls when desired for forward operation or reversing by action on relays. These relays link up the electric motor with the stabilized power supplies (A_1) for forward operation and (A_2) for reversing. The computer also carries out the pressure measurements in the sonde (or possibly pore pressure measurements) by means of electrical transducers (1) as well as the displacement of the CPV piston by the reading of electrical comparators (2) measuring the displacement of this piston (3).

The measurement data are then stored on a magnetic cartridge for more thorough subsequent interpretation and for filing.

II - DEFINITION OF PROPERTIES MEASURED

Figure 2 shows two pressiometric curves ($a = \Delta V/V_0$, p) :

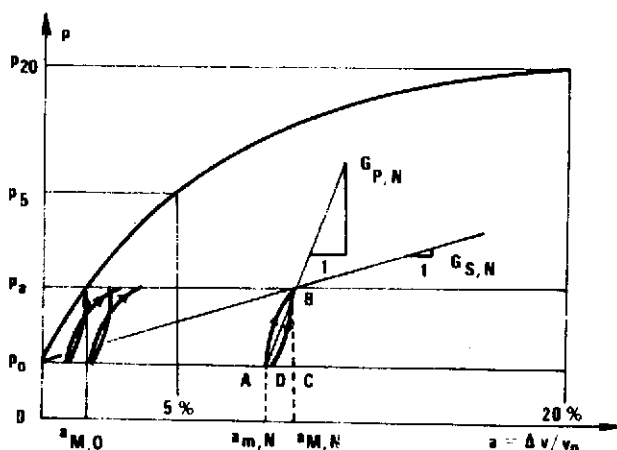


Figure 2 - Typical cyclic loading curve : definition of parameters measured.

- A monotonic expansion curve corresponding to the reference test. It is pointed out that in this curve secant shearing moduli are calculated for each deformation (a) :

$$G_{pa} = \frac{p_a - p_0}{a}$$

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- A cyclic loading curve corresponding to N successive cycles between two pressure bounds p_a and p_o , where p_o is the total soil pressure at rest and p_a the pressure corresponding to the deformation $\Delta V/V_o = a$ of the first cycle (and hence the monotonic loading test).

For the N cycle, the deformation under maximum pressure p_a is noted $a_{M,N}$ and the deformation under the minimum pressure p_o is noted $a_{m,N}$.

For this type of cyclic loading, two moduli are consequently always defined :

$$G_{pN} = \frac{p_a - p_o}{a_{M,N} - a_{m,N}}$$

$$G_{sN} = \frac{p_a - p_o}{a_{M,N}}$$

G_{pN} is a secant modulus relative to only the cycle N .

G_{sN} is a secant modulus which takes into account the cumulative deformation a_M relative to the set N of cycles.

Two characteristic areas are also defined for these cyclic loadings (Figure 3) :

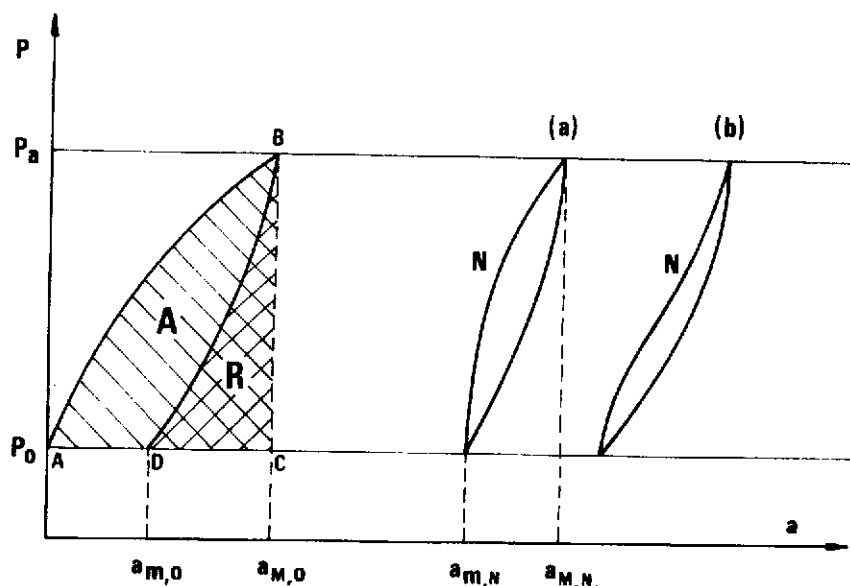


Figure 3 : Cyclic loadings between two pressure bounds : shape of loading loops and definition of characteristic areas.

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- The area $A = (ABC)$ which is proportional to the total energy developed by the pressuremeter during the expansion phase.
- The area $R = (BDC)$ which is proportional to the energy restored by the soil during the deflation of the pressiometric cell.

III - CYCLIC TESTS BETWEEN TWO PRESSURE BOUNDS (p_o , p_a).

We shall be giving the results of loading tests carried out on the Plancoët site in France, the main geotechnical characteristics of which are given in Table I.

Depth	Nature of soil	W	W_L	I_P	Cu or c' and ψ'
0 to 5 m	Silt	41	42	21	Cu = 20 kPa $c' = 0$ $\psi' = 37^\circ$
5 to 9 m	Loose sand	44	49	-	$c' = 0$ $\psi' = 43^\circ$
9 to 13 m	Silty clay	45	57	28	Cu = 65 kPa

Table I - Geotechnical characteristics of Plancoët site.

Three complete test profiles were carried out (boreholes 8-3, 16-1 and 7-3). These tests were all carried out at the same expansion and deflation rate (2 % per minute). All the tests were conducted between two pressure bounds :

- p_o , the total horizontal earth pressure at rest obtained after complete relaxation,
- p_a , the maximum test pressure (Figure 3) defined by the deformation a_{Mo} of the first cycle.

From one borehole to another, only this deformation a_{Mo} was made to vary, as indicated in Table II.

The last column of the table gives the average number of cycles N carried out on the considered profile. Certain tests were pursued up to 10,000 cycles, leading to test durations of several days.

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Borehole No.	a_{Mo}	Average N
8-3	0.5	5000
16-1	1.0	1500
7-3	5.0	1000

Table II - Main characteristics of cyclic tests carried out on the Plancoët site.

The main qualitative and quantitative results of these tests are given below :

Figure 3 represents the shape of the loading loops. After a few cycles, the loops appear to close back on themselves but nevertheless continue to move slowly toward the right as indicated by the plot in Figure 4 (real-time output on plotter of measurement setup). To avoid overloading the graph, only certain cycles are represented (the choice of the cycles represented is determined beforehand in the program). After a large number of cycles, the loops can have either the shape (a) or the shape (b) shown in Figure 3.

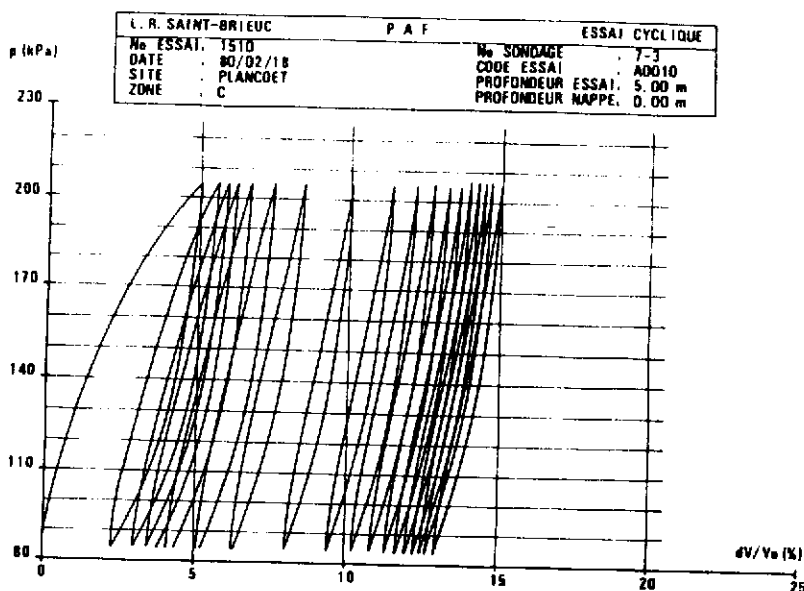


Figure 4 - Plancoët site. Borehole 7-3 (depth $z = 5.00$ m).
Real-time test plot.

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Whatever the number of cycles N and the maximum deformation a_M reached, it is noted that this deformation always tends to become stabilized, i.e. the corresponding curve is of the type (A) and not of the type (B) (Figure 5).

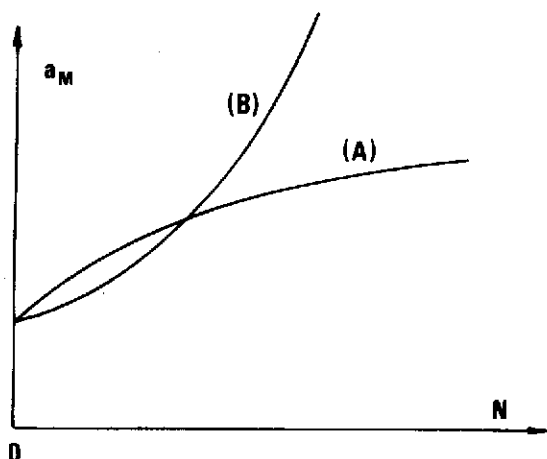


Figure 5 - Evolution of maximum deformation a_M as a function of the number of cycles N .

However, this stabilization is practically never reached even for very high cycle numbers, as indicated in Figure 6 in which is represented the evolution of the modulus G_s as a function of the logarithm of the number of cycles which, for the test concerned, was $N = 10^4$. In Table III is given, for the same test, the evolution of the moduli G_s and G_p as a function of N as well as the ratio A/R . In order to provide an idea of the evolution of stabilization, we have also represented the value $\Delta(G)$ defined as follows :

$$\Delta G = (G_N - G_0) / (G_{10\ 000} - G_0).$$

It is noted that the modulus G_p tends to become stabilized faster than the modulus G_s . This indicates that the loops reach constant shape fairly rapidly but continue to move in the direction of the increasing deformations.

The other important point of this test series concerns the influence of the deformation a_{M0} of the first cycle. To study more simply the influence of this parameter, the average behaviour was investigated by means of boreholes (since each of the tests on these boreholes is carried out for the same value of a_{M0}) (see Table II). The results are given in the graph of Figure 7 in which the maximum

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deformation a_M in % is plotted on the abscissa and the average pressure difference $p_a - p_o$ on the ordinate. Four curves are represented in this graph corresponding respectively to the first loading ($N = 0$), and then to the cycles $N = 50, 100$ and 1000 . For the last curve ($N = 1000$), the broken-line part indicates extrapolated values. This graph clearly indicates that if the initial deformation rate is low (for example of the order of 0.5 %) the effect of the number of cycles, i.e. the effect of time, is not significantly felt. On the other hand, if the initial deformation rate is higher (2 % for example or more) the effect of the number of cycles, and hence of time, becomes quite significant.

N	G_s (kPa)	$\Delta(G_s)$ %	G_p (kPa)	$\Delta(G_p)$ %	A/R
1	6 542	0	6 542	0	3.06
2	5 825	19	11 454	43	2.04
3	5 524	27	13 145	57	1.65
4	5 207	36	13 781	63	1.49
5	5 113	38	14 044	65	1.43
10	4 695	49	14 617	70	1.31
25	4 286	60	15 181	75	1.24
50	3 970	69	15 523	78	1.25
100	3 755	74	15 955	82	1.20
500	3 486	82	17 228	93	1.17
1 000	3 469	82	17 176	92	1.14
2 000	3 272	87	17 235	93	1.16
3 000	3 183	90	17 432	95	1.11
4 000	3 093	92	17 694	97	1.12
5 000	2 991	95	17 747	97	1.10
6 000	3 118	91	17 565	96	1.12
7 000	2 962	96	17 607	96	1.11
8 000	2 922	97	17 772	98	1.10
9 000	2 805	100	17 867	98	1.09
10 000	2 799	100	18 050	100	1.10

Table III - Plancoët site. Borehole 8-3. Depth $z = 8$ m.
Evolution of pressiometric characteristics as a function of number of cycles N .

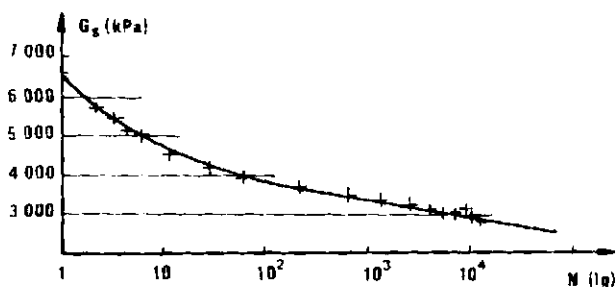


Figure 6 - Plancoët site. Borehole 8-3 (depth $z = 8.00$ m). Evolution of modulus G_s as a function of the logarithm of the number of cycles.

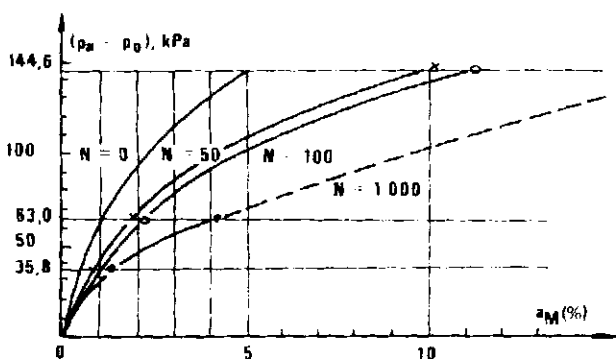


Figure 7 - Plancoët site. Average behaviour as a function of loading rate and number of cycles.

IV - OTHER TYPES OF TESTS

More complex cyclic loading programs were also carried out. A few examples are given below :

1. Loading between two pressure bounds p_1 and p_2 higher than the natural earth pressure at rest p_0 .

A loading program of this type is represented in Figure 8. This program includes three successive phases :

- 1st phase : 800 loading cycles between the pressure bounds $p_1 = 0.4 p_r$ and $p_2 = 0.6 p_r$, where p_r is the "failure" pressure measured during monotonic loading (pressure corresponding to the maximum deviator stress).

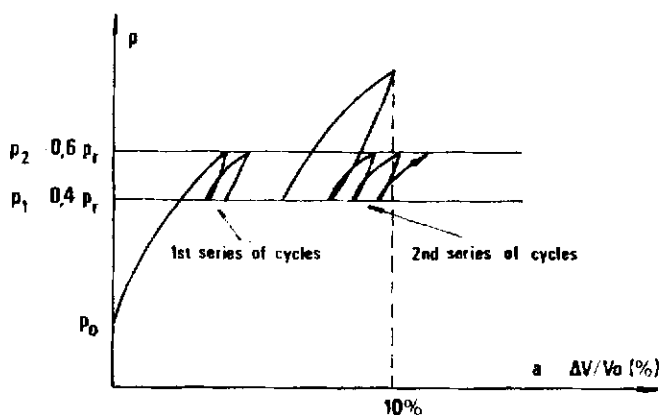


Figure 8 - Loading between two pressure bounds p_1 and p_2 higher than the natural pressure p_0 .

- 2nd phase : expansion of pressiometric cell up to the deformation of 10 %, i.e. approximately up to the pressure corresponding to half the limit pressure in this type of soil with immediate return to $0.4 p_r$.
- 3rd phase : application of a new series of cycles identical to those of the first series.

In Figure 9 is given the variation of the modulus G_s as a function of the cumulative number of cycles. During the first series of cycles, it is noted that the modulus G_s decreases normally and then exhibits a tendency toward stabilization. The "impact" of the expansion a 10 % causes significant permanent deformation which is not eliminated afterward. The pursuit of the second series of cycles leads to constant moduli G_s : there is no more loop translation ; the deformation is stabilized.

Figure 10 gives the corresponding evolution of the modulus G_p . It is observed that during the second series of cycles, this modulus is stabilized rapidly at the value close to that noted at the end of the first series of cycles (the number of cycles of the two series have not been cumulated in this graph).

2. "Storm" type loading

Figure 11 shows briefly a special test called the "storm test".

CYCLIC TESTS WITH THE SELF-BORING PRESSUREMETER

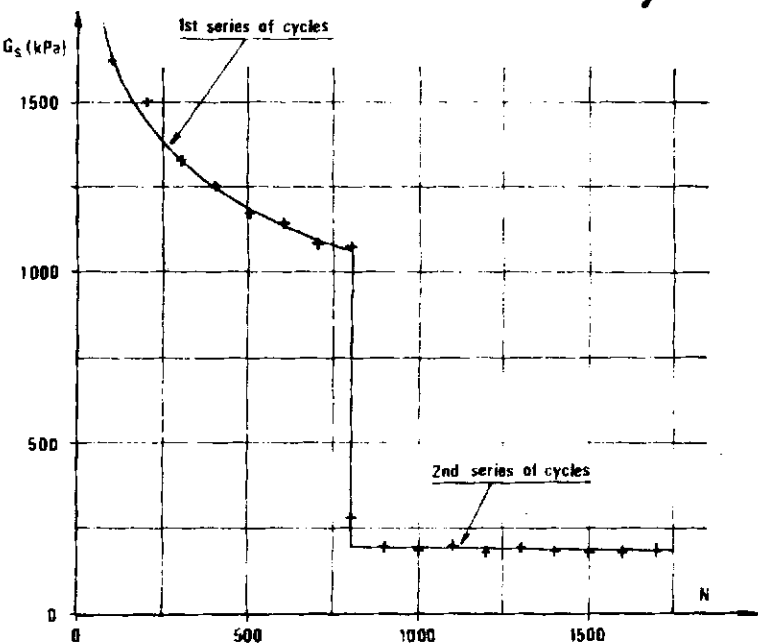


Figure 9 - Plancoët site. Borehole 13-3 (depth $z = 10$ mètres).
Cyclic loading between two pressure bounds : evolution of
modulus G_s as a function of number of cycles N .

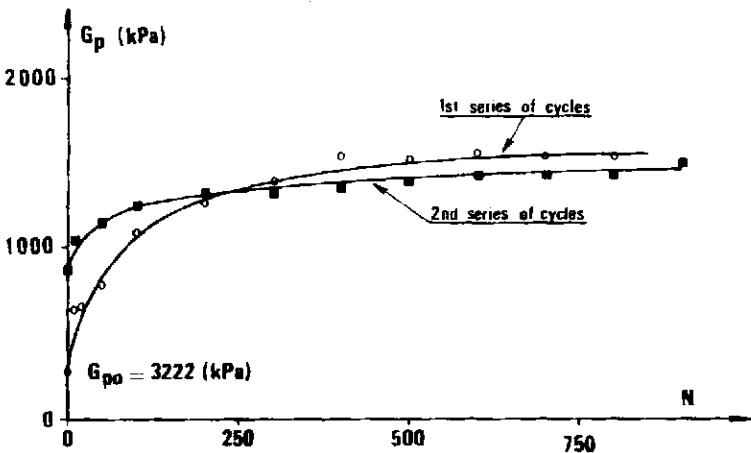


Figure 10 - Plancoët site. Borehole 13-3 (depth $z = 10$ metres).
Evolution of modulus G_p as a function of number of
cycles N .

Loading program

This program consists in applying a series of cycles between pressure bounds centered around an average pressure p_m equal to half the failure pressure p_f .

In Table IV below are given the characteristics of each series of cycles :

Series No.	Number of cycles	p max (kPa)	p min (kPa)	Duration of series
1	100	68.2	49.4	48'
2	100	77.7	39.9	78'
3	100	87.1	30.1	162'
4	100	77.7	39.9	84'
5	100	68.2	49.4	42'

Table IV - Plancoët site. Borehole 3-13 (depth = 1.00 m). Storm test loading program.

All the inflations and deflations of the pressiometric cell are carried out at the same rate of 2 % per minute.

In the second part of the figure has been represented the pressure/deformation envelope for the five series of cycles. As the loads are built up (series 1, 2, 3) one obviously observes a gradual increase in the amplitude of the deformation for the same number of 100 cycles : if the value of a is 1 for the first cycle, it is 1.15 for the second and 1.7 for the third. On the other hand, it is observed that for the last two series of cycles (4 and 5), there is practically no increase in the deformation : the modulus G_s remains practically constant while the modulus G_p increases very little.

CONCLUSIONS

We have presented a few typical cases of cyclic loading capable of being carried out by means of the Self-Boring Pressuremeter Model 76. This type of apparatus is perfectly suited to this type of investigation because it has the two features which are necessary in this case :

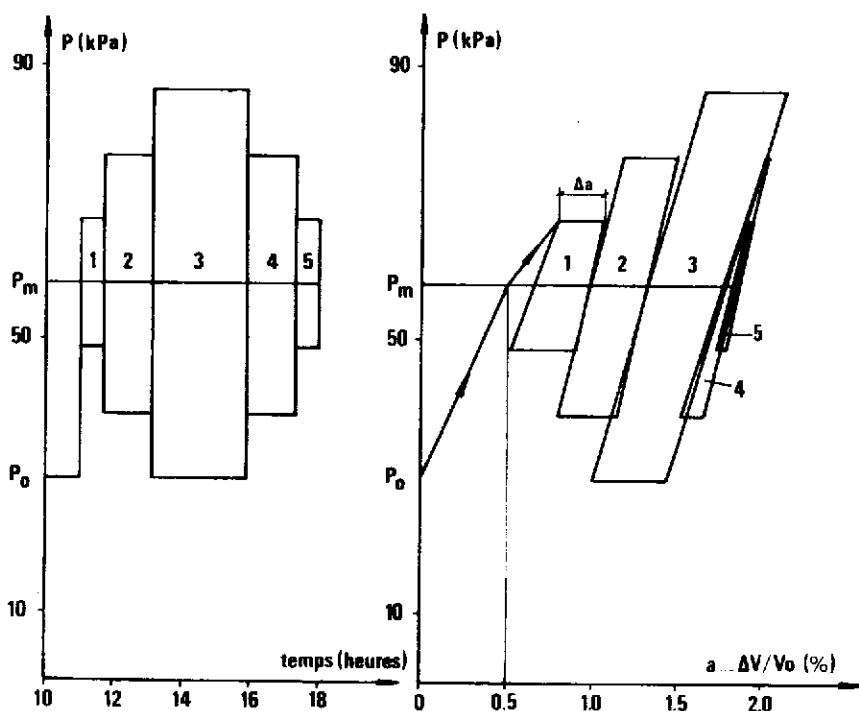


Figure 11 - Plancoët site. Borehole 3-13 (depth $z = 1.00 \text{ m}$). "Storm" type cyclic test.

It allows measurements on an undisturbed soil within the wide range of deformations applicable to foundations, including the very small deformations of a few per-thousand.

Owing to its expansion system (controlled deformation CPV) it allows very fine, very reliable and automated measurements. It also permits a wide variety of loading modes making it possible to simulate simply, for example, the action of waves or the action of shocks on the foundations.

The tests presented are research tests intended to provide general guidelines regarding soil behaviour. It is clear that during conventional reconnaissance campaigns in marine sites it will not be profitable to carry out such long tests. The purpose of the present research is thus also to bring out the empirical rules which will make it possible to extrapolate the results of the tests conducted on a limited number of cycles, notably according to the various types of soils which may be encountered, and for various types of loading simulating the forces acting on the structures.

SESSION 2

ADDITIONAL CONTRIBUTIONS AND DISCUSSION

DISCUSSION

J.L. BRIAUD- (*Texas A & M University*) asked Mr. JEZEQUEL

What is the influence of relaxation time after the probe has been driven in and before the pressuremeter test on the value of measured p_0 ?

J.F. JEZEQUEL- (*Laboratoire des Ponts et Chaussées de St-Brieuc*)

Insertion of the self-boring pressuremeter into the ground requires the respecting of various precautions such as the speed and steadiness of driving in, the geometric shape of the self-boring module, the nature and regularity of the membrane, the calibration of the pressuremeter cell, etc. If any one of these precautions is not respected, a disturbance of the soil occurs which cannot necessarily be obliterated by the relaxation operation.

Two cases can be schematically be distinguished :

(i) The insertion of the probe caused a decompression of the soil. During the relaxation phase, a continuous decrease in the pressure inside the probe will almost always be seen, and after a certain lenght of time this pressure will attain an asymptotic level which is the neutral pressure inside the formation. This phenemenon occurs most frequently in loose saturated sands.

(ii) Insertion of the probe caused pressure to be exerted on the formation (bulging^(*)). During the relaxation phase a pressure drop

(*) lateral displacement

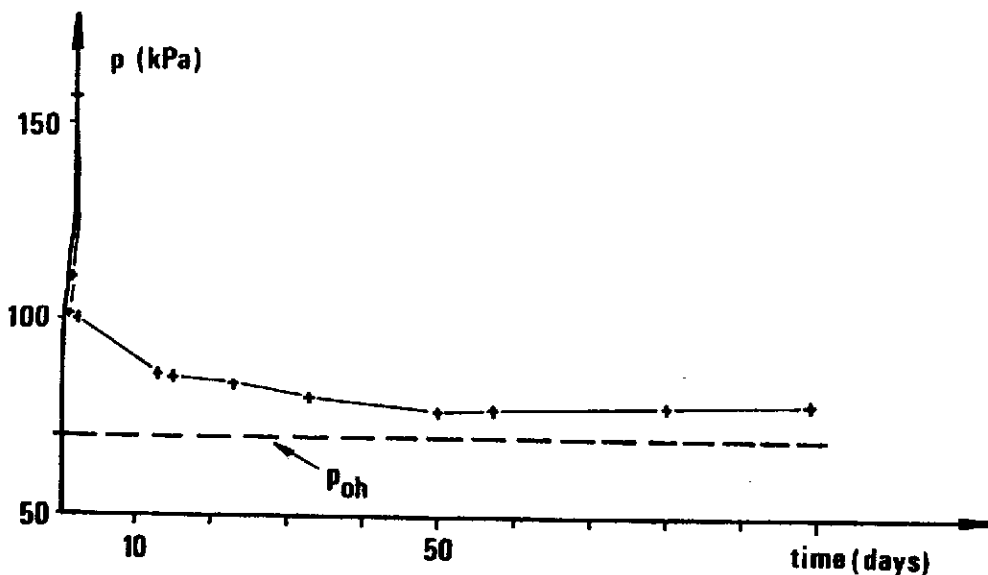
SESSION 2 ADDITIONAL CONTRIBUTIONS AND DISCUSSION

inside the probe will also be apparent. After a certain time this pressure will reach a higher extreme level than the neutral pressure, and by general agreement this level is called the total horizontal pressure of the formation at rest (p_o).

The problem is to know whether — no matter what the bulging amplitude may be — this pressure always tends to the same value of p_{oh} .

Systematic tests have been performed on this problem on the Cran site where the pressuremeter was inserted at different bulging amplitudes.

The figure shows the result of a very long relaxation time (3 months) for implementation with maximum bulging. The cutting edge was obstructed by a typical penetrometer tip. After 50 days the pressure could be seen to have reached a quasi asymptotic level (allowing for measurement errors). At the same time however, this pressure was higher than the measured p_{oh} pressure during the normal implementation of the pressuremeter (these results were published in the Proceedings of the Soil Mechanics Congress, Moscow, 1973, Volume 4-3, p 243-247).



Long-time relaxation curve after implementation by total bulging (Cran site, $z = 5.50$ m).

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To make things clearer, even though it is p_{oh} that is been sought after and not K_o , the latter value has been calculated for both measurements :

- . after normal implementation, the measured p_{oh} value corresponds to a K_o of 0.50 ;

- . after implementation by complete bulging, the pressure measured after three months of relaxation should correspond to a K_o of 0.81 !

This shows that a long relaxation time does not necessarily suffice to correct defective implementation. Above all therefore, the implementation precautions must be respected, and then objective criteria must be used to check and see whether the operation has been carried out correctly.

One of the criteria included in the operational procedure of the PAF is that the pressure drop noted during the relaxation phase is less than the pressure increase noted during the expansion test for the deformation of the pressuremeter cell from 0 to 0.50%.

To sum up, it can be affirmed that relaxation times of between at least one hour and at most several hours are sufficient to obtain realistic values of p_{oh} if the implementation precautions have been respected.

M. GAMBIN- (*Etudes Pressiométriques Ménard*)

The influence of some parameters of the self-boring equipment and testing procedure on the results as they are obtained now, still probably need some more research work. In my opinion the proper size of the cutting shoe and the height of the drilling tool inside the shaft of the probe are of major importance. Eden and Law working at the National Research Council of Canada have already expressed their views in this respect (*Canadian Geotechnical Journal*, Vol. 17, numbers 2 and 3, 1980 ; discussion in Vol 18, number 2). Also the influence of the diameter proper should be investigated.

M. JAMIOLKOWSKI- (*Technical University of Turin*) and M. GAMBIN- (*Etudes Pressiométriques Ménard*).

Discussion concerning the value of the Poisson ratio used to calculate the pressuremeter modules :

- M. Jamiolkowski referred to undrained conditions with $\nu = 0.5$,

- M. Gambin pointed out that the standard pressuremeter modulus is calculated with $\nu = 0.3$ and, hence, that it cannot be compared with the modulus obtained by assuming $\nu = 0.5$.

SESSION 2 ADDITIONAL CONTRIBUTIONS AND DISCUSSION

CONTRIBUTION

CYCLIC PREBORING PRESSUREMETER TESTS

J.L. BRIAUD- (*Texas A & M University*)

A 100 cycle preboring pressuremeter test was performed and the results are presented in Figures 1 and 2.

- Figure 1 shows the raw data, the membrane calibration and the volume losses calibration for the 100 cycles,

- Figure 2 shows the corrected data and the definition of the secant cyclic modulus E_s and the cumulative strain ϵ_p ,

- Figure 3 shows the plot of E_s and ϵ_p versus number of cycles.

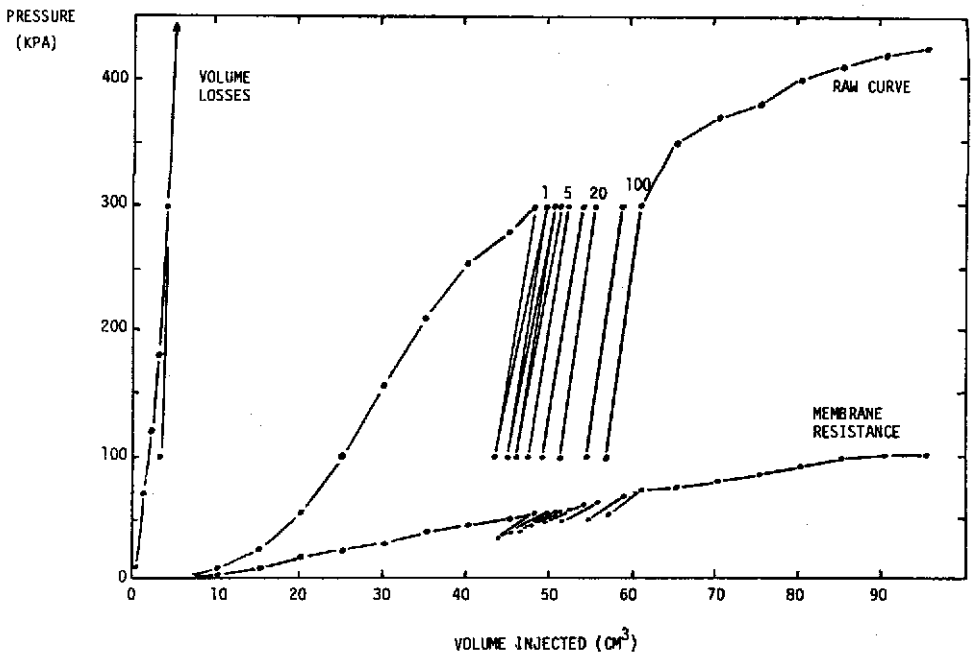


Figure 1

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PRESSURE (KPA)

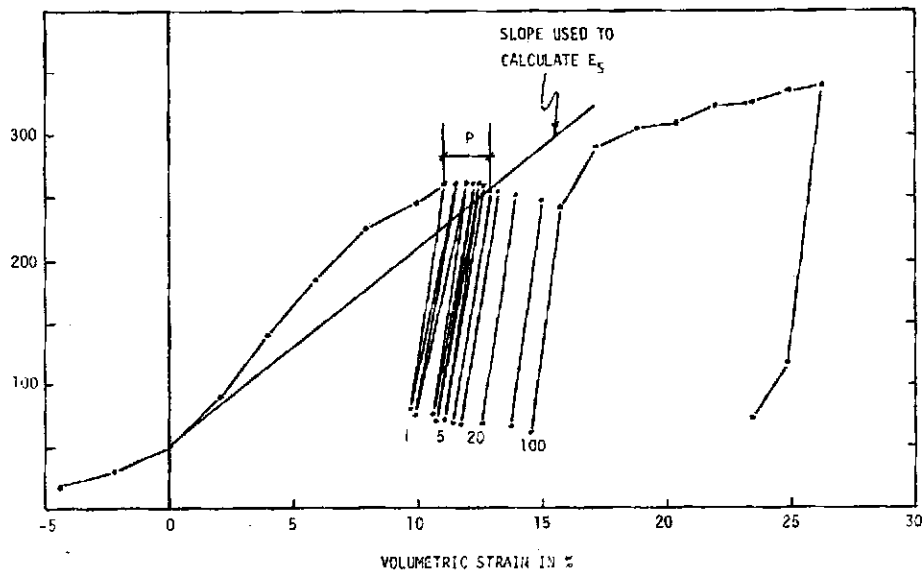


Figure 2

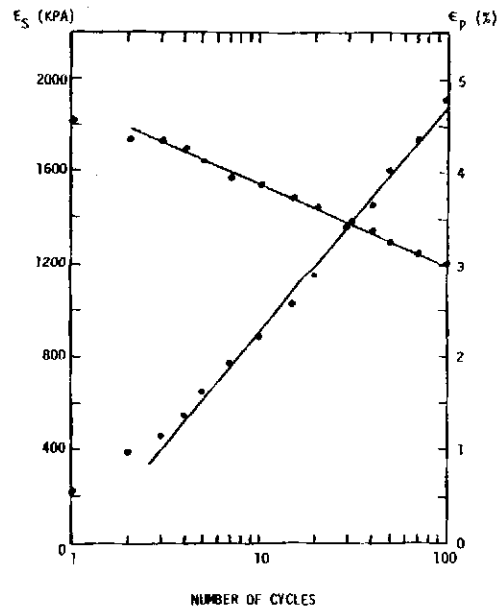


Figure 3

SESSION 2 ADDITIONAL CONTRIBUTIONS AND DISCUSSION

DISCUSSION

J.L. BRIAUD- (*Texas A & M University*) asked Mr. JEZEQUEL

How can temperature-correction and membrane-effect problems be solved during cyclic pressuremeter tests ?

J.F. JEZEQUEL- (*Laboratoire des Ponts et Chaussées de St-Brieuc*)

All long-range pressuremeter tests (relaxation, cyclic or monotonic loading, etc.) may be influenced by the following two types of problems :

(i) Influence of temperature variations on surface equipment, on the water or gas injected into the cell, and on the flexible pipes linking the surface equipment to the pressuremeter cell.

(ii) The properties of sheaths and membranes. It is known that their strength depends both on the temperature and on the type and duration of the sollicitations involved.

Therefore, an effort must be made to minimize the influence of these disturbing parameters which could entirely mask the measurement results if proper care is not taken. In particular, the surface equipment must be thermally stabilized, and of course the resistance of the probe must be measured using the same loading program as the one that will be applied inside the soil.

In the case of simple procedures such as relaxation or monotonic loading, these problems can be properly mastered.

In the case of cyclic loading the situation is much more difficult, especially with regard to the resistance of the probe. This resistance depends on the "stress/pressure/deformation/time" path which obviously cannot be known at the moment of calibration.

For long-term cyclic loading (several days of testing), it is merely possible to estimate a supremum for the errors caused by different disturbing phenomena. When all precautions are taken, it is judged that a shear modulus can be determined in soft to moderately consistant soils within $\pm 10\%$, which is an excellent approximation nonetheless.

J.L. BRIAUD- (*Texas A & M University*)

The number of cycles possible during a pressuremeter test or a pile test is much smaller than the number of cycles experienced by a

platform during its service life. In 50 years, the number of cycles is around 10^8 .

P. LE TIRANT- (*Institut Français du Pétrole*)

It is true that during its life an offshore platform undergoes a number of cyclic stresses around $5 \cdot 10^7$ to 10^8 . However, the only really important ones for the behavior of the structure are the major storm stresses.

Cyclic tests on experimental foundations (piles, gravity structures, etc.) cannot exceed a few thousand cycles in practice.

Although cyclic tests with the self-boring pressuremeter generally conducted to a few hundred cycles have not led to the perfect stabilization of the mechanisms investigated, they have nevertheless offered reliable and representative measurements of cyclic shear moduli of the soil for use in calculations to forecast the behavior of piles, for example. Hence we hope that practical experience in cyclic measurements using the self-boring pressuremeter will, based on a relatively limited number of cycles (about 100 ?), enable predictions of the soil parameters in the long term.

A. VAN WAMBEKE- (*Ecole Royale Militaire, Brussels*) asked Mr. JEZEQUEL

Accidental pulses (which are dynamic processes) exert a considerable effect on soil "aging". A simple hammer blow on the dynamic oedometer considerably accelerates the consolidation process. Has the application of cyclic tests for dynamic consolidation been considered ?

J.F. JEZEQUEL- (*Laboratoire des Ponts et Chaussées de St-Brieuc*)

This problem has not been examined.

CONTRIBUTION

COMPARISON OF DYNAMIC MODULI DETERMINED BY SELF-BORING PRESSUREMETER AND CONVENTIONAL DYNAMIC METHODS.

G.W. JOHNSON- (*Mc Clelland Eng. Houston*)

K.H. STOKOE, II- (*The University of Texas, Austin*)

L.C. REESE- (*The University of Texas, Austin*).

INTRODUCTION

A field site was selected and three methods were used to obtain values of shear modulus of a clay at small levels of strain. The crosshole seismic method was used at the site, and the resonant column device was used in the laboratory to obtain *values of shear, moduli at shearing* strains less than 0.001 percent. In addition, the self-boring pressuremeter was used in the field, and shear moduli were obtained by finding the initial slope of the pressure-deflection curves. The use of each of these methods is discussed in the following sections, and the close comparison between moduli measured by the crosshole method and the self-boring pressuremeter is shown.

CROSSHOLE SEISMIC TESTING

In situ seismic testing was performed to determine the in situ low-amplitude shear modulus. The particular in situ test that was employed was the crosshole test. Two boreholes were drilled about 3 m apart, seismic waves were generated in one of the boreholes, and wave velocities were determined by measuring wave arrivals with a transducer in the second borehole. Waves were generated at various depths to determine the profile of shear wave velocity. The crosshole test is considered by many investigators to be the most reliable seismic method for determining shear and compression wave velocities from which shear and constrained moduli are calculated. Moduli at very small strains, less than 0.001 percent, are determined by use of crosshole testing. For the study presented herein, the shear waves were of most concern.

The seismic equipment and testing procedure used in crosshole testing is discussed in detail by Hoar and Stokoe (1978). Only a brief presentation is given here. The equipment required to perform crosshole seismic tests include : 1) a source ; 2) borehole receivers, 3) a trigger mechanism, and 4) a recorder.

An in-hole mechanical source was used for generation of the shear and compression waves. The source consisted of two wedges. The lower wedge was held at the desired elevation while the upper wedge was forced against it by dropping a 14 kg hammer about 20 cm. This source is normally used in cased boreholes and generates vertically polarized shear while simultaneously generating compression waves.

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In these crosshole tests, the velocity of vertically polarized shear (SV) waves and compression (P) waves was measured. A vertically oriented geophone was used to monitor the shear wave arrival while a horizontally oriented geophone, aligned parallel to the line between the source and receiver, was used to monitor the arrival of compression waves.

The time between the moment the impulse is applied to a source and the moment the recorder is initiated is controlled by the trigger mechanism. This trigger time must be accurately measured so that the time for the wave to propagate from the source to the receiver can be correctly measured. In this testing, a vertical geophone was used at the base of the in-hole source so that an accurate zero time could be determined.

A digital oscilloscope with magnetic recording capabilities was used to measure wave travel times and store data on magnetic disks for later recall and analysis.

Travel times of the shear waves were determined from the magnetic record which was replayed on the oscilloscope in the laboratory. By knowing the distance between boreholes and the travel times of the shear waves at each depth, shear wave velocities were calculated. Shear wave velocities were then used to determine low-amplitude, in situ shear moduli of the soil from :

$$G_{\max} = \frac{\gamma_T}{g} v_s^2 \quad (1)$$

where $G_{\max}$ = low-amplitude shear modulus.

γ_T = total unit weight of the soil,

v_s = shear wave velocity, and

g = acceleration due to gravity.

RESONANT COLUMN TESTING

With resonant column devices, a soil specimen is excited in either torsional or axial vibration while the specimen is generally confined under a hydrostatic pressure. During excitation, measurements are made of the first-mode resonant frequency and amplitude of vibration from which wave velocity and strain amplitude are computed. Wave velocity determined from a torsionally or axially vibrated specimen is then used to compute shear modulus or Young's modulus, respectively.

SESSION 2 ADDITIONAL CONTRIBUTIONS AND DISCUSSION

The resonant column device used in this study was a fixed-free type ; that is, the sample was fixed at the bottom and free to vibrate at the top. The device consisted of the following four components : 1) a confining pressure system ; 2) a torsional drive system ; 3) a height-change measurement system ; and 4) a resonant-frequency measurement system. A solid, cylindrical soil specimen was tested. The specimen had a height-to-diameter ratio of two to one, with a diameter of 3.8 cm.

The soil specimen was contained inside a stainless steel chamber with an outside diameter of 21.6 cm and a height of 39.4 cm. Compressed air was used to apply the desired hydrostatic confining pressure to the system. To lessen air migration into the soil, the specimen was surrounded on the sides by a silicon oil bath.

Cyclic shearing stresses were applied to the top of the fixed-free specimen by means of a torsional drive system. The drive system consisted of a drive plate, drive coils, sine wave generator, and power amplifier. The drive plate was composed of four magnets mounted on four perpendicular arms. Eight drive coils, connected in series, encircled and ends of the four magnets and acted together as an electrical motor to apply a torque to the top of the specimen when a current was passed through the coils.

Measurement of the change in height of the specimen with time is important in the calculation of changes in void ratio and mass density at the various confining pressures. The change in height of the soil specimen was measured with a linear variable differential transformer (LVDT).

The resonant-frequency measurement system consisted of a piezoelectric accelerometer mounted on the drive plate to pick up torsional vibrations, a charge conditioner to linearize the accelerometer output, an analog voltmeter, and a period measuring device.

The shearing strain at resonance was calculated using the measurement of the resonant period and the accelerometer output. The resonant frequency was determined from the resonant period. For this solid specimen of clay, the shearing strain was determined at two-thirds the specimen radius.

Data reduction is based on the wave equation in torsion and requires measurement of the resonant frequency and length of the soil specimen. It is also necessary to know accurately the mass moments of inertia of the specimen and the drive system. The analytical expression used in this resonant column analysis is (Richart, Hall, and Woods, 1970) :

$$\frac{I}{I_0} = \frac{\omega l}{V_s} \tan \frac{\omega l}{V_s} \quad (2)$$

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where I = mass moment of inertia of the soil specimen,
 I_0 = calibrated mass moment of inertia of the drive system,
 ω = first-mode natural circular frequency,
 l = height of the soil specimen, and
 V_s = shear wave velocity of the soil.

TESTING WITH SELF-BORING PRESSUREMETER

The self-boring pressuremeter used had a diameter of 10.8 cm. The exposed membrane had a length of 30.5 cm, and its middle was located 50.8 cm from the tip. The pressure was applied with nitrogen gas, and the pressure was measured by use of a commercial pressure transducer. The deflection at the middle of the membrane was measured by three LVDT's, located 120 degrees apart.

It was necessary to fit a curve to the early portion of the pressure-deflection curve (shearing strains less than 0.30 percent) in order to make an accurate determination of the modulus at low strains. The following expression developed by Ramberg and Osgood (1943) was found to be useful.

$$\epsilon = \frac{\sigma}{E} + K \left(\frac{\sigma}{E}\right)^n \quad (3)$$

where ϵ = strain,
 σ = stress,
 E = Young's modulus, and
 K and n = constants.

The first term of Eq. 3 describes the line drawn from the origin of the stress-strain plot with a slope of E . The second term describes the difference between the linear plot of slope E and the actual measured strain, ϵ , for stress, σ .

The expansion curve of a pressuremeter test has the same basic shape as the Ramberg-Osgood curve if the origin is shifted to account for the fact that the applied probe pressure must first overcome the in situ horizontal pressure of the soil. Equation 3 may be rewritten as :

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$$\epsilon_r = \frac{\sigma_a - \sigma_h}{2G_i} + K \left\{ \frac{\sigma_a - \sigma_h}{2G_i} \right\}^n \quad (4)$$

where ϵ_r = radial strain,

σ_a = applied probe pressure,

σ_h = total horizontal in situ stress,

G_i = initial shear modulus, and

K and n = constants.

Using Eq. 4 and the mechanics of an expanding cavity, the shear modulus was obtained.

To determine the value of the maximum shear modulus, $G_{\max}$, a computer program was employed and a least-squares fit was performed.

TEST SITE

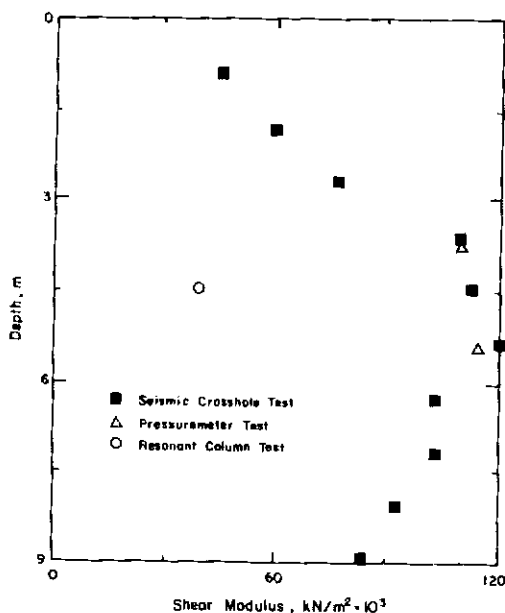
The site is located about 9.7 km east of the campus of The University of Texas at Austin. The soil is an overconsolidated clay with an undrained shear strength ranging from 60 kN/m² at a depth of 5.2 m to 71 kN/m² at a depth of 6.8 m. The average water content of the soil was about 21 percent.

RESULTS OF TESTS

The values of initial shear modulus, $G_{\max}$, obtained by all three methods are shown in figure for the clay deposit at this site. The values of initial shear modulus determined from the pressuremeter tests are within 5 to 10 percent of the values determined by seismic crosshole tests.

Shear modulus determined by the resonant column test is significantly lower than the in situ $G_{\max}$. The initial laboratory shear modulus, $G_{\max \text{ lab}}$, is only 35 percent of the initial shear modulus value, $G_{\max \text{ field}}$, determined by seismic tests. Stokoe and Chen (1980) noted that, " $G_{\max}$ determined in the field consistently ranges from 20 percent to more than 100 percent above laboratory $G_{\max}$ values". The differences between field and laboratory moduli are believed to be due to unavoidable disturbance and the effects of stress relief during sampling and the inability to simulate in situ conditions during laboratory tests.

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CONCLUDING COMMENTS

The self-boring pressuremeter can be used to measure the initial, in situ shear modulus of a clay soil with an accuracy that is quite good, as shown by the close agreement between moduli from pressuremeter tests and from crosshole seismic tests. Crosshole tests were used for comparison purposes because this test is thought to give the best estimate of in situ shear moduli under small strains. It is important to note that when the expansion curve from the pressuremeter test was fit using the Ramberg-Osgood equation, only data from the initial part of the curve was used to determine the initial modulus. If the complete expansion curve was used, a much lower initial modulus was obtained.

As expected, the modulus determined by the resonant column test in the laboratory was much lower than field moduli determined by either the seismic tests or the self-boring pressuremeter.

The results from this study show that the self-boring pressuremeter has application in obtaining soil properties at small strains for use in dynamic analyses.

SESSION 2 ADDITIONAL CONTRIBUTIONS AND DISCUSSION

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RICHART, F. E., Jr., HALL, J. R., Jr AND WOODS R. D. (1970), Vibrations of Soils and Foundations, Prentice Hall, 414 pp.

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DISCUSSION

COUPRIE- (*Solétanche*)

I should like to ask Professor Wroth two questions concerning the results obtained on London clay using the Camkometer :

(1) What is the accuracy in the determination of p_0 ?

(2) Can the undrained cohesion C_u measured in this way be compared with the values inferred from other tests which attempted to determine the effect of the dimensions of the measurement instrument ?

C.P. WROTH- (*University of Oxford*)

(1) The accuracy of p_0 is difficult evaluate.

(2) It is difficult to make a comparison between the results obtained for C_u using the Camkometer and using other methods.

M. JAMIOLKOWSKI- (*Technical University of Turin*)

I should like to ask Mrs. Brucy, Mr. Fay and Mr. Marignier two questions.

(1) Does a slenderness ratio (length/diameter) of 2 for the pressuremeter cell not lead to an overestimate of C_u and of the limite pressure due to the shape of the expanded cavity ?

SESSION 2 ADDITIONAL CONTRIBUTIONS AND DISCUSSION

(2) It would seem that the diameter of the cutting edge should be smaller than that of the membrane rather than larger. In the latter case, the results are overestimated and hence unreliable.

F. BRUCY- (*Institut Français du Pétrole*)

(1) The slenderness ratio selected for the pressuremeter cell is that of the PAF76, which has been subjected to many comparative investigations.

(2) The 1% oversize of the cutting edge effectively yields results different from those given by the PAF76. The shear strength in particular is increased by about 20% while the moduli are slightly decreased.

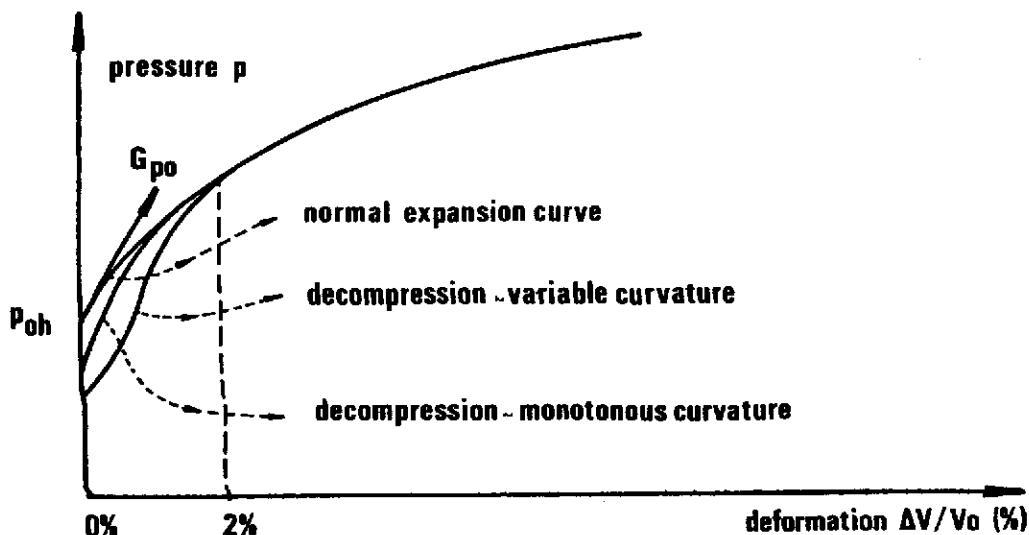
J.F. JEZEQUEL- (*Laboratoire des Ponts et Chaussées de St Brieuc*)
in reply to Mr. JAMIOLKOWSKI

1) Theoretical and experimental research has been performed to analyze the influence of the slenderness ratio of the pressuremeter cell on the properties that are measured. It has been found that these properties can effectively be influenced by the slenderness ratio of the measuring cell. In coherent formations a slenderness ratio of about six is required so as to more or less respect the plane deformation condition. In sandy formations this slenderness ratio must be much higher in order to reach this plane deformation condition. In any case the pressuremeter test is a conventional one, and this should not be forgotten during any interpretation.

2) In all cases it is preferable to position the cutting edge exactly in the prolongation of the measuring cell rather than in front of or behind it.

To reduce lateral friction on the probe during driving in, and hence to increase the penetration capacity, cutting edges are sometimes used which project slightly beyond the pressuremeter cell. It is then found that the values of P_{oh} and of the initial tangent modulus G_{po} are influenced. However, there is no impact on moduli G_{p2} and G_{p5} nor, a fortiori, on pressure P_{20} . Depending on the extent of overthrust by the cutting edge, the expansion curve may - despite decompression - show monotonic concavity or, as in Menard's test, reveal a change in curvature after an initial "recompaction" phase.

In both cases the modulus G_{po} is underestimated. With monotonic curvature, measured pressure p_{oh} is lower than the pressure of the formations at rest. With an expansion with a recompaction phase, end-of-compaction pressure p_o is higher than the pressure of the formations at rest (see figure).



Influence of formation decompression on the shape of the expansion curve.

Mr OHYA- (*OYO Corporation, Japan*)

Addressing Miss Lacasse concerning horizontal stress measurements in soils :

- . by hydraulic fracturing,
- . using measurement cells for total stress,
- . using the LLT ('pre-boring') pressuremeter,

Mr. Ohya believes that the first method is poor, that the second requires very long stabilization time, and that the results obtained with the LLT lateral load test are far better.

S. LACASSE- (*Norwegian Geotechnical Institute*)

I also think that the results obtained with the pressuremeter are more reliable.

COMMENTS by

F. BAGUELIN- (*Laboratoire Central des Ponts et Chaussées*) about :

THE METHODS FOR DETERMINING p_{oh}

During this session, many methods have been presented for determining the horizontal soil pressure at rest (p_{oh}) using the self-boring pressuremeter, especially by S. Lacasse and T. Lunne, and their merits have been discussed, implicitly taking the determination of p_{oh} on a sample as a reference, by the K_0 triaxial test.

This is a perfectly arbitrary hypothesis, and the problem concerning the measurement of horizontal soil pressures at rest is that, in actual fact, we do not have an absolute reference.

In these conditions, to go beyond the raw measurement of p_{oh} , by trying for example to determine p_{oh} from measurements taken during the pressuremeter expansion phase, appears to be ineffectual. These attempts amount to postulating a completely arbitrary hypothesis concerning the shape of the shear curve (linear or bilinear in arithmetic, logarithmic, hyperbolic scale, etc.).

If we stick to the raw data from the different tests, it may be inferred that the values obtained by the self-boring pressuremeter are, probably with Marchetti's dilatometer (DMT), the closest to reality, because these techniques cause the least disturbances in the soil, in comparison with other in situ tests and with sampling and sample preparation procedures. Furthermore, it may be noted that the self-boring pressuremeter provides perfectly significant variations in coefficients K_0 , as in the case of normally consolidated soil with an overconsolidated crust at the surface.

What are the consequences of errors in the determination of p_{oh} ?

In the procedure which we adopted for PAF76 reference tests, the relaxation time between the interruption of driving and the start of pressuremeter expansion is limited to 10 minutes. As a rule, this leads to an overestimate of the value of p_{oh} , and implies greater safety for determining the other parameters (shear strength, net limit pressures, etc.).

Anyway, it is important to realize the relative value of the parameters used in Soil Mechanics, and the need to correlate their use with the behavior of real structures. In this respect, parameter p_{oh} is just as valid as the cohesion determined with the vane probe, a modulus measured with the triaxial apparatus, or strength parameters c and ϕ measured using the shear box. Their only practical value is related to formulas and theories, to the extent that the latter yield realistic, duly calibrated forecasts.

CONTRIBUTION

CHARACTERISTICS OF SATURATED CLAY AS OBTAINED FROM SBP TESTS
FURTHER EXPERIMENTAL RESULTS

V. GHIONNA, M. JAMIOLKOWSKI AND R. LANCELLOTTA- (*Technical University of Turin - Politecnico - Italy*)

Just after the presentation of the writer's paper to this Symposium, the Technical University of Turin, in cooperation with Aquater S.p.A. Geotechnical Consultants, have performed a series of Self-Boring Pressuremeter tests offshore from La Spezia in water depths of ≈ 7 m using the modified Camkometer probe which allows the independent measurement of radial displacements for each of the three strain feelers located at midheight of the probe.

A series of 17 quick expansion tests have been performed adopting the installation and testing procedures summarized in Table I.

The tested soil is a highly plastic NC clay the index and stress-strain-strength properties of which may be deduced from figure 1 and Table II.

The interpretation of these tests with respect to undrained strength and stiffness put into evidence once more that the SBP tests lead to much higher values than those resulting from field vane tests and laboratory tests run on high-quality undisturbed specimens, see Ghionna et al. (1981, 1982).

The most interesting feature of these tests consists in the fact that at the end of the relaxation period which follows the self-boring process in each test, three independent values of the initial total pressure, p_0 , have been measured.

These values of p_0 are compared in figure 2 with the best estimate of the existing in situ total horizontal stress, σ_{ho} , as deduced from triaxial and instrumented oedometer tests.

The values of p_0 in figure 2 have been determined directly from the initial portion of the expansion curve a typical example of which is shown in figure 3. It can be seen that the improved probe allows a very straight-forward and easy detection of the so-called "lift-off pressure" (see for example Denby (1977) and Lacasse and Lunne (1982)).

The analysis of the experimental data shown in Figure 2 allows the following comments :

- (1) The tests discussed here confirms the previously gained

CHARACTERISTICS OF SATURATED CLAYS FROM SBP TESTS

TABLE I: SELF-BORING AND TEST PROCEDURES

$-0.9940 < \frac{\text{Shoe diameter}}{\text{Membrane section diameter}} < 0.9976 (*)$ - Retraction of chopper inside cutting shoe ≈ 0.7 cm - Rate of advance = 2 to 2.5 cm/min - Hydraulic pressure 40 to 55 t/m ² - Drilling water flow rate = 20 lt/min - Surface motor rotation = 60 rpm (*) see Law and Eden, 1982	- Relaxation time ranging from 80' to 2840' - All tests run at a constant rate of strain - Adopted strain rate $\approx 1\%$ / min - Volumetric strain reached during the tests $\approx 12\%$ to 17%

TABLE II: SOIL CHARACTERISTICS

INDEX PROPERTIES

γ (t/m ³)	W_n (%)	LL (%)	PI (%)	CF (%)
1.60 to 1.65	45 to 60	70 to 80	45 to 65	40 to 60

STRESS-STRAIN AND STRENGTH CHARACTERISTICS

UNDRAINED BEHAVIOUR						DRAINED BEHAVIOUR	
Type of test	$\frac{c_u}{\sigma_{1c}}$	γ_f (%)	A_f	$\frac{E_{u50}}{\sigma_{1c}}$		$\bar{\phi}$ (°)	26 to 29
FVT	0.26	NA	NA	NA		$\bar{c}$ (t/m ²)	0
TX	CL	0.32	6.8	0.95 to 1.05	47	M (t/m ²)	$18 \cdot \bar{\sigma}_{Vo}$
CK _O U	EL	0.21	20.0	0.85 to 0.90	35	C_v^L (m ² /y)	1.6 to 9.5
DSS - CK _O U		0.27	12.8	NA	109	C_h^F (m ² /y)	7.5 to 16.0
PSD CK _O U	CL	0.33	-	-	-	CR	0.15 to 0.25

TX Triaxial
 CL Compression Loading
 EL Extension Loading

DSS Direct Simple Shear
 PSD Plane Strain Device
 CF Clay Fraction
 NA Not Available

ST Sensitivity from FVT
 FVT Geonor Field Vane Test
 L Laboratory c_v value
 F Field C_h Value-Piezometer Probe

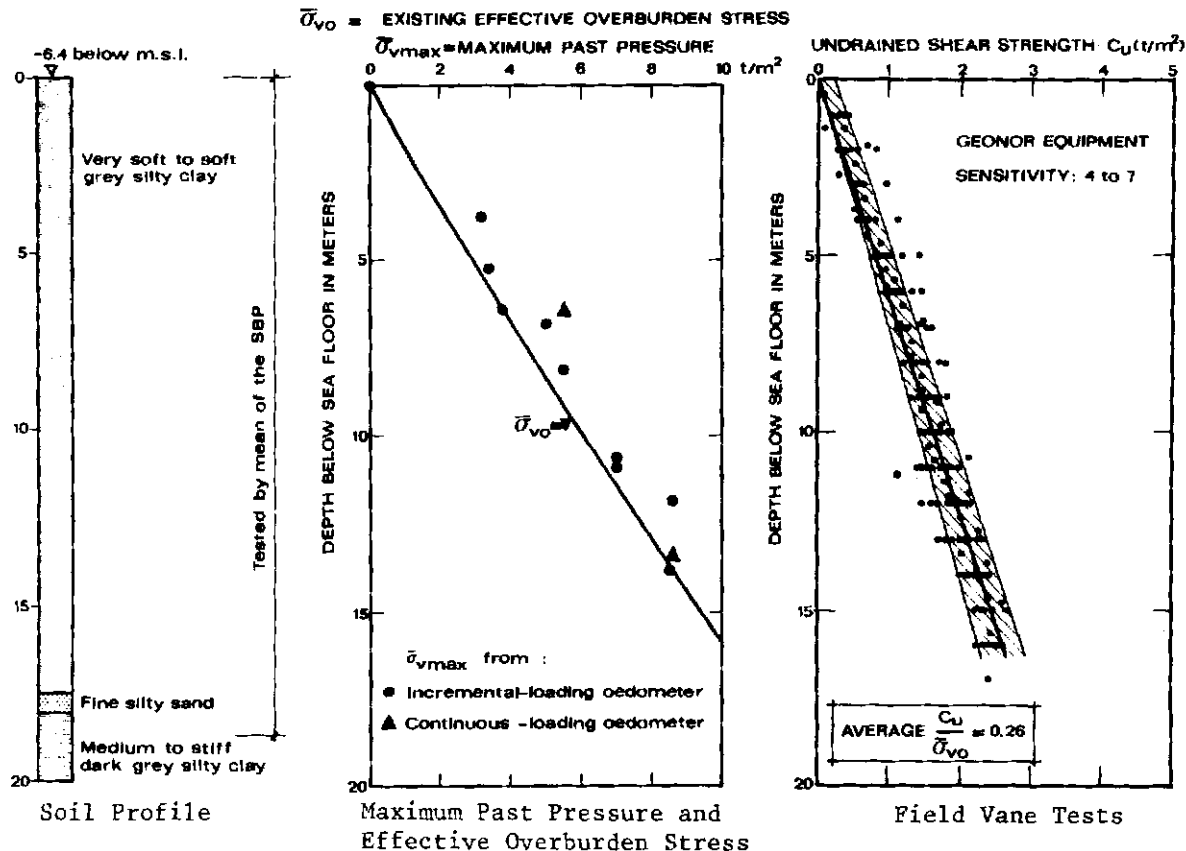


Fig.1. Soil characteristics

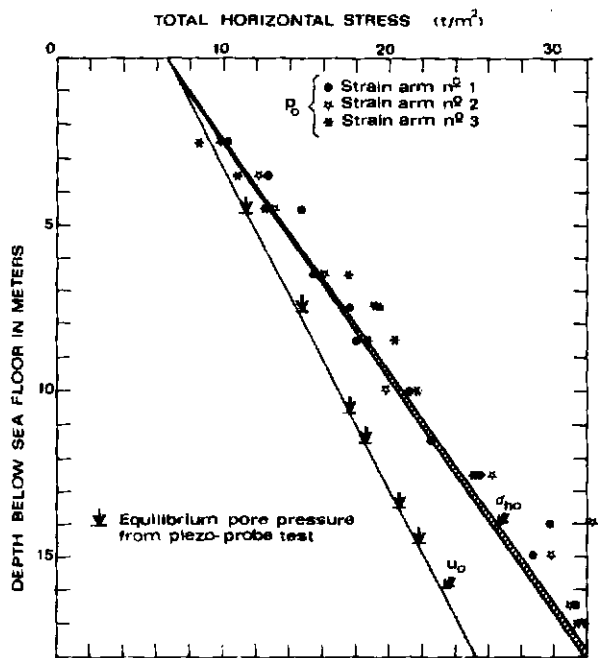


Fig.2. In situ total horizontal stress best estimate vs. values measured with the SBP

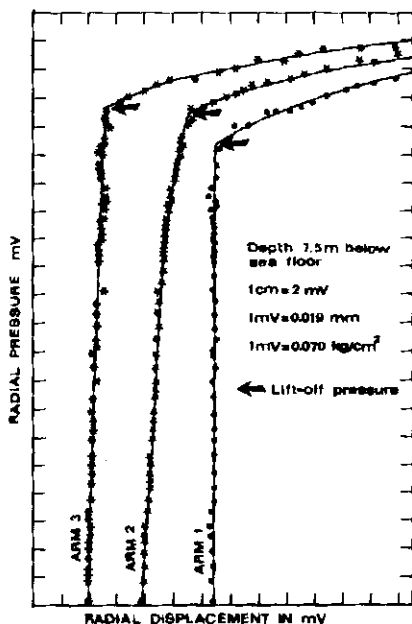


Fig.3. Example of measured lift-off pressure

experience that the SBP allows the determination of σ_{ho} values which are in good agreement with those estimated from other sources.

(2) Each one of three strain feelers yields slightly different values of p_o at the same depth.

It is not possible to decide at the present stage of knowledge if the latter phenomenon, already observed by Dalton and Hawkins (1982), must be attributed to the slight disturbance of the soil adjacent to the probe caused by the self-boring process or to a inherent non-uniformity of the total in situ horizontal pressure of the tested soil deposit.

It may also be pointed out that similar results, as far as p_o measurements are concerned, have recently been obtained at the Porto Tolle site in a very recent NC young silty clay which is well documented from a geotechnical point of view [see Jamiolkowski et al. (1980) and Ghionna et al. (1981)]. It must be added that at this site the three strain feelers yielded three different values of p_o during the first expansion (performed up to volumetric strains of $\approx 15\%$), but led to substantially the same three values of p_o during a second expansion after unloading.

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SESSION 2 ADDITIONAL CONTRIBUTIONS AND DISCUSSION

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SESSION **3**

Chairman : J. DELACOUR

Institut Français du Pétrole, Rueil-Malmaison, France

The use of pressuremeters in a marine environment

THE PUSH-IN PRESSUREMETER

INTRODUCTION

The recovery of oil from the North Sea, with its relatively deep water and hostile climatic conditions, has given rise to the need for production platforms of a highly complex nature designed to withstand a combination of high wind pressures and large wave forces. The design methods adopted for these platforms are sophisticated and involve both static and dynamic analyses of structural performance. If realistic mathematical predictions of the structural performance are to be achieved, it is now recognised that the foundations and surrounding soil must be accurately represented in the analytical model and a number of design packages have been produced which are capable of incorporating soil-structure interaction effects. The inclusion of soil properties within the structural analysis package places an additional emphasis on the need for accuracy in establishing parameters to define soil response to applied loading.

It has generally been the practice in the North Sea to carry out Geotechnical investigation using a combination of static cone penetration testing and "undisturbed" sampling with laboratory testing. Although this procedure gives acceptable accuracy to predict upper and lower bound strength parameters, it does not provide accurate information on in-situ soil response to applied loading.

With a view to extending the range of equipment available for soil testing in the North Sea, the Department of Energy commissioned the Building Research Establishment (BRE) in 1975 to develop a pressuremeter suitable for use in U.K. Continental Shelf waters. The initial conclusion from the BRE desk study was that the offshore pressuremeter should be wireline operated to be compatible and interchangeable with equipment in general use by the major Geotechnical site investigation contractors. This dictated that the only

* *Stressprobe Limited, Glasgow, Scotland, UK.*

** *Building Research Establishment, Garston, Watford, England, UK.*

****The British Petroleum Company Limited, London, England, UK.*

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connection between the equipment and the survey vessel would be seven electrical conductors within the wireline for raising and lowering the device. Further study confirmed that seven conductors were sufficient to allow the use of an electrically driven "down-hole" pump with the facility to relay volume change and pressure measurements through the conductors to the survey vessel.

A study of alternative forms of insertion procedure was also undertaken with initial emphasis placed on a self-boring device. It was concluded however that a design for a self-boring device which incorporated both wireline operation and a maximum diameter of 78mm would be too complicated and would be limited in application to gravel free soil. The concept of an "open hole" pressuremeter was also rejected on the grounds that the hole would be liable to fill with cuttings prior to insertion and in addition it was considered that the relaxation in the borehole walls would be of unacceptable proportions for a significant number of soil conditions.

The solution finally researched and developed was an open-ended push-in pressuremeter which allows soil to pass up inside the head on insertion. A specially designed cutting shoe incorporating a stress relieving step on the inside was adopted to minimise the thrust required for insertion and it was considered that this design achieved a balance between undisturbed insertion and practical operation.

Between 1977 and 1979, the prototype instrument was tested on land-sites and 'calibrated' against other tests, including laboratory tests on samples, other in-situ tests and full scale pile tests (see Marsland and Windle 1982). A number of opportunities were offered by BP and other oil companies to carry out sea trials, and these gave further understanding of the required operational procedures. In 1980 a private company, Stressprobe Limited, was formed primarily for the purpose of commercial development and operation of the push-in pressuremeter (now called the "Stressprobe") and a number of improvements to the prototype instrument and operating procedures have since been incorporated in a new version of the device.

The first offshore tests with the Stressprobe were performed in July 1980 under contract to BP in Block 7/12 of the North Sea from the drillship M.V. MARINER. Since then the Stressprobe has been used offshore at various locations in the North Sea and on the Australian North West Continental Shelf (see Table 1). It has been used in water depths of 70-185m and tests have been performed at depths between 3 and 110 metres below the seabed.

Tests have been carried out offshore in clays with undrained shear strengths ranging from 15 to 470 kN/m², as well as calcareous sandy silts.

Series of Stressprobe tests have also been carried out on land at test bed sites at Cowden and Madingley and the results from those tests have been compared (Fyffe et al 1982) with strength parameters derived from large diameter plate loading tests and from Camkometer self-boring pressuremeter tests. In both cases good correlation was obtained.

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TABLE 1

OFFSHORE PRESSUREMETER TESTS 1980/1981 -
STRESSPROBE LIMITED

SITE	NO. OF TESTS	TEST DEPTH (below mudline)	SOIL TYPE	WATER DEPTH
ULA FIELD BLOCK 7/12 Norwegian Sector	4	5-20m	Silty Clay Cu=40-100kPa	70m
MAGNUS FIELD BLOCK 211/12 British Sector	5	4-70m	Stiff Boulder Clay Cu=160-350kPa	185m
GAS GATHERING BLOCK 16/16 British Sector	4	4-15m	Clay Cu=30-100kPa	135m
ANDREW FIELD BLOCK 16/28 British Sector	16	4-75m	Very soft to very stiff silty Clay Cu=15-460kPa	120m
NORTH RANKIN FIELD Australian North West Continental Shelf	10	3-110m	Fine sandy silt	125m
BLOCK 34/10 Norwegian Sector	10	5-30m	Sandy Clay Cu=180-470kPa	135m

2. DESCRIPTION OF THE STRESSPROBE EQUIPMENT

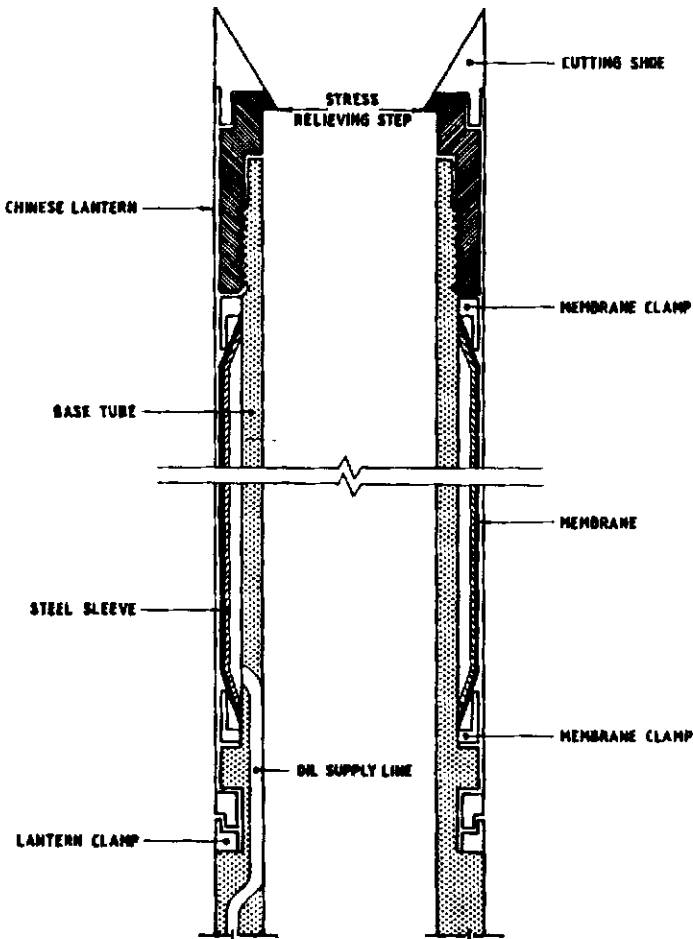
The details of the push-in pressuremeter are described by Henderson et al (1979) and a brief resume only will be given here.

The equipment comprises:

- a hollow pressuremeter head with a protected adiprene membrane on its outside (Fig. 1).
- a spacer section up which soil can pass.
- a pressure developer from which oil is pumped to inflate the pressuremeter (Fig. 2).

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- (iv) a latch against which the pressuremeter is pushed into the soil using the drill string for reaction.
- (v) an electro-mechanical cable to the surface.
- (vi) a control box.
- (vii) a recording system.



SECTION THROUGH PRESSUREMETER HEAD

FIGURE 1

THE PUSH-IN PRESSUREMETER

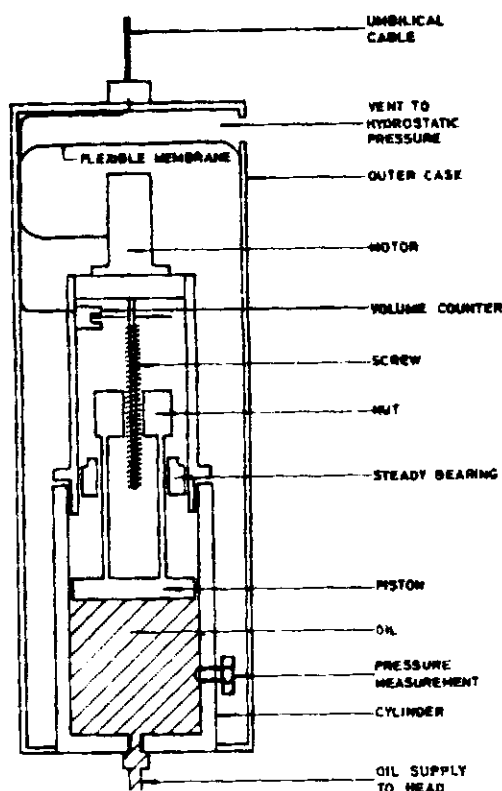


DIAGRAM OF PRESSURE DEVELOPER
FIGURE 2

The original development of the Stressprobe was carried out in collaboration with McClelland Engineers Limited and Fugro B.V. The hardware was therefore manufactured to be compatible with both the McClelland "Stingray" and Fugro "Wison" systems. Commercial Stressprobe tests have now been carried out successfully using both types of equipment.

A land based version of the Stressprobe has also been developed which can be pushed from the surface using drilling rods or latched in and pushed by the drill pipe.

Since publication of the 1979 paper, improvements have been made to the adiprene membrane protection system and in the adoption of a new type of electronic control box. Modifications have also been made to facilitate rapid turn round time between tests by incorporating quick release couplings in the oil system and between the various sections of the pressuremeter.

Dimensions and specifications for the device are given in Table II.

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TABLE II

PHYSICAL CHARACTERISTICS OF THE STRESSPROBE

<u>DIMENSIONS</u>	<u>OFFSHORE VERSION</u>	
	Length (mm)	Diameter (mm)
Pressuremeter Head	695	78
Inflatable Membrane	330	78
Spacer	Variable to suit latching equipment	78
Pressure Developer	1740	85
Latching Equipment	Variable to suit insertion system	
<u>DETAILS</u>		
Insertion Stroke	750 mm (approx.)	
Insertion Capacity	160 kN	
Volumetric Expansion	20% of initial volume	
Rate of Inflation	2% volume increase/minute	
Inflation Capacity	3500 kN/m ² above hydrostatic	
Resolution of Recording	Volume : $\pm$ 0.125% of full scale	
	Pressure : $\pm$ 0.5 kN/m ²	
Control of Inflation	Strain controlled (can be stopped and reversed at any stage of test).	

3. DEPLOYMENT AND SHIP BOARD OPERATIONS /

Since the Stressprobe is compatible with existing wire-line systems, it is possible to carry out pressuremeter tests, C.P.T.'s and sampling in the same hole. It is also possible, and sometimes desirable, to take a 50mm diameter sample in the test position prior to inserting the pressuremeter head. In very stiff soils this precoring facilitates insertion of the head.

Prior to the start of operations the Stressprobe hardware is assembled on the deck of the ship and checked, both electrically and mechanically. The oil supply system is de-aired and the pressure transducer is then calibrated against a reference gauge. When the system is calibrated ready for testing, each head assembly is inflated in air and the pressure recorded through a complete cycle. This procedure defines the pressure generated within the

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system at any given point in the inflation or deflation cycle and allows the appropriate correction to be made to the pressures recorded during actual testing.

Once the hole has been drilled to depth and flushed with drilling mud to remove cuttings, the instrument is introduced into the drill pipe. It is then lowered down the drillstring and on reaching the bottom automatically latches into position. The reaction system is mobilised and the pressuremeter is inserted into the soil beneath the base of the borehole. On insertion, the pressuremeter is left for a short period, to allow the pressure to stabilise prior to performing the test. The test includes inflation to maximum volume, at least one unload-reload loop over a range of 3% volumetric change and deflation to zero strain. The reload loop is usually performed once the maximum strain has been reached. The rate of inflation and deflation is constant at approximately 2% volume change per minute.

Once the test is completed, the Stressprobe is withdrawn by means of the reaction system, and then unlatched and retrieved by pulling up on the wire-line. When on deck, the head and spacer assembly which is full of soil, is removed and a calibrated replacement fitted to prepare the equipment for further testing. This allows a quick "turn round" time between tests. Soil can subsequently be extruded from the detached head and spacer assembly using the standard extrusion equipment carried on the drillship.

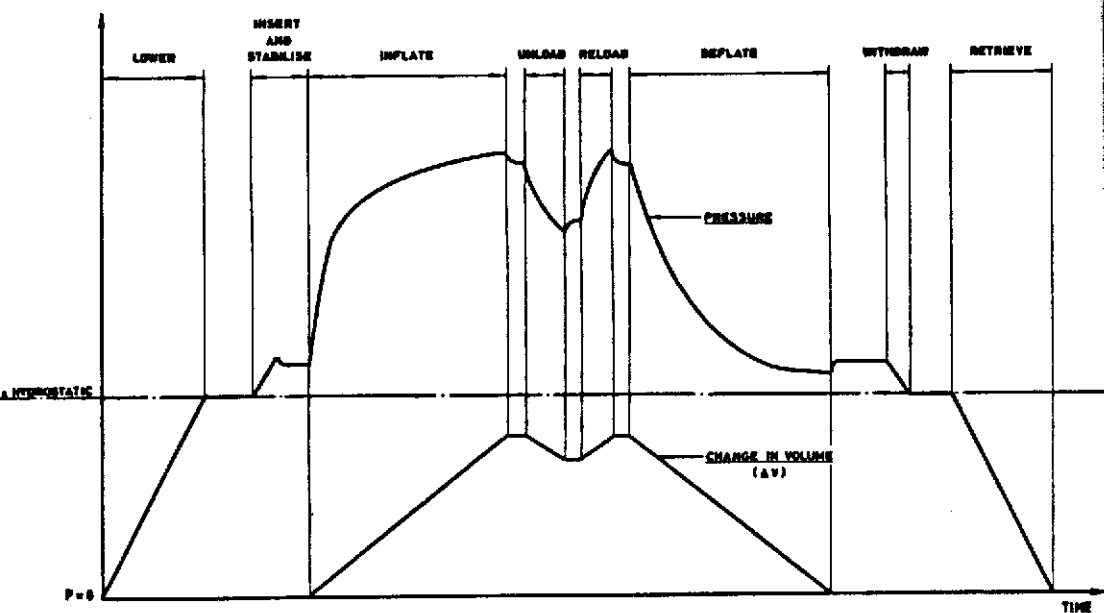
4. TREATMENT OF DATA

The output from the pressuremeter is recorded on standard data loggers or chart recorders in the form of volume change and pressure developed against time. A typical plot from the chart recorder is shown in Figure 3. Prior to analysis, the output is split into the following sections.

- (i) pressure recorded with pressuremeter hanging at deck level prior to lowering.
- (ii) pressure increase due to hydrostatic/fluid pressure when lowering the pressuremeter down the drillstring.
- (iii) pressure increase on insertion of the pressuremeter head into the soil and any subsequent pressure drop after insertion.
- (iv) pressures developed during the test.
- (v) pressures recorded on retraction of instrument from soil and reeling it back to the ship.
- (vi) pressure recorded with pressuremeter hanging at deck level after retrieval.

The pressure recorded at deck level before lowering is taken as the datum for the test. This is checked with that recorded when the pressuremeter is

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STRESSPROBE PRESSUREMETER
TYPICAL PLOT FROM CHART RECORDER

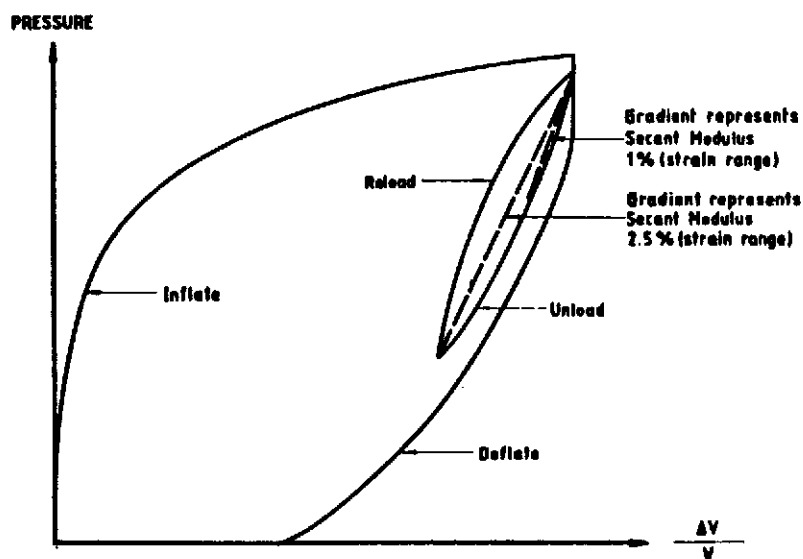
FIGURE 3

returned to the ship to ensure that no "zero shift" of the pressure transducer has occurred. The downhole fluid pressure is noted, but no check may be obtained from this as densities of mud and suspended solids may cause it to differ significantly from hydrostatic pressure.

The first of the measured data used in analysis of results is the pressure developed on insertion. This confirms that the pressuremeter has been pushed into the soil and if the in-situ horizontal stress is already known, can be used to give an indication of the quality of the insertion. Insertion pressures varying from "at rest" pressures influence the method of interpretation, but do not render the test data invalid.

The pressures measured during the test arise from the soil resistance to loading, along with pressure generated within the equipment. As these internally generated pressures are measured before testing commences, they are subtracted prior to carrying out any analyses. A typical soil response throughout a complete pressuremeter cycle is given in Figure 4.

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STRESSPROBE PRESSUREMETER

TYPICAL TEST RESULT

FIGURE 4

5. INTERPRETATION OF TEST RESULTS

The data available from the Stressprobe differs from that of most other pressuremeters with respect to the fundamental feature that volume change is totally controlled and the pressure response to the induced deformation is continuously monitored. This system has the following advantages:-

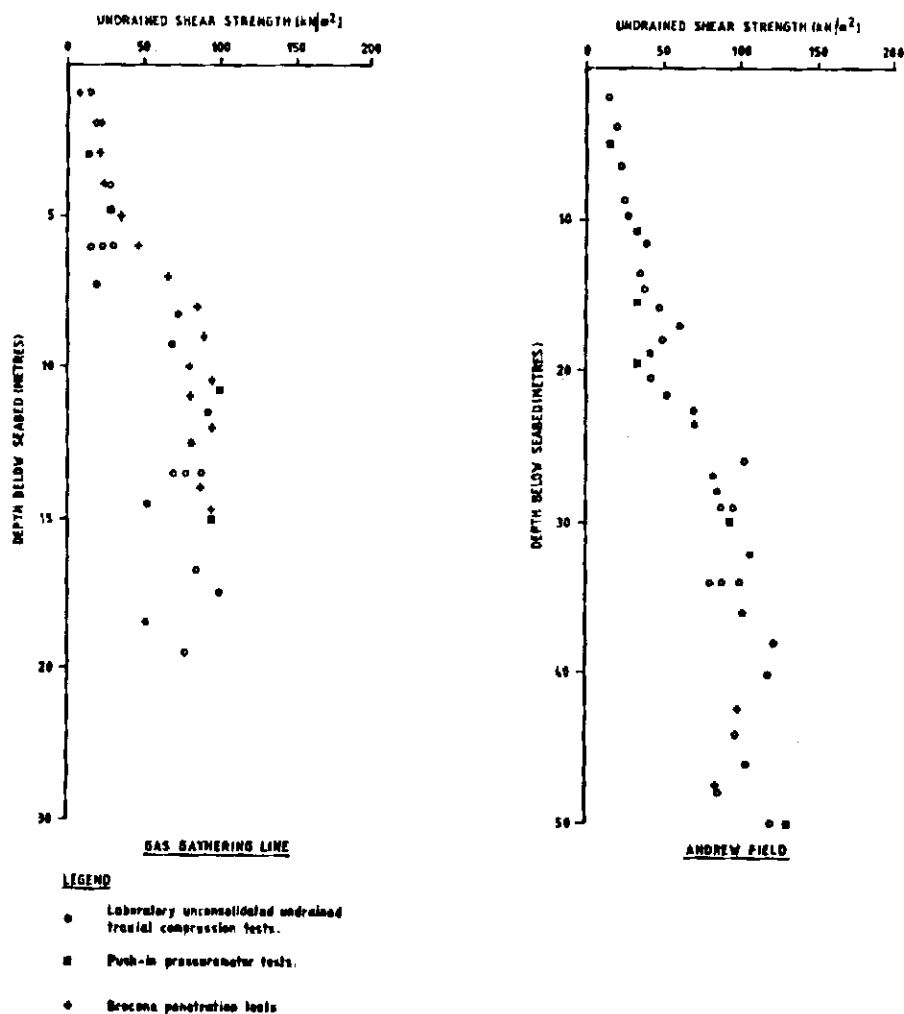
- 5.1 It is possible to perform the tests at a controlled rate of strain.
- 5.2 Once the test has commenced there is no necessity to wait for pressure to stabilise at a given volume.
- 5.3 The test starts immediately, with no "lift off" point to be identified.
- 5.4 It is simple to perform reload loops of any required strain increment.

The interpretation of the Stressprobe test results to obtain shear strength parameters is generally carried out using the techniques described by Gibson and Anderson (1961) and Marsland and Randolph (1977). A discussion on the relative merits of these alternative systems and the method adopted to compensate for disturbance on insertion are given in a paper by Fyffe et al (1982).

Shear strengths determined, using the Gibson and Anderson theory from

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two series of Stressprobe tests carried out on behalf of B P are given in Figure 5. It can be seen that the values obtained are in good agreement with laboratory tests and with shear strengths derived from cone testing. The shear strengths obtained from the alternative Marsland and Randolph technique give similar results.



In addition to determining the undrained shear strength of soil from a Stressprobe test it is also standard practice to determine values for Shear Modulus.

Shear modulus is generally calculated from the pressure variation during a

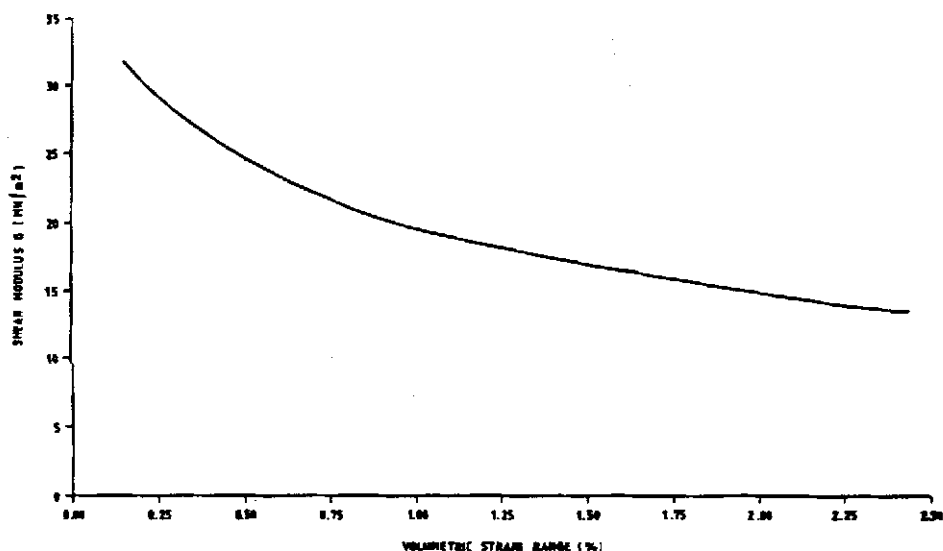
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relatively small strain change in a reload loop using the Gibson and Anderson equation,

$$G = \Delta p / \Delta \left(\frac{\Delta V}{V} \right)$$

where G is shear modulus and ΔP is pressure variation over a strain range of $\Delta \left(\frac{\Delta V}{V} \right)$. Reload loops are used in an attempt to isolate the elastic response of the soil and although this can be performed at any stage during the inflation or deflation cycle, the standard practice is to perform a reload loop at the maximum strain of 20%. The interpretation of Shear Modulus is apparently insensitive to disturbance caused during insertion and the values obtained have consistently been found to be repeatable. Figure 4 shows a typical reload loop and the method of obtaining the moduli values is indicated.

In practice soils do not exhibit perfectly elastic/plastic behaviour as is assumed in the theory and for this reason the reload loop is not perfectly linear. It is therefore necessary to choose the secant modulus appropriate to the required strain or stress range for the design condition. Figure 6 shows a typical variation of secant modulus with strain range and a similar relationship is found when modulus is plotted against stress range.



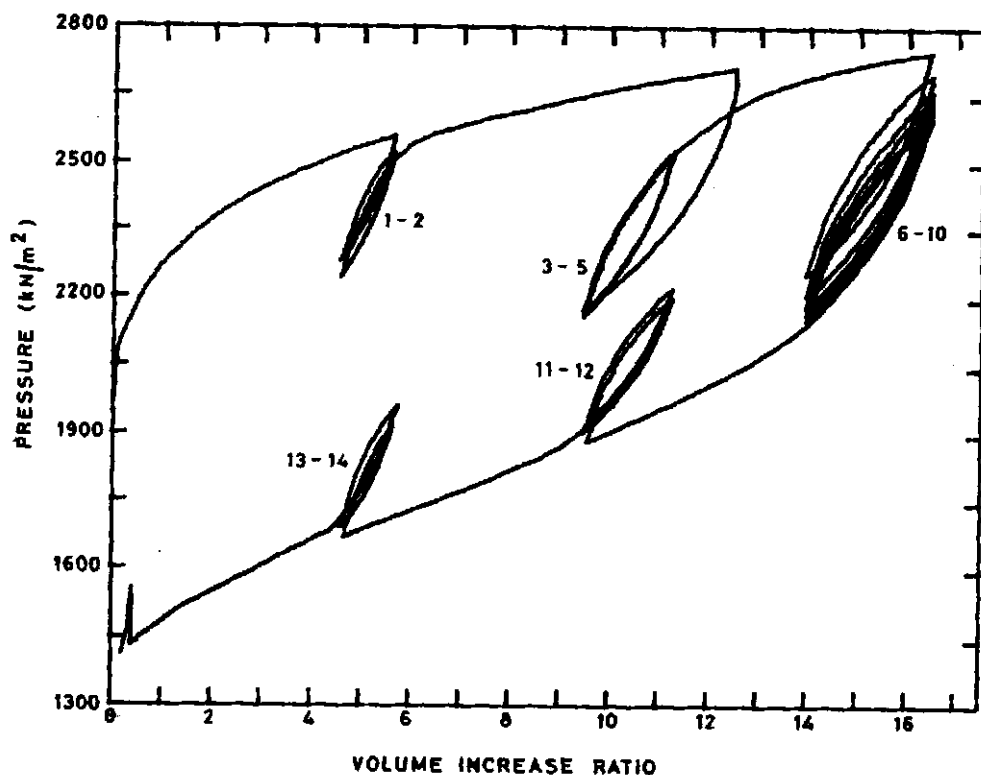
VARIATION OF SECANT SHEAR MODULUS WITH STRAIN RANGE (SEE FIG. 4)

FIGURE 6

For a series of tests commissioned by the Norwegian Geotechnical Institute on behalf of Statoil at Block 34/10 of the North Sea, a number of

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reload loops were carried out at different stages of the test and at different stress levels. Figure 7 shows a test with 14 reload loops performed at various stages of the inflation and deflation cycle. The soil was a Glacial Till and the depth below sea bed was 26.7 metres.



OUTPUT FROM PRESSUREMETER TEST WITH MULTIPLE RELOAD LOOPS

FIGURE 7

A study of this test reveals the consistent shape of the reload loops and confirms a number of interesting features. It can be seen that the gradients of the reload loops are very similar irrespective of the stage of the test at which the reload loop is performed. Figure 8 shows shear modulus plotted against strain range for a number of the individual reload loops. The shear modulus values for one particular strain range do not vary from the average by more than 8% for all reload cycles.

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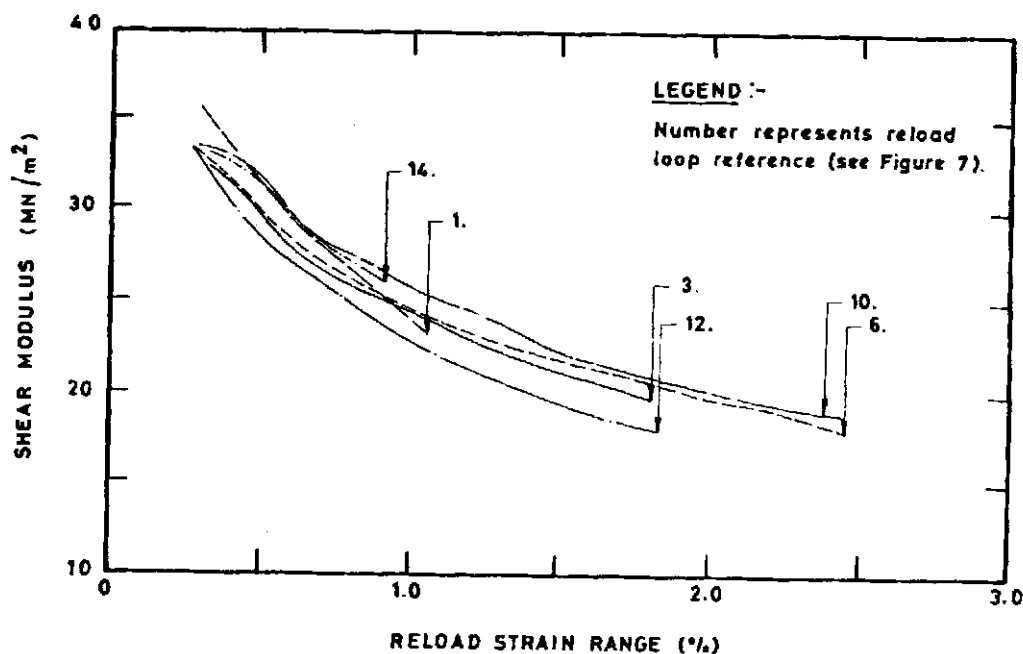


FIGURE 8

As only one offshore test with reload loops performed during the deflation cycle as shown in Figure 7 has been performed at the time of writing this paper, it is not possible to conclude that a similar repeatability will be obtained for all soils.

For the test illustrated the shear strength of the soil as determined from the test is 211 kN/m². This gives ratios of shear modulus to shear strength which vary from 90 to 200 depending on the strain range chosen for modulus calculation.

A further interesting feature of the test illustrated is the tendency for a progressive loss of pressure during the execution of the multiple reload loops. This is due to a combination of soil creep and excess pore water pressure dissipation during the test. It is interesting to note that no such loss of pressure occurred during reload loops 4 and 5.

CONCLUDING REMARKS

The ability of the Stressprobe equipment to perform tests in a relatively wide variety of soils has been established. The form of test can be varied to accommodate the determination of shear modulus values for any strain range at a variety of stress conditions throughout the test cycle. The actual test specification should vary depending on the information required and in particular reload loops should be performed at stress levels simulating working conditions.

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The tests performed to date have suggested that shear modulus values are insensitive to the stage of the test at which the reload loop is performed and in addition it is apparent that soil stiffness is not destroyed by the imposition of relatively large deformations and subsequent relaxations.

Although the performing of multiple reload loops during the inflation cycles is of interest, the decision to do so must be tempered by the knowledge that this reduces the accuracy with which undrained shear strength can be established. Multiple reload loops performed at maximum inflation however, or during the deflation cycle have no effect on the calculation of undrained shear strength.

Further testing incorporating multiple reload loops is highly desirable to establish the universal applicability of the comments made in this paper regarding the particular test shown in Figure 7.

ACKNOWLEDGEMENTS

The Authors wish to express their gratitude to the Director of the Building Research Establishment for his permission to publish this paper and to the British Petroleum Company Limited and Statoil for their permission to discuss particular results from tests carried out on their behalf. Assistance from the Norwegian Geotechnical Institute is also gratefully acknowledged.

SYNOPSIS

The paper discusses the development of a strain controlled push-in pressuremeter (Stressprobe) specifically designed for offshore use. The background to the development work and to the alternatives considered are discussed and the reasons for the final choice of design are given. A description of the operation of the equipment and an outline of the test procedure are presented with particular reference to practical aspects such as time required to prepare equipment and perform test routines.

The paper also presents some typical results obtained from Stressprobe soil surveys at sites in the North Sea and incorporates a brief discussion on the flexibility of the available test procedures and how these can be adapted to provide particular information on ultimate shear strength and on shear modulus across predetermined stress or strain ranges.

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WIRE-LINE EXPANSOMETER

ABSTRACT

The actual development of offshore structures in deep water is characterised by an acute need of soil survey tools for testing the structures' foundations. The aims of these tools is to determine in-situ the characteristics of undisturbed soils.

Devices using inflation of a cylindrical probe - pressuremeters and expansometers - allow the measurements of load-displacement relationship which can give directly the stress-strain law of a given soil condition.

In this way, SYMINEX developed two kinds of expansometers :

- the PITE (Portable Intervention Expansometer) adapted to field and nearshore surveys - in exploitation since January 1981 ;
- the WILE (Wire-Line Expansometer) for offshore use. This equipment is adapted to the wire-line method and is lowered through a geo-technical drill string with an electrical connecting cable.

The WILE contains three parts :

- a measuring sonde, the same for WILE and PITE, with pore pressure sensor and latching device contained in drill bit ;
- a watertight container including the oleopneumatic injection system, the electronic assembly and the pressure transducers. On the top, charge and discharge containers are overlaid by cable head connection ;

* Syminex, Marseille, France.

WIRE-LINE EXPANSOMETER

- a surface unit for data acquisition and control.

A methodology concerning setting, test and interpretation has been set up after extensive experiments performed on various on-shore and nearshore sites at "Etang de Berre". Qualification tests have been conducted offshore, in 300 m. depth of water. These tests have verified the overall operation of the complete system.

The SYMINEX Wire-Line Expansometer is now entering into its commercial exploitation phase.

1. DESCRIPTION OF THE EQUIPMENT

The expansometer is composed of the following elements :

- the expansometric probe
- the energy compartment - regulation/measurement
- the control and data acquisition unit

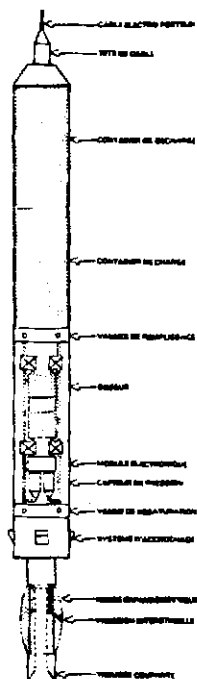


Fig.1 Pressuremetric device

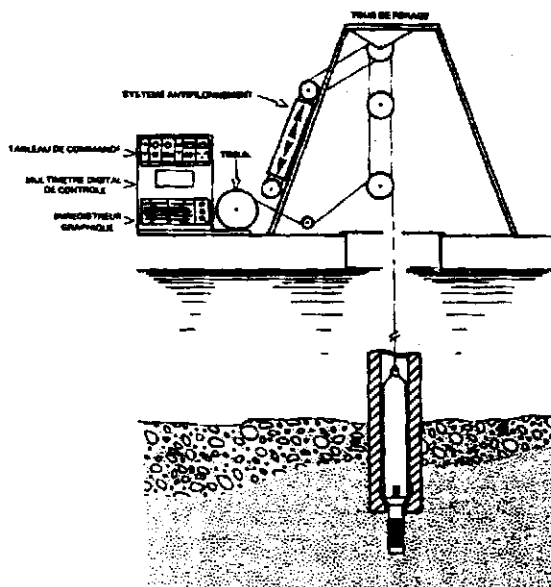


Fig.2 Working diagram

WIRE-LINE EXPANSOMETER

A. THE EXPANSOMETER PROBE

This unit consists of the probe and the latching device. It is a cell ($\emptyset$ 2,6" : 34,7") composed of a stainless steel cylinder clothed with a flexible neoprene membrane. The membrane is fixed by means of a clamping-cone which allows easy changing of the membrane. A tubulure ensures the arrival of the expansion oil between the metallic cylinder and the membrane.

The probe has a diameter of 2,6" (6,5mm) and an active length of 13" (332mm), which ensures a L/D ratio of 5. The membrane can be inflated up to 3,9" (100mm) in diameter corresponding to a 93,5 cu.ins. (1500 cm³) volume of injected oil. The pore pressure is measured through a filter ceramic, set in the middle of the membrane. The probe is equipped on its base with a cutting edge of 2,64" (67cm). The interior of the cell is lined with a sampling tube which enables the catching of re-moulded soil samples ($\emptyset$ 1,7" 32mm).

On the top of the corer, a ball opturator allows the outflow of water during the installation of the tool. A flyleaf catcher holds the sample in position.

The latching device allows the gravity locking of the wire line probe to the drilling tool. A special notch in the drill casing enables the setting of the latching tool. The latching device is located between the regulation/measurement compartment and the probe. During the descent of the tool, the knives located at the base of the latching device is compressed by the weight of the tool and the four clamping fingers are pushed out into the fitted notch of the casing. At the end of the test the extraction of the cable extends the latching device, the four clamping fingers retract and the pressuremeter tool can be lifted.

B. THE ENERGY/REGULATION/MEASUREMENT COMPARTMENT

1. The Energy Container

This consists of a 4,9' (1,5m) high cylinder, including, in its upper part, the binding head between the electro-bearing cable and the pressuremeter tool, and in its lower part a set of valves for the preparation of the tool before the execution of the test.

The container consists of two reservoirs each of 2,5 litres. The injection fluid (1,5l of low viscosity oil) is pumped to the loading reservoir and then placed under pressure by introduction of compressed nitrogen. At the end of the test, oil circulates from the probe to the

WIRE-LINE EXPANSOMETER

to the discharge reservoir, maintained under a relative void. On the surface, a valve system enables the return of the oil to the load reservoir. The fluid has then only to be put under pressure for the tool to be ready for the execution of a new test.

2. The Regulation/Measurement Container

This consists of a tubular unit the ends of which are obstructed by watertight walls. The energy container and the regulation/measurement device are connected by means of half shells easily dismountable.

The container houses -

- a feed regulator controlled by a set of electrovalves
- an electrovalve for the discharge circuit
- an electronic module
- two pressure gauges.

The system has a set of valves on its extremity allowing the saturation of the total pressure and pore pressure circuits and the corresponding measuring gauges.

- The feed regulator

This is composed of a jack with a free piston which enables the injection, through a set of electrovalves, of 10 cm³ of oil at each movement.

- The discharge electrovalve

At the end of the test, the discharge electrovalve connects the inflated probe with the discharge reservoir. The difference in pressure between the probe and the reservoir enables the rapid circulation of the total amount of oil.

- The electronic module

This regulates the electric power supply of the electrovalves and the pressure gauges and defines the necessary temporization for the correct working of the system.

- The pressure pumps

These are electronic stress gauges with a full measuring range from 0 to 30 bars and an accuracy of 2% of the measured value and a resolution of 10 mbars.

Other gauges exist with ranges of 20, 50, 100 and 200 bars which can be installed as required.

WIRE-LINE EXPANSOMETER

One gauge is used for the measurement of the pore pressure (PI), another for the measurement of the inflating stress or total pressure (PT). Both are supplied with a circulation system to ensure a good hydraulic saturation of the instrument rooms.

3. The Data Acquisition Control System

The surface device consists of 3 elements -

- a winch
- a command panel
- a graphic recorder with a digital multimeter
- The winch

This ensures the lifting and lowering of the tool and is classical logging equipment with a four conductor cable type 4D3/16" that allows the electronic communication for command, control and measurement.

- The Command Panel

The supply section allows the unit to function below the surface. The voltage value is related to the length of the cable. The command section comprises several signals :

- the injection, with indirect indication of the selected volume
- the measurement with indication of the voltages corresponding to the pore pressures (PI) and the total pressures (PT) and recording of the values on a graphic recorder
- the deflation.

- The Graphic Recorder

Type 1220 MF E portable. This allows the recording on two tracks of the pore pressure (PI) and total pressure (PT) signals. A 50160 recorder in the logging rack is another possibility. The digital multimeter allows the control of any anomaly during the test (burst membrane, badly locked tool, etc).

II. INSERTION PROCEDURE

A. INSERTION PRINCIPLE

The tool is designed to carry out mechanical soil resistance tests

WIRE-LINE EXPANSOMETER

in a geotechnical boring. The test is carried out in the soil under the drill casing. The boring support must be constituted of a set of drill rods and drill collars with a minimum inner diameter of 3,35" (85mm). The apparatus is lowered on a cable following the wire-line method, through the tubing and is locked to the lower part of the casing in a special notch. The tool, locked with the casing, is then inserted by gravity into the soil. The drill tubing is immobilised by a seabed control system (Syninex Sea-Jack) in order to avoid any heave action.

To limit soil removal to a strict minimum, the following insertion method has been adopted.

- A sampler, (type push sampler), of a diameter inferior to that of the probe, is inserted into the soil and then extracted. A sampler of this type, 1,97" diameter (50mm), is used for the coring of undisturbed samples for laboratory tests.
- The probe ($\emptyset 2,56"$ 65mm) with the cutting edge rebores the former hole.

The advantage of this method consists in the fact that an undisturbed sample is obtained for testing, the soil remains in its elastic state and the insertion of the probe is facilitated.

B. THE INSERTION PROCEDURE IS SHOWN BELOW

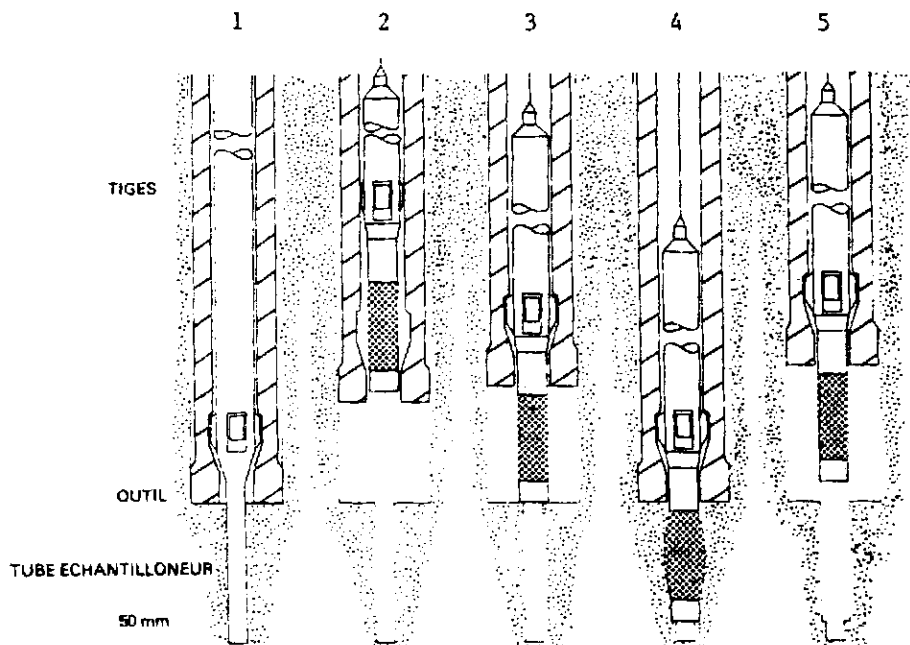


Fig. 3 Description overleaf

WIRE-LINE EXPANSOMETER

1. Coring with a sampler ($\phi 50\text{mm}$)
2. Lifting the tubing of the cell length
3. Locking the pressuremeter tool by means of the latching device
4. Insertion of the probe with cutting edge ($\phi 67\text{mm}$) and execution of the pressuremeter test
5. Lifting the tubing and unlocking the pressuremeter

III. TYPES OF TESTS AND DATA INTERPRETATION

A. TYPES OF TESTS

Three test procedures exist :

- stress controlled tests
- strain controlled tests
- relaxation tests.

Stress and strain controlled tests can be executed in a cyclic form, inflating and deflating the membrane. They can be carried out with a number of constant pressure or volume increments or at constant evolution during the pseudo-elastic phase of the test. Relaxation tests allow the testing of the soil either in the plastic phase, by applying a stress or strain close to the limit pressure, or in the pseudo-elastic phase with a pressure close to the creep limit.

From any expansion test three parameters should be acquired :

- volume
- expansion pressure
- pore pressure.

Time can be mentioned as a fourth intrinsic parameter.

1. Stress Controlled Tests

The operator controls the expansion pressure.

2. Strain Controlled Tests

The operator controls the strain volume. This type of test allows the control of the loading speed of the soil. Tests with constant increments and continuous inflation can be distinguished.

- Tests with Constant Increments

Volume increments are made by a process similar to the stress

WIRE-LINE EXPANSOMETER

controlled tests. Stresses for a given volume are measured after each constant increment.

- Continuous Inflation Tests

These allow uniform testing of the evolution stages of the soil during the test. The result takes the form of a recording of the pressure/volume relationship.

3. Relaxation Tests

These tests consist in the loading of the soil to a given pressure and in following the evolution of the pressures (PI and PT) in relation to the time.

The Syminex Wire-Line Expansometer has been conceived to carry out strain controlled and relaxation tests.

B. INTERPRETATION OF TEST RESULTS

Only the tests which can be carried out with the Syminex Expansometer will be mentioned.

1. Strain Controlled Tests

Pore pressure (PI) and total pressure (PT) are collected. The inflation volume is known.

From the beginning of the test, pore pressure increases until the creep limit is reached. Above the creep limit the pore pressure tends to increase at a slower rate.

The measured total pressures allow the setting of loading diagrams of the pressure in relation to the volume

$$\left(V, \frac{1}{V}, \frac{\Delta V}{V_0}, \frac{\Delta V}{V_0 + \Delta V} \right)$$

From these diagrams the principal mechanical soil parameters can be defined, deformation modulus and limit pressure.

The pressuremeter modulus characterises the pseudo-elastic phase of the test and has an important role in foundation settlement calculation. The limit pressure PI corresponds to the ultimate rupture point of a soil, submitted to a uniform increasing pressure on the walls of a cylindrical cavity. Limit pressure is used in the calculation of foundation bearing capacity.

Another soil characteristic is given by the creep pressure (Pf).

WIRE-LINE EXPANSOMETER

P_f is generally used for the characterisation of the soil in relation to the limit pressure. In the case of a saturated clay, the differentiating of the pressure/volume function gives the stress/strain curve which maximum value, at low strain, corresponds to the undrained cohesion.

During these strain controlled tests, with constant volume increments or with continuous inflation, several inflation/deflation cycles can be effected in the pseudo-elastic phase. Both rebound and reload moduli may then be calculated even as alternate moduli. These tests are useful to understand the behaviour of soils subjected to cyclic loading as occurs in a marine environment.

2. Relaxation Tests

These tests are carried out when the approximate values of P_f and P_l are known, and consist in rapid soil loading. This can be done in the range of pseudo-elastic conditions (pressure inferior to P_f). The relaxation in time of the total pressures and the pore pressures can then be read. The total pressure tends, after a variable time range, to a value close to the "at rest" horizontal pressure. When stabilization is reached, soil can be loaded again in the range of plastic conditions, the pressure relaxation then tends to a residual value close to the creep pressure.

These relaxation tests are lengthy and are, therefore, rarely applied.

IV. TEST RESULTS

Several test series have been carried out :

- strain controlled tests (Fig.4)
- creep and relaxation tests (Fig.5)

The principle of the strain controlled test has been improved and gives entire satisfaction. The advantage of the test consists in the perfect control of the soil loading rate.

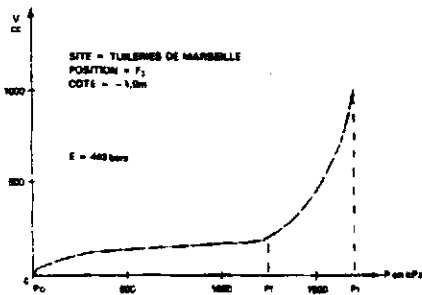


Fig. 4 Strain controlled tests

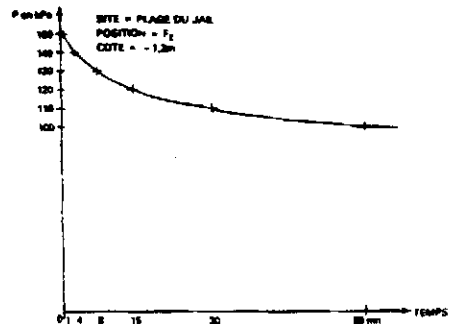


Fig. 5 Creep and relaxation tests

WIRE-LINE EXPANSOMETER

- The measuring cells (membranes and probes), the same for WILE and PITE, have been tested with the PITE during a geotechnical survey in the Gulf of Guinea. These, realised jointly with a Menard pressure-meter, allowed the comparison of results obtained with both devices.

Symynex membranes were shown to be of a very high resistance for a relatively low inertia. Their on-site changing can be carried out with ease and rapidity.

The values of limit and creep pressures obtained with both tools were similar. The probes were set in the ground by jacking.

- The WILE was tested on different sites near Marseilles :
 - . Site N°1 : red dumping muds from Vitrolles : unconsolidated clays
 - . Site N°2 : Tuileries Marseilles Quarry : stiff overconsolidated clays
 - . Site N°3 : Jail Beach Etang de Berre : clayey silts
 - . Site N°4 : Salt Flats of Fos-sur-Mer : silty sands with organic matter.

For each test, first of all a $\phi 100\text{mm}$ hole was made for the insertion of the main part of the apparatus, then a boring with a hand auger $\phi 63,5\text{mm}$ is made to carry out the test. The probe was inserted with its cutting edge ($\phi 67\text{mm}$). The insertion conditions were comparable to the offshore insertion method.

Results of the Strain Controlled Tests with Constant Increments

Red muds from Vitrolles : bauxite treatment waste (embankment)

tests value	1 F0=0,9m	2 F1=1,0m	3 F2=2,1m
PfKpa	135	1050	124
PlKpa	250	165	140
EKpa	1950	1300	800

These values are characteristic of a loose soil (large strain values during the pseudo-elastic phase)

E — varies in range between $P_l - p_o$

6, 5 and 9 which represents an underconsolidated clay.

The characterisation of the soil by means of the test corresponds well with the reality.

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Tuileries - Marseilles : stiff overconsolidated clays

tests	1	2	3
value	F1=-1,1m	F2=-0,9m	F3 = 1m
Pf Kpa	800	1350	1000
P1 Kpa	1080	1700	1500
E Kpa	24400	44300	23300

The values of P1 are normal for this type of clay.

$\frac{E}{P1}$ varies in range between 15, and 26, which represents an overconsolidated soil (measurements were taken at the bottom of the quarry)

Jail Beach : clayey silts with organic matter

tests	1
value	F1=-1,3m
Pf Kpa	75
P1 Kpa	90
E Kpa	250

$\frac{E}{P1} = 9$; these values characterise a silty soil

Salt Flats of Fos : silty sands with organic matter

tests	1
value	F1=-1,2m
Pf Kpa	90
P1 Kpa	110
E Kpa	1250

$\frac{E}{P1} = 11$; these values characterise that a muddy sand has been encountered

WIRE-LINE EXPANSOMETER

V. CONCLUSIONS

An expansometer has been developed to carry out pressuremeter tests in water depths up to 300m or even more.

- . The WILE is a wire-line tool. It can thus be used at any moment of a geotechnical boring. It is easily installed and requires no special arrangements.
- . The WILE can carry out strain controlled tests and relaxation tests. It will soon be fitted out for the execution of cyclic tests.
- . The main advantage of the WILE is the maximum reduction of loading losses since the oil compartment and injection device are close to the measuring cell. It can work as well in the small strain range (doses of 10 cm³) as in the large strain range (injected volume can reach 1500 cm³).
- . The WILE allows the definition of mechanical marine soil properties and the calculation of the foundation settlements and stability.

THE LPC-TLM PRESSIO-PENETROMETER

ABSTRACT

New equipment designed for in situ investigation of soils under depths of water of as much as 100 meters is presented. This equipment may be used in particular to measure the static or dynamic penetration resistance of the soils and to carry out pressuremeter tests, especially cyclic ones, with a high degree of reliability and precision. The equipment includes an electronic system for automatic control of the tests and real-time processing of the test data. This makes it possible to determine the bearing capacity and settlement characteristics in real time and so to give an immediate opinion concerning the conditions affecting structural foundations.

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** Techniques Louis Ménard, Granville, France.

*** Commissariat à l'Energie Atomique, Bruyères le Châtel, France.

INTRODUCTION

The demand for soil reconnaissance has grown rapidly in recent years, and this growth has been accompanied by more stringent requirements on the part of consultants, who wish to obtain complete investigations rapidly and at the lowest possible cost. Where, not so long ago, it was still possible to bring several types of instrument to a site, alter the approach employed as the reconnaissance progressed, and then process the information so gathered once back in the office, it is today necessary to operate quickly under the most difficult site conditions, and often to give the consulting engineer or the contractor the information he wants immediately.

Developments in operations at marine sites are typical in this respect : the working conditions, the cost of the naval support, and the great variety and complexity of the geotechnical problems posed have all been incentives for the development of new methods of investigation meeting the criteria set forth above.

It is to deal with a particular problem of this type that the Atomic Energy Commission, the French Roads and Bridges Laboratories, and the Société Techniques Louis Ménard have worked together to develop a submersible system suitable for marine soil investigation under depths of water of as much as one hundred meters.

In this paper, we shall first give a brief description of the system as a whole, then describe in turn the sounding device, the soil properties measurement circuitry, and the procedure for using the system. We shall conclude by giving the results of an actual investigation operation performed under a 50 meters depth of water in coralline sands on which a jack-up drilling barge is to be located.

SUMMARY DESCRIPTION OF THE EQUIPMENT USED

The schematic diagram of figure 1 shows the whole of the submersible system, which consists of the following basic parts :

THE LPC-TLM PRESSIO-PENETROMETER

A vibratory driving system (1) mounted on an underwater frame (2), intended to drive into the soil a string of tubes (3) which goes through the vibrohammer and is terminated at its bottom end by an assembly of measurement modules that usually consists of a penetrometric point (4) and a pressuremeter probe (5). If necessary, it is also generally possible to use a piezometer to measure the pore water pressure (6).

A sealed tank (7) encloses the control and measurement components of the various test modules.

The vibrodriving system is connected to a naval support by hydraulic hoses (8). The sealed tank (7) is itself connected by a multiconductor cable (9) to a data acquisition and processing system located at the surface.

The naval support also carries the power sources and winching devices needed for lowering the system to the bottom of the water.

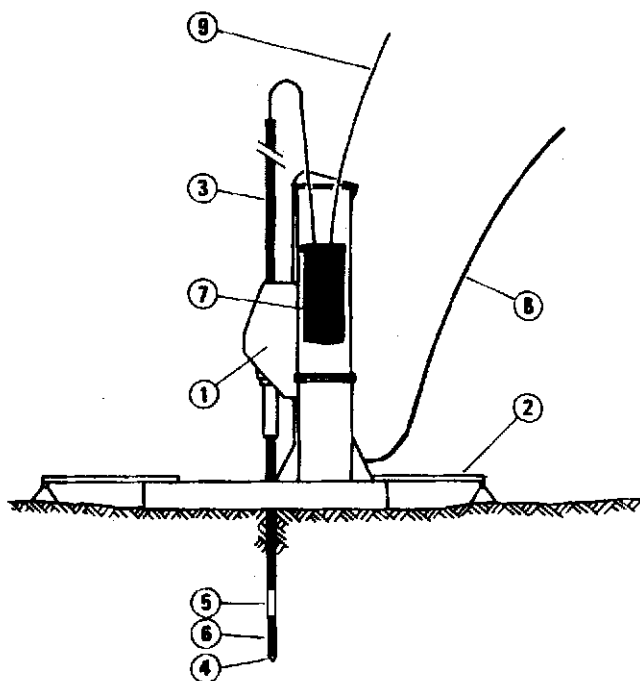


Figure 1. Schematic view of the LPC pressure-penetrometer operated from the BSMVA of the Société Techniques Louis Ménard.

THE UNDERWATER VIBRODRIVING RIG BSMVA

This machine (figure 2) was developed by Techniques Louis Ménard (T.L.M.) - reference 1.

The rig consists of three main parts : the base, the mast, and the annular vibratory hammer. The base assures firm support on the sea bottom. It holds the vertical mast on which the hammer slides ; this hammer is actuated by two 150-kN hydraulic jacks on the sides.

The characteristics of the hammer are as follows :
(reference 2).

- weight of the vibrating unit of the counterweights : 580 kg,
- speed of rotation : 800 to 1 200 rpm,
- hammering frequency adjustable from 10 to 20 Hz.

The driving forces created by the counterweights rotation are transmitted by an anvil coupled to an annular hydraulic chuck that is clamped round the string of tubes carrying the measurement modules.

The string of tubes is driven into the soil in a series of 1.25 meter passes. When this distance has been covered, the annular chuck is released and the hammer is raised by the two hydraulic jacks on the sides. When the hammer has reached the top position, the annular chuck is actuated to clamp the string of tubes again and the vibratory driving operation is resumed.

During the driving operation, the stress transmitted to the soil by the frame is on the order of 30 kPa. When the string of tubes is pulled up at the end of drilling, this stress may reach as much as about 100 kPa. Hydraulic power is transmitted to the vibratory hammer by a bundle of high-pressure hydraulic hoses (20 MPa) connected to the 130-Hp hydraulic generating unit on the naval support. The units for the control and monitoring of the various functions of the BSMVA are placed on this generating unit.

We should point out two further possibilities provided by this equipment :

- jacking the string of tubes up to a maximum reaction of the order of 30 kN (the weight of the submersed system is 50 kN),
- either jacking-up or jarring up the string of tubes.

THE LPC-TLM PRESSIO-PENETROMETER

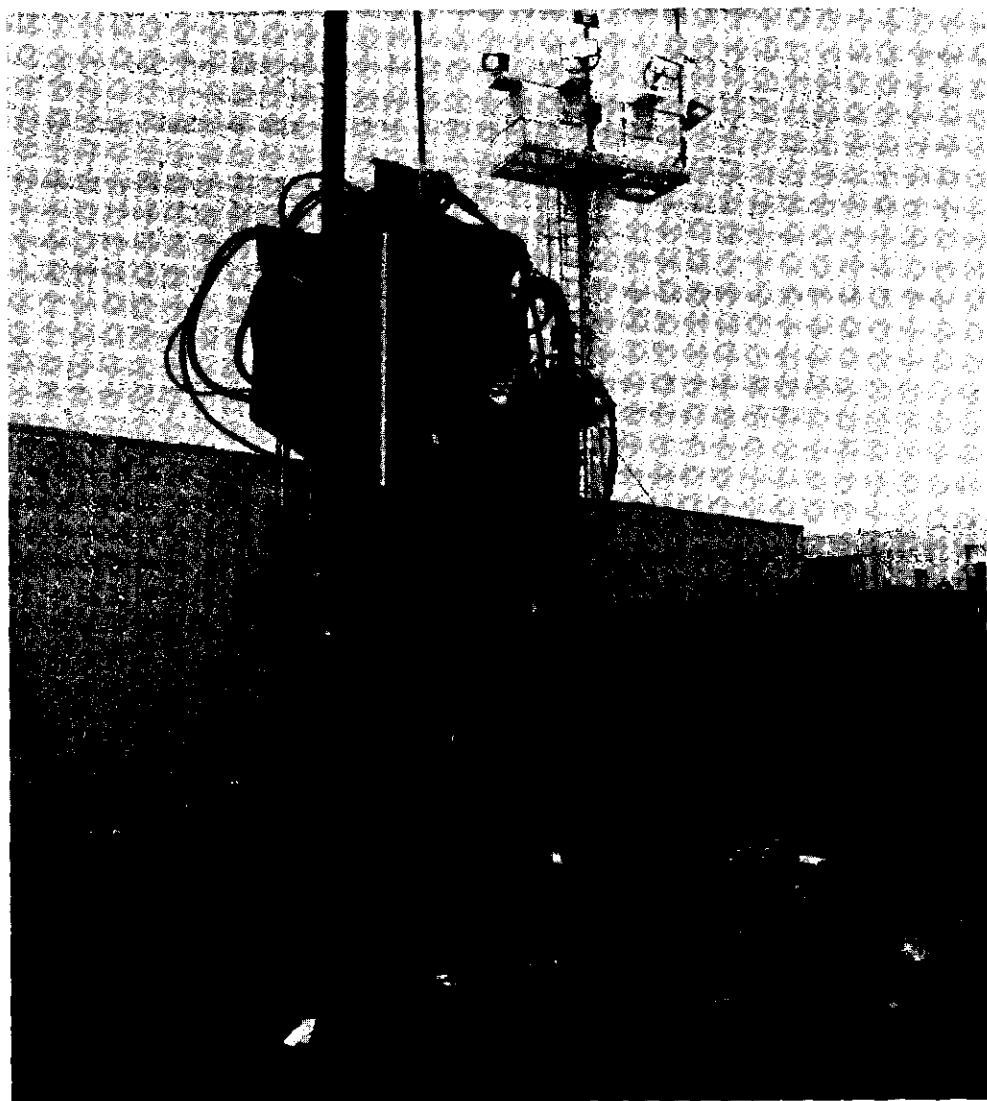


Figure 2. View of the BSMVA developed by the Société Techniques Louis Ménéard (photo CEA-TME).

THE LPC-TLM PRESSIO-PENETROMETER

THE LPC PRESSURE-PENETROMETER

This is in three parts :

- the actual measurement modules, attached to the end of the string of drilling tubes,
- a "pressure-volume controller", placed inside the sealed tank of the submersed rig,
- electronic automatic-control and data-processing circuitry, placed on the floating support.

A. The measurement modules

These modules are 89 mm in diameter. They are assembled as the need arises, then attached to the end of the string of tubes, 114 mm in diameter.

- The penetrometer cone :

This is a conical point with a strain-gauge balance (full-bridge layout). The balance is designed to withstand a maximum load of 300 kN. The complete measurement system is capable of determining the penetration resistance of the soil to within ± 50 kPa.

Point resistance measurements may be made either with static penetration or with dynamic penetration under the effect of the impacts of the vibratory hammer.

- The pressuremeter module :

This module consists of a single cell (there is no guard cell) having a length ratio of four. The covering of the cell is specially designed to withstand the operation of forced insertion into the soil by vibratory driving. It is also designed to withstand without damage a doubling of the cell volume during the expansion test.

- The piezometer module :

This is a constant-volume piezometer consisting of a filtering area of approximately 10 cm^2 . This module may be placed either directly behind the point of the penetrometer or above the pressuremeter cell. Pore water pressure measurements are made either during or after penetration (e.g., at the same time as the pressuremeter tests).

THE LPC-TLM PRESSIO-PENETROMETER

Remark : The depth of penetration is determined by measurement of the vertical displacement of the seat, by means of a rotary potentiometric sensor attached to the frame of the BSMVA. The depth reached by the point with respect to the plane of support of the frame can by this means be measured to within about ± 5 cm.

Figure 3 shows the measurement modules of the pressure- penetrometer in place on the BSMVA. These are, from bottom to top :

- the conical point of the penetrometer module,
- the filtering portion of the piezometer,
- the pressuremeter probe (here shown undergoing expansion for calibration of the membrane).

B. The pressure-volume controller (CPV)

This is the name given to the set of equipment placed inside the sealed tank attached to the frame of the BSMVA. This equipment is connected by various hoses and cables to the measurement modules and to the recording instruments on the floating support.

This equipment includes in particular the pump used to expand the pressuremeter probe. This pump consists of a cylinder and piston assembly driven by a small electric motor. This system makes it possible to apply a constant deformation rate to the pressuremeter probe (reference 3) during both expansion and deflation. The volume injected into the pressuremeter probe is determined by means of electrical comparators that measure the movement of the piston inside the cylinder. The corresponding pressure is measured by an electrical sensor.

The main characteristics of this pump are as follows :

- maximum allowable pressure 2,500 kPa,
- usable injection volume 2,500 cc, giving a maximum volumetric expansion of the pressuremeter probe of 113 %,
- precision of volume measurement ± 4 cc, or ± 0.16 % in volumetric expansion.

The flow rate may be adjusted at will within certain limits. Normally, a flow rate of 240 cc per minute is used ; this makes it possible to reach conventional limiting pressure p_L (doubling of the volume of the initial cavity) in about ten minutes.

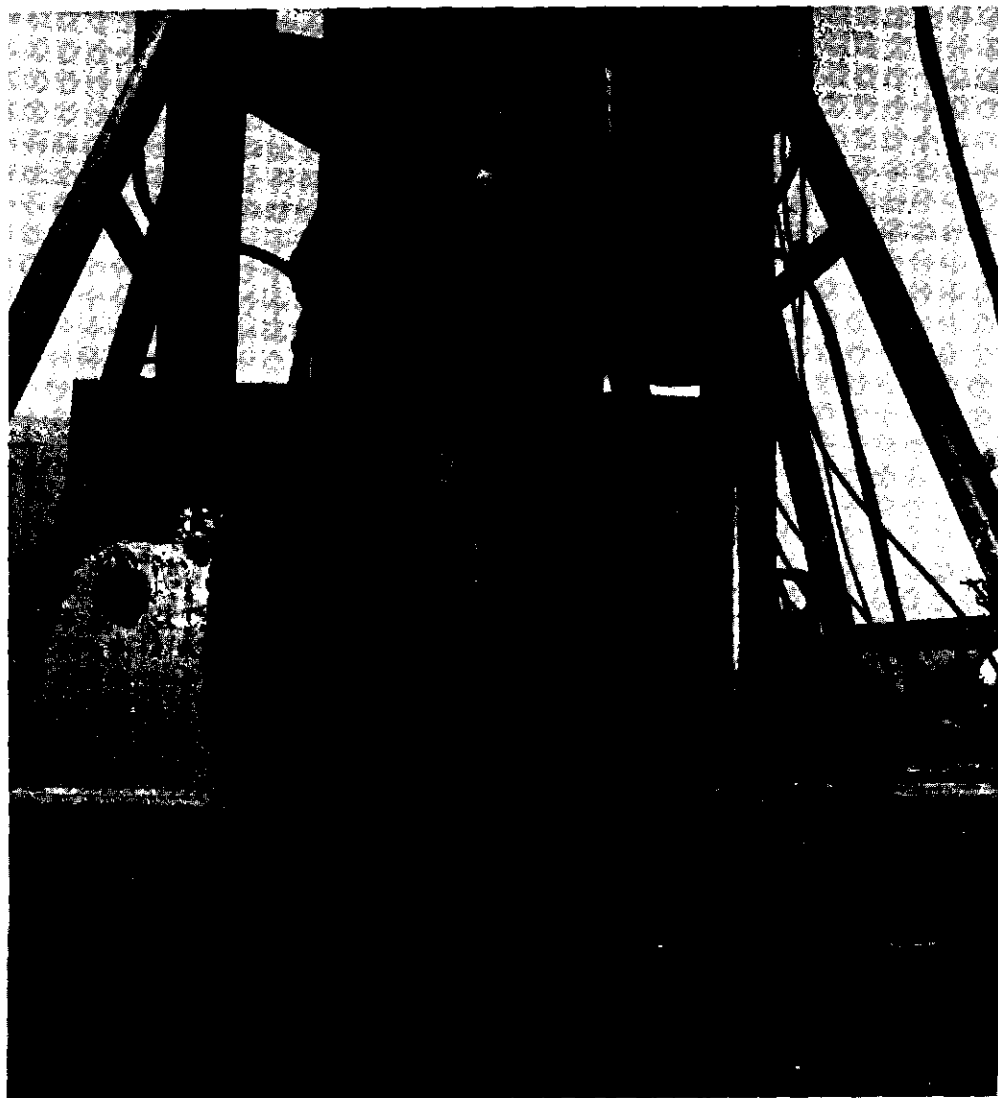


Figure 3. The measurement modules of the LPC pressure-penetrometer in place on the BSMVA (photo CEA-TME).

THE LPC-TLM PRESSIO-PENETROMETER

C. The electronic automatic-control and data-processing system.

This system is organized around a programmable calculator, making it possible to run off the measurement sequences automatically, record the data, and process them in real and/or deferred time. Figure 4 is a block diagram of the system. The "A" symbols represent the stabilized power supplies of the various pressure and movement sensors.

The presence of two voltmeters will be noted :

- the first is used only for measurements of the penetrometer balance, measurements having a dynamic character that calls for special capabilities,
- the second is used for all the other measurement channels.

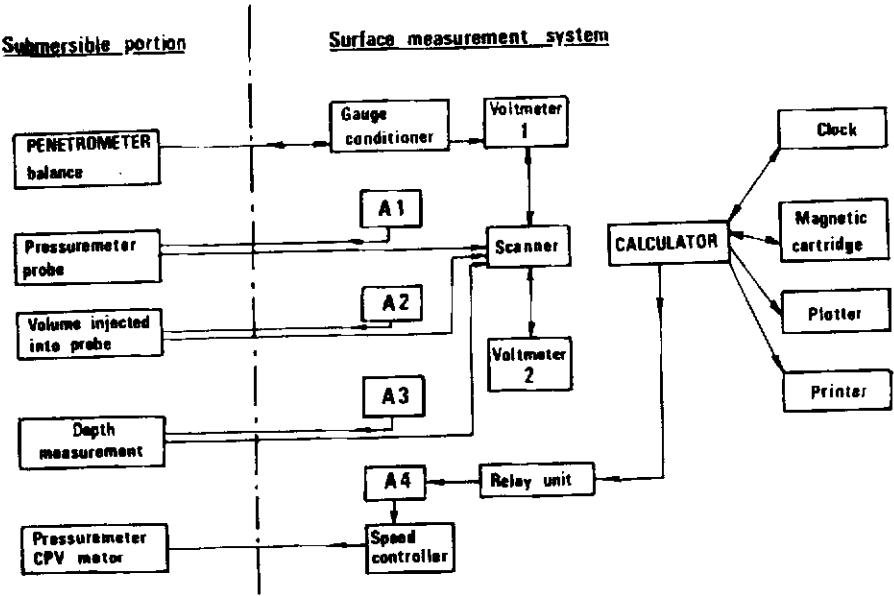


Figure 4. Block diagram of the automatic control and data processing system.

OPERATING PROCEDURE

Here we describe the normal operating procedure, which may have to be modified in certain respects according to environmental conditions.

The operation begins with the assembly of the parts of the string of tubes, to the end of which are attached the measurement modules chosen, generally the penetrometer and the pressuremeter probe.

The control and measurement cables are run inside the string of tubes. Thanks to a suitable system of connections, it is possible to remove the modules from the string of tubes without having to repeat the calibrations and other preparations carried out on these modules.

With the seat of the BSMVA in the top position, the tubes with the measurement modules are placed in the annular jack of the vibrating hammer.

The various connections, between the measurement modules and the sealed tank and between the sealed tank and the data acquisition and processing system, are then made. This is followed by the usual saturation and calibration operations.

The system may then be lowered into the water using a suitable winch. Once the equipment has been set in place on the bottom, the proper operation of the measurement system is checked before proceeding to the vibrodriving, performed in the sequence described earlier.

The penetration parameters are traced in real time on the plotter of the measurement system, giving the (q_{cd}, z) diagram. This serves both to measure the "hardness" of the soil and to assist in the judicious choice of levels at which it may be useful to conduct expansion tests.

The operator controls the starting of the expansion tests, but the test then proceeds automatically and is interpreted in real time by the plotting of the corrected curve in the form of a (p, v) diagram, with immediate calculation of modulus E_M and of limit pressure p_L .

Deflation follows and penetration is resumed. When the profile has been completed, the string of tubes is pulled from the soil and brought back to its initial position.

The usual length of the set of tubes and modules is 15 meters overall, sufficient for 12 meters penetration into the soil.

THE LPC-TLM PRESSIO-PENETROMETER

In certain special cases, longer strings of tubes may be used.

In the case of closely-spaced borings, there is no need to raise the system to the surface for readjustment or checking, since this can be done remotely by the calculator.

AN EXAMPLE OF APPLICATION TO OFF-SHORE RECONNAISSANCE

We present an example of the use of the system for soil reconnaissance under 50 meters of water with a view to investigate the stability of a jack up barge.

The site is highly heterogeneous. It includes loose to moderately compact fine sand with coral debris equivalent to coarse sands. Locally, there may be indurated seams and branching coral.

Figure 5 is a penetration profile giving the dynamic point resistance q_{cd} versus depth of penetration z . The penetration curve clearly shows the interruptions of driving made necessary by the 1.25-m strokes of the BSMVA.

The chart also shows the levels at which pressuremeter tests were made, identified by small triangles.

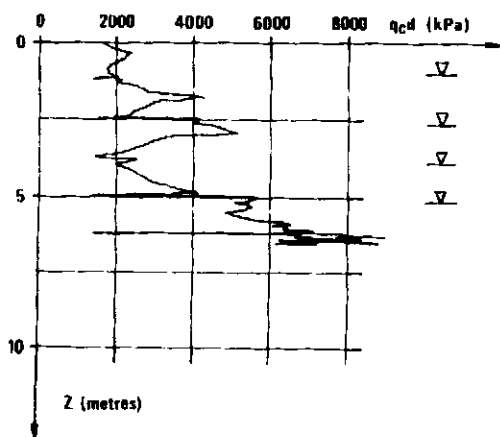


Figure 5. Penetration profile giving dynamic resistance q_{cd} versus penetration depth z .

THE LPC-TLM PRESSIO-PENETROMETER

Figure 6 shows pressuremeter curve obtained during the test at a depth $z = 4.25$ meters.

It can be seen that, even though the pressuremeter was forced into the soil, the expansion curve shows an initial "recompaction" phase. This phenomenon is explained by the nature of the soil itself.

At the end of this series of tests, application of the standard pressuremeter design rules made it possible to give an immediate prediction of the behaviour of the soil, a prediction that was borne out when the barge was set in place.

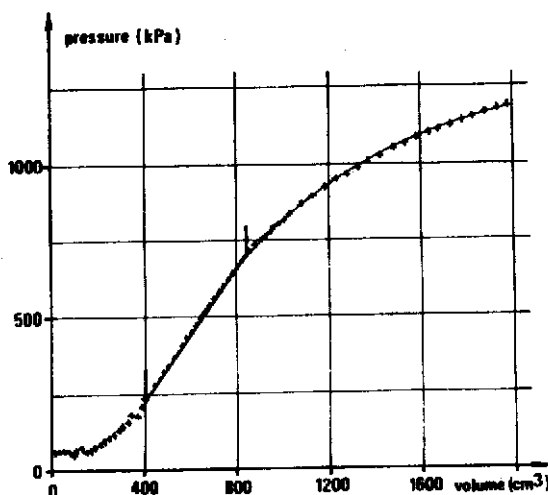


Figure 6. Test curve giving pressure p versus volumetric deformation v of the measurement cell.

CONCLUSIONS

The system presented, which has already demonstrated its effectiveness in off-shore soil investigation, has several interesting original features :

- The BSMVA developed by the Société Techniques Louis Ménéard is distinguished by the capability of driving a continuous string of tubes in a single 15 meters length, using a compact system of moderate weight. Within the limits stated earlier, it may be used for static or dynamic driving of the string of tubes.

THE LPC-TLM PRESSIO-PENETROMETER

- The LPC pressure-penetrometer is capable of providing a great variety of precise measurements under conditions demanding robustness. The fact that the pressuremeter is expanded by a volumetric pump on the sea bottom, at the minimum distance from the measurement cell, eliminates technological problems. This fact, together with the automation of the expansion procedure, means that cyclic loadings in off-shore investigation are now very simple to perform and yield very reliable determinations of soil characteristics.

Real-time data processing is also a major advantage ; it means that certain tests may be repeated if necessary and, most important of all, makes it possible to give the engineer an immediate opinion concerning the properties of the soil.

Improvements are now being investigated with a view both to increasing the penetration power (increasing the driving power of the BSMVA) and to adding further measurement modules, in particular a permeameter and a skin friction sleeve.

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SELF-BORING PRESSUREMETER FOR SHALLOW DEEP WATER

ABSTRACT

This article describes apparatus that may be used for pressiometric tests under depths of water of as much as 40 m. The system consists of :

- a four-legged hydraulic platform set on the bottom on adjustable legs, equipped with a driving device, drilling rods, and an arm for feeding these drilling pipes ;
- a self-boring pressiometric probe ;
- a control and monitoring unit grouping the controls and indicators, on a floating support (ship or pontoon) ;
- two multi-line hoses with their take-up reels, interconnecting the control unit and the platform.

A first operational test of this equipment, in connection with a planned harbour work, is described. The results of the measurements of soil properties so obtained are interpreted and commented on.

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SELF-BORING PRESSUREMETER FOR SHALLOW DEEP WATER

The expansion of existing major ports and the creation of port complexes abroad require increasingly sophisticated techniques of design and construction. The increasing scarcity of the most suitable sites makes it necessary to work on weaker soils and at greater depths.

From a geotechnical viewpoint, port works have many different aspects : retaining walls for wharfs, foundations for dikes, landing stages, dolphins, etc. Such operations as dredging and the laying of underwater piping and cables are also a direct concern of the geotechnician.

The existing means for soil surveys at maritime sites are only transpositions of conventional on-shore methods. In general, the measurement and sampling devices used are based on pontoons or self-supporting platforms which impose certain restrictions : a platform standing on legs resting on the bottom may be used at an exposed site, but is economical only in shallow water ; as for the pontoon, the rigid coupling between it and the survey device penetrating the soil imposes very narrow limits on the permissible angular and linear movements of the pontoon, restricting its use in offshore work to especially favourable conditions.

However, in the case both of sampling and of direct in situ measurements, determination of the mechanical properties of the soils requires minimum remolding of the samples taken or of the zones tested.

Thinking about these special problems, related to the aleatory character of the weather and to the action of wind, currents, and waves on the equipment, led to the idea of developing a special instrument for offshore geotechnical exploration.

It also became apparent that, in most cases, the aim was to determine the behaviour of the soil under load, and that the pressiometric tests was best for this purpose. However, the practical difficulties of conducting a conventional pressiometric test offshore (e.g., inserting the measurement probe into a bore hole prepared in advance), together with the aim of minimum remolding of the soil to assure the reliability of the measurement, led at the outset to the designing of apparatus capable of employing a self-boring pressiometric probe.

The Direction des Ports Maritimes et des Voies Navigables accordingly asked the Laboratoire Central des Ponts et Chaussées and the Laboratoire of Saint-Brieuc and the Centre d'Etudes et de Construction de Prototypes of Rouen to design and develop this apparatus.

The development specifications included, in particular, the following points :

- the apparatus should be suitable for soil explorations in

SELF-BORING PRESSUREMETER FOR SHALLOW DEEP WATER

unsheltered zones (open sea, roadsteads, estuaries, etc.) ;

- in operation, the maximum wave height would be : 2 m
and the maximum current : 1 m/s
- the maximum depth of water would be : 40 m
- the depth of soil explored would be : 30 m

The solution chosen consists of a submersible frame holding the drilling pipe and the self-boring pressuremeter, connected to a floating support from which the main operations are remotely controlled. *This may be a simple pontoon, or a tender ; it must have adequate handling gear and be capable of holding station by means of suitable anchors, but the tolerances are rather large. A diver is needed to insert the drilling pipe, and to place and remove the frame.*

From the geotechnical viewpoint, the capabilities of the apparatus are for the moment those of the self-boring pressuremeter : it can operate in any type of soil that does not contain large elements capable of blocking the advance of the probe.

Construction of the apparatus was completed in 1980. It has since been tested and perfected during a series of trials made on shore, then at maritime sites at Rouen, Le Havre, and Dunkirk.

1. DESCRIPTION OF THE EQUIPMENT

The complete equipment of the offshore self-boring pressuremeter consists of the following elements (fig. 1, 2, 3) :

- the PAF 76 self-boring pressiometric probe and its pressure-volume monitor (CPV) ;
- the submersible hydraulic drilling platform (PHYF1) (Plate-forme HYdraulique pour Forages Immergés) ;
- a hydraulic power unit with its engine ;
- two multi-line hoses and their take-up reels ;
- a unit grouping the controls and indicators of the system, and possibly a measurement acquisition and processing system.

A. Description of the elements

1. THE PAF 76 PRESSIOMETRIC SONDE AND ITS CPV

These elements are the same as those developed for onshore surveys.

SELF-BORING PRESSUREMETER FOR SHALLOW DEEP WATER

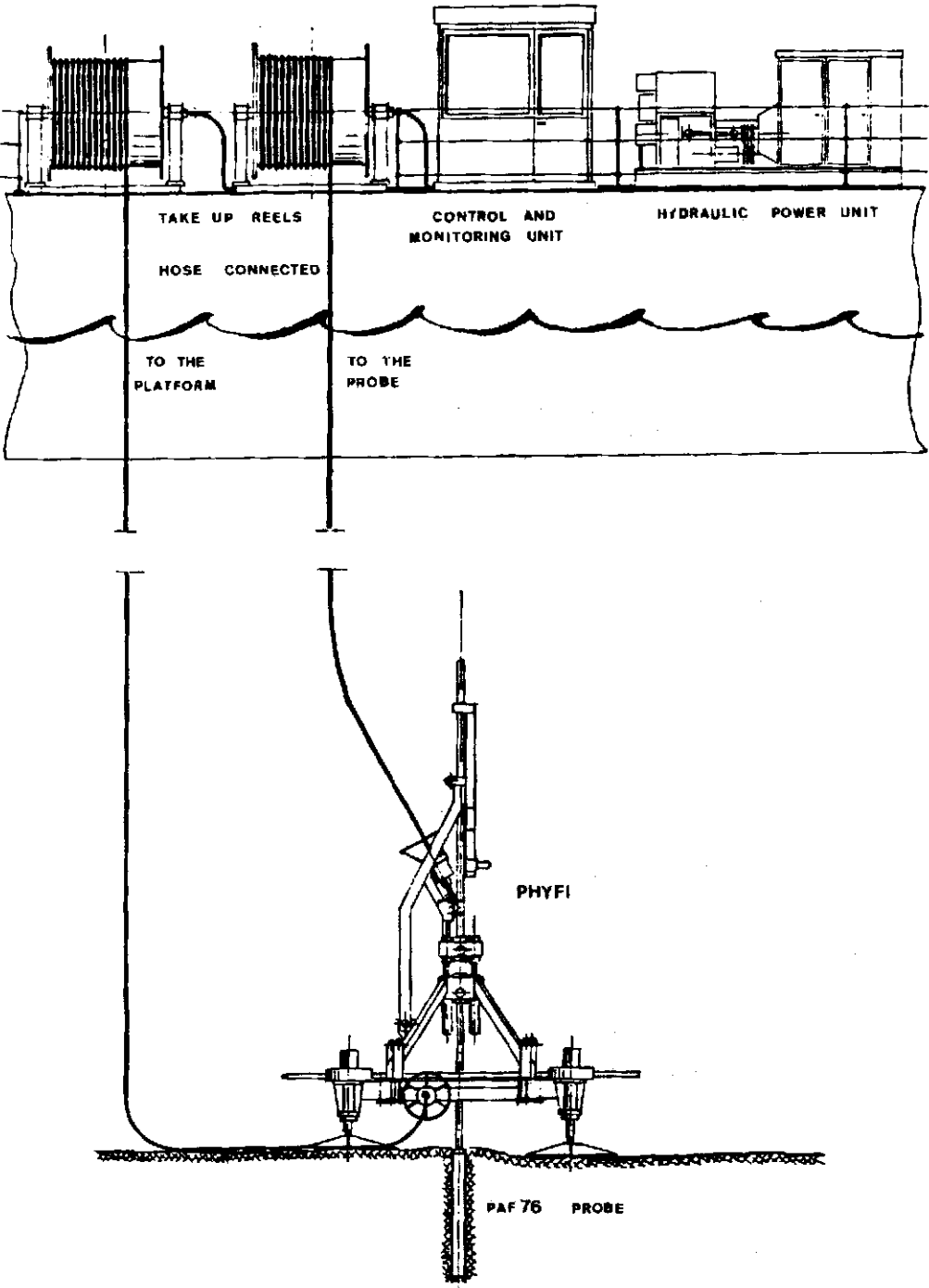


Figure 1- Platform, probe, and equipment located on the naval base.

SELF-BORING PRESSUREMETER FOR SHALLOW DEEP WATER

However, they have been treated against corrosion by sea water, and the pressure is measured by a transducer built into the probe.

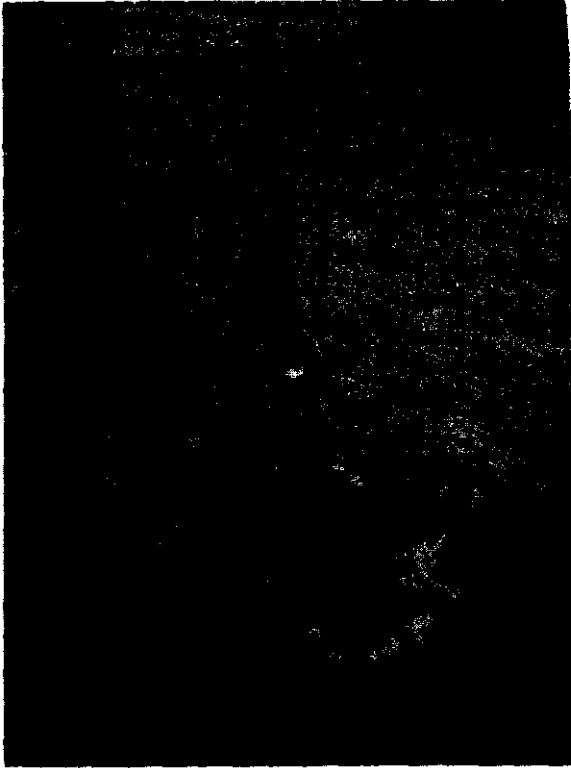


Figure 2 - Platform before submersion.

2. THE PHYFI PLATFORM (fig. 4)

PHYFI is set on four lift legs and is equipped with a step-by-step driving device and a drilling-rod feed arm.

a. The drilling pipe sections

Designed in a U shape, these are equipped internally with clips serving to snap on the hose containing the sonde operating lines. The

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rods have flats on their sides at regular intervals, forming a lock. One step is equivalent to 50 cm of drilling. The platform holds eight 4-metre rods on a rack. A handling arm operated by a hydraulic cylinder is used to feed them in a vertical position and connect them to the drilling string.

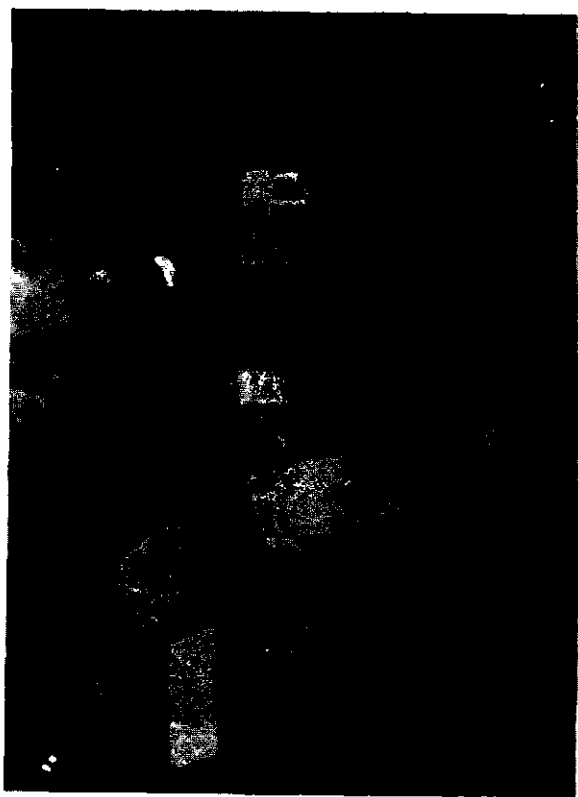


Figure 3 - Equipment on the naval base : hoses with take-up reels, hydraulic power unit, CPV.

b. The driving device

Of the step-by-step type, this includes two clamping heads, one fixed and the other mobile, equipped with hydraulic bolts, and two cylinders providing a maximum driving force of 50 kN and a maximum pulling force of 100 kN.

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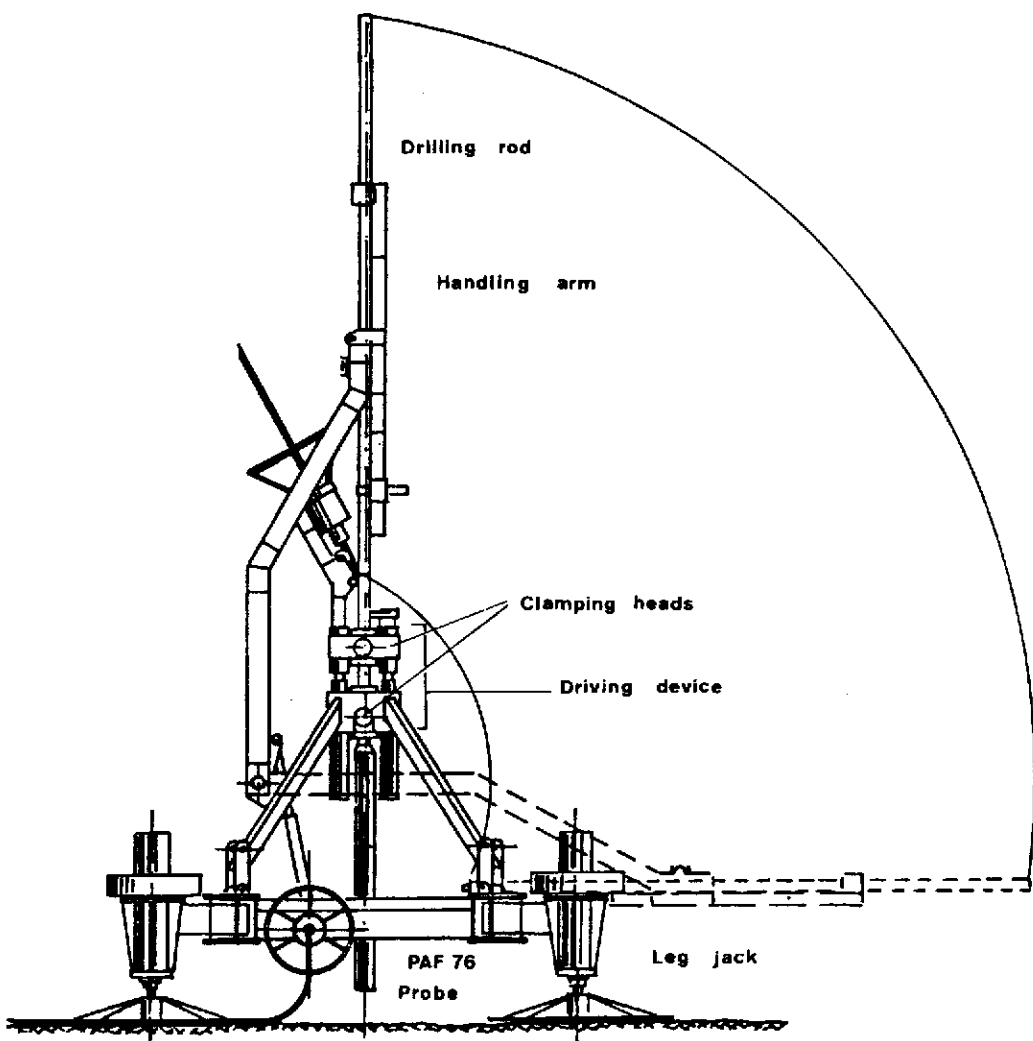


Figure 4 - PHYFI Platform.

c. The chassis and its equipment

This stands on four retractable legs each having a contact area of 1 m^2 , for a total weight of 3800 kg, including the drilling rods (this weight could be increased to 5000 kg if required), and has overall dimensions of 4 m x 4 m x 3 m.

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The platform is equipped with a hydraulic distribution system :

- with electrical controls for manoeuvring the driving device ;
- and manual controls for operating the feed arm and the leg jacks.

A two-direction inclinometric indicator indicates the inclination of the platform ; this information is repeated on the surface.

3. THE HYDRAULIC POWER UNIT

This is equipped with an 85-kW diesel engine and includes the following elements :

- one hydraulic pump supplying the PAF 76 probe ;
- one hydraulic pump for the units of the PHYFI platform ;
- two hydraulic pumps for the hose take-up reels ;
- one 350-l oil reservoir ;
- one sea water pump ;
- one 24 V DC generator.

4. THE HOSES AND THEIR TAKE-UP REELS

All interconnections are grouped into two multi-line hoses stowed on two take-up reels with a constant-tension control device :

- one of the hoses, 100 m long, connected to the PHYFI platform, contains two "high-pressure" hydraulic lines and three electric cables ;
- the other hose, 120 m long, connected to the PAF 76 probe, contains three "high-pressure" hydraulic lines, two "low-pressure" hydraulic lines, and two electric cables.

5. THE CONTROL AND MONITORING UNIT

This groups the control and monitoring units of the system : the sequential automaton, the survey data, the CPV associated with the PAF 76 sonde. Although not essential, it is more effective to have a data-acquisition and process-control system in conjunction with the CPV. This makes it possible to acquire the measurements with greater accuracy and to obtain the pressiometric curve uncorrected immediately after each test.

SELF-BORING PRESSUREMETER FOR SHALLOW DEEP WATER

B. Setting up

The setting up of the offshore self-boring pressuremeter requires a floating support with an adequate anchoring system and a lifting device for handling the PHYFI platform. A pontoon or a ship will be used as appropriate (sheltered site or exposed site). A team of divers and a team of technicians (pilots and soil engineers) are also required.

1. PLACEMENT OF PLATFORM

After the calibration operations that are indispensable before each survey, the PHYFI platform is lowered from the support vessel to the bottom. It is then stabilized in a horizontal position by a diver, using the hydraulic jacks of the legs and guided by the inclinometric indicator. When the platform is securely in place, the diver feeds the first drilling rod using the handling arm described earlier, then surfaces. The system is ready to bore 4 metres ; this operation takes about an hour.

2. BORING AND THE PRESSIOMETRIC TEST

When the boring cycle is started, the sequential automaton orders the release of the retaining bolts, the driving of the probe for one 50-cm step, and the return of the drive head for the next step. In general, one pressiometric test is made every metre (or two cycles).

During the boring period, the following information is available :

- the thrust on the probe ;
- the boring depth ;
- the disaggregation pressure ;
- the water pressure for raising sediments ;
- the inclination of the platform.

The operation consisting of one metre of boring and a pressiometric test lasts about an hour.

Every four metres, a diver goes down to feed the next drilling rod (this takes roughly a quarter to half an hour). When the final depth is reached, or refusal, the probe is brought back up using the same procedure in reverse (this takes about half an hour per length of rod).

The elements of the offshore self-boring pressuremeter have been designed so that they can be transported without special restrictions.

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The chassis of the PHYFI platform can be folded to within road transport size limits. A standard platform, 14 metres long, is large enough to carry all of the equipment.

II. EXAMPLE OF USE OF THE "PHYFI" EQUIPMENT FOR A PORT PROJECT

We were led to make operational use of the offshore platform for the first time in connection with the designing of the slurry trench wall of the bulk terminal of the port of Dunkirk-West.

A. The problem posed

Figure 5 gives a schematic section of the structure. It shows that the wall is set in Flanders clay to a depth of about 6.50 m. Investigation of the stability of this wall called for a knowledge of the characteristics of the clay, both in its natural loading conditions (behind the structure) and after unloading by dredging (in front of the structure) from an initial depth of - 4.00 (C.M. : with respect to sea level) to a final depth of - 23.00 (C.M.). The characteristics of the clay behind the structure were determined by a pressiometric survey made with the PAF 76 probe and a conventional drilling machine (profile B). To determine the mechanical properties of the unloaded clay

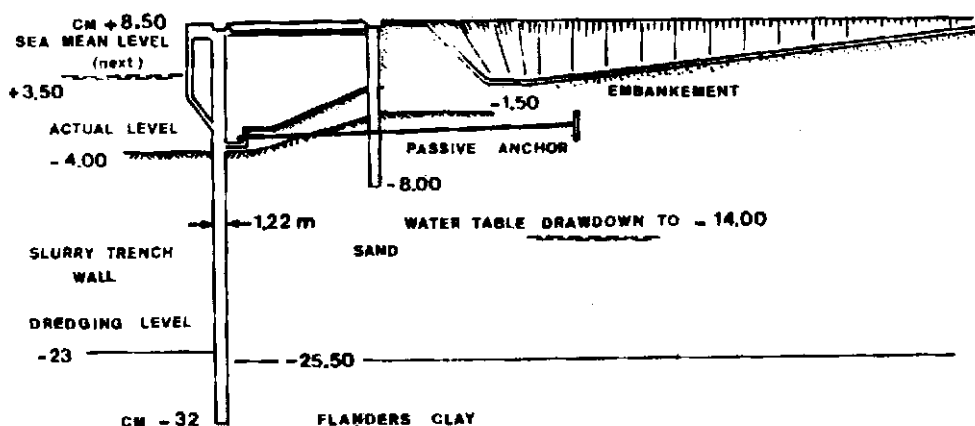


Figure 5 - Dunkirk slurry trench wall.

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that would subsist in front of the structure another pressiometric survey was made with the PAF 76 sonde, but this time from the PHYFI platform, in the outer (oil) harbor of Dunkirk-West (profile A). At this location, where about 15 m of sand had been dredged about five years earlier, the Flanders clay was in conditions substantially the same as those that would prevail in front of the wall after dredging. This latter test is presented below, and the results obtained in the clay of profile A, in which it is unloaded, are compared to those of profile B, in which it is kept in its initial loading condition.

B. Conditions of execution of the offshore survey

At the location of survey A, the depth of water is of the order of 30 m, with a tide range of about 3.50 m. The equipment was lowered from a pontoon measuring 15 x 31 m. The survey, begun towards 11 a.m., was completed towards 9 p.m. : ten hours' work for a 10-m bore hole with a pressiometric test every metre. For surface measurements, a team of three is sufficient for a survey made in one day. For a deeper survey, or for successive bore holes made without interruption, it would have been necessary to have two teams working alternating eight-hour periods, for example. The underwater work at the platform is done by one diver, but in view of the rest periods necessary between two dives to a depth on the order of 30 m, a team of three divers assisted by a diving master is necessary, whether for a single survey or for a series of bore holes.

The characteristics of the boring and of the test were as follows :

- boring speed 0.25 m per minute ;
- measurement cell volumetric deformation rate approx. 3 % per minute ;
- cell relaxation time before each test deliberately limited to 30 minutes so as to shorten the overall duration of the survey.

C. Results obtained

1. EXAMPLES OF THE EXPANSION AND SHEAR CURVES OF PROFILE A

The expansion curve (p , $\Delta V/V_0$) and shear curve (τ , $\Delta V/V_0$) are given for the depths of 1.30 and 8.30 m (corresponding to depths - 26,30 and - 33,30 C.M.) on the figure 6 (p is the pressure in the cell, $\Delta V/V_0$ its variation of relative volume, and τ the shear stress). The shape of these curves is characteristic of a clay, especially that of the shear curve, which has a maximum $\tau_{\max}$ corresponding to undrained

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cohesion c_u , the solicitation in this case being of the undrained type.

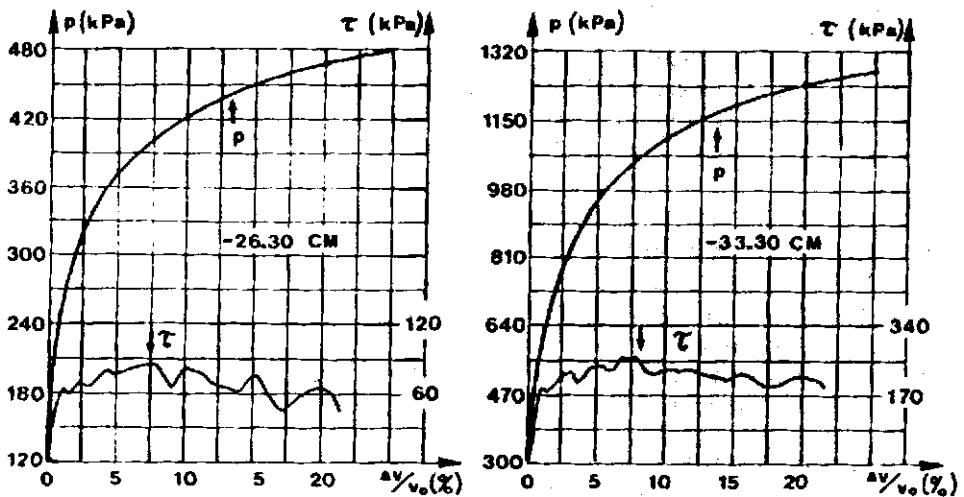


Figure 6 - Pressuremeter curve and shear curve (Dunkirk site).

2. SOIL IDENTIFICATION PARAMETER β

This is given by :

$$\beta = \frac{p_{20} - p_5}{p_{20} - p_0} \times 100$$

(p_α is the pressure in the cell at α % deformation). The mean values of β , ($\bar{\beta}$), found for the two profiles, A and B, are the following :

A : $\bar{\beta} = 28.4$ (extreme values : 26 - 31) ;

B : $\bar{\beta} = 26.6$ (extreme values : 23 - 32) ;

A + B : $\bar{\beta} = 27.4$ (extreme values : 23 - 32).

The dispersion is small, and the values found are characteristic of a clay (at the limit between sensitive and non-sensitive clays).

3. EFFECTIVE HORIZONTAL PRESSURE p'_0 AND EARTH PRESSURE COEFFICIENT K_0

Figures 7 and 8 show the variations in p'_0 and K_0 in the layer of

SELF-BORING PRESSUREMETER FOR SHALLOW DEEP WATER

clay at profile A (unloaded clay) and at profile B (clay overlaid by about 21 m of sandy deposits). It can be seen that, in a large part of the layer of clay (depths - 27 to - 34 C.M.), pressures p'_0 are maintained after unloading. Between these same depths, the mean values K_0 of the earth pressure coefficient ($K_0 = p'_0/\sigma'_v$, where σ'_v is the actual effective vertical pressure) are :

- $\overline{K_0} = 0.84$ for the non-unloaded clay ;

- $\overline{K_0} = 3.67$ for the unloaded clay.

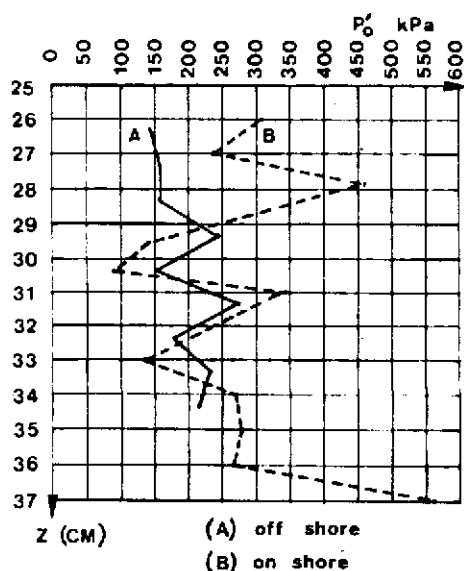


Figure 7 - Initial pressure p'_0 .

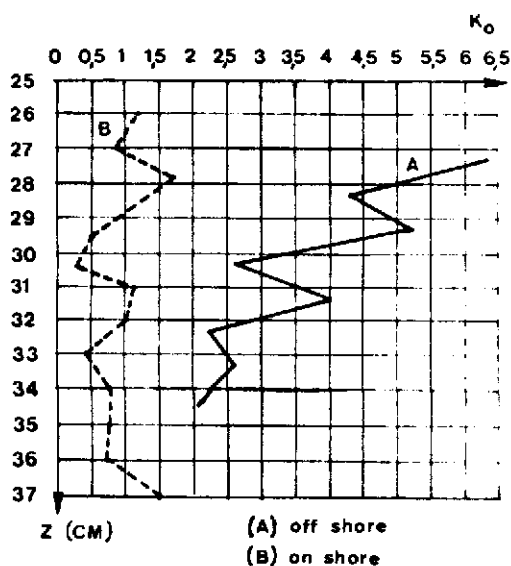


Figure 8 - K_0 factor.

This information is of interest in defining the initial condition of the soil, especially for the calculations of retaining walls using the concept of reaction modulus. It should be noted that p'_0 could be determined with greater accuracy by leaving a longer cell relaxation time after each driving period.

4. ULTIMATE PRESSURE AND SHEAR STRENGTH OF THE SOIL

The pressure at 20 % deformation ($p_{20}^* = p_{20} - p_0$) is used in many foundation design problems :

- the bearing capacity of footings or piles, which is proportional to p_{20}^* ;

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- the lateral reaction of piles on the soil, for which the ultimate pressure is usually taken to be equal to half or three-fourths of p_{20}^* .

It is therefore interesting to compare profiles A and B with respect to (fig. 9) :

- pressure p_{20}^* ;
- and shear strength $\tau_{\max}$, derived from the elementary shear curves (and corresponding, in the present case of a clay, to undrained cohesion c_u).

It is found in this way that the unloading of the clay by dredging causes a large drop, averaging about 40 %, in both p_{20}^* and $\tau_{\max}$, since in terms of the mean values between depths - 27 and - 34 C.M. we obtain :

$$\frac{\tau_{\max} (A)}{\tau_{\max} (B)} = 0.61 \qquad \frac{p_{20}^* (A)}{p_{20}^* (B)} = 0.63$$

5. SOIL STRAIN MODULUS

The moduli secant to the expansion curve ($p, \Delta V/V_0$) at 2 % and 5 % strain, G_{p_2} and G_{p_5} , are used for the following problems :

- calculation of the settlement of foundation footings ;
- calculation of horizontally-loaded piles ;
- the design of retaining walls (slurry trench walls, sheet piling, etc.).

The differences between these moduli in the layer of clay, for profiles A and B, are shown in figure 10.

Here again, if we compare the mean values of these moduli between the depths of - 27 and - 34 C.M., we obtain :

$$\frac{G_{p_2} (A)}{G_{p_2} (B)} = 0.62 \qquad \frac{G_{p_5} (A)}{G_{p_5} (B)} = 0.61$$

This shows a drop of about 40 % in the unloaded clay.

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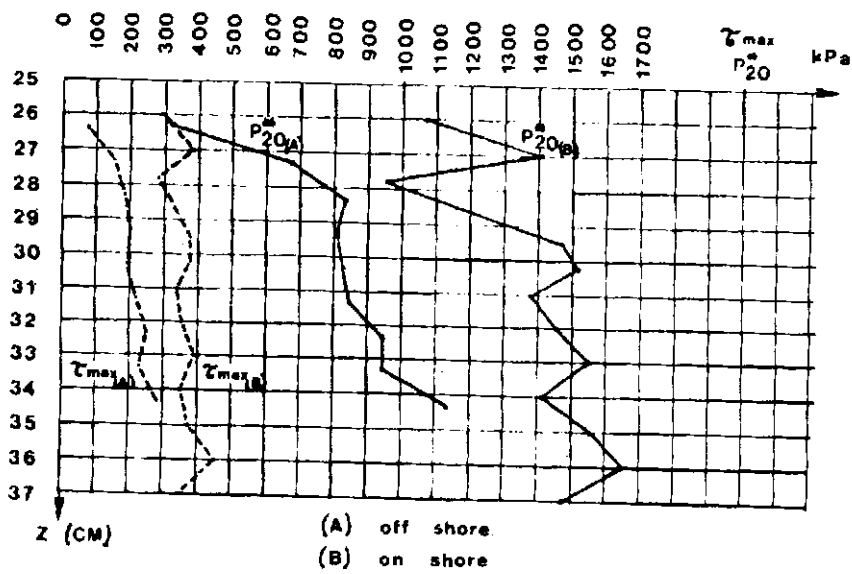


Figure 9 - Test profiles in Flanders clays : p^*_{20} and τ_{max} .

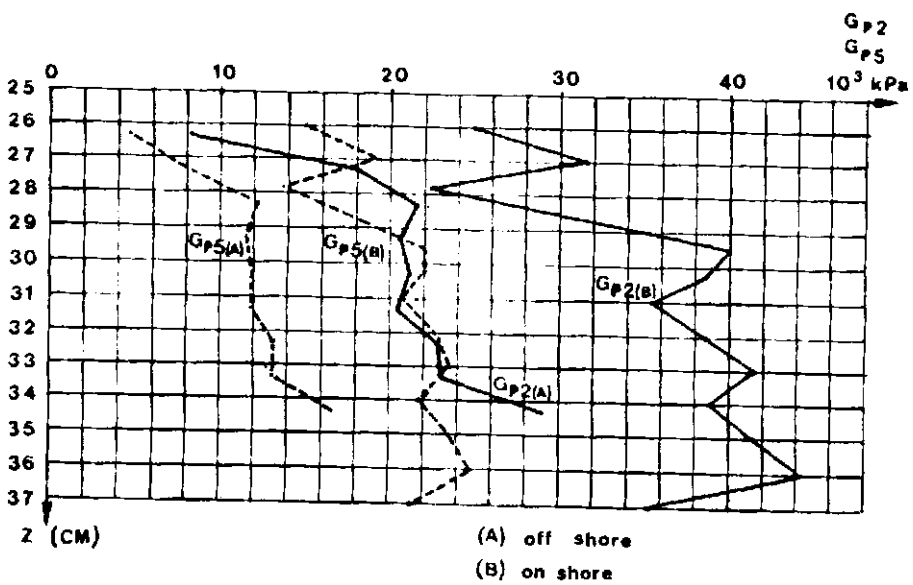


Figure 10 - Variations of the modulus at 2 % and 5 % volume strain, in the clay.

SELF-BORING PRESSUREMETER FOR SHALLOW DEEP WATER

In other words, all of the mechanical properties of the Flanders clay (p_{20}^* , $\tau_{\max}$, G_{p2} , and G_{p5}) have lost about 40 % after dredging of 15 to 20 m of the covering soils.

CONCLUSION

With the PHYFI platform, we now have equipment well suited to the use of the self-boring pressuremeter for determining the characteristics of soils under moderate depths of water (less than 40 m). The first operational use, which took place recently at Dunkirk, was conclusive. In particular, the test made served, by comparison with a test at an onshore site, to quantify the decline in the mechanical properties of Flanders clay after dredging of the covering soils. It is obvious that such tests can be of use for any structures requiring dredging of the soil in their vicinity (e.g., for slurry trench walls and sheet piling where the characteristics are not the same on both sides, even in a homogeneous soil layer). And of course this equipment will also be of use for all offshore structures in shallow or moderately deep water, in particular harbour and coastal works.

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- Baguelin, F., Bisson, D., Jezequel, J.-F., and Jolivet, P. (1982), "A self-boring pressuremeter for shallow water depth". Paper presented at the International Congress of Oceanology at Brighton.

OFFSHORE SELF-BORING PRESSUREMETER FOR DEEP WATER

ABSTRACT

The Offshore Self-Boring Pressuremeter (known as the PAM from its initials in French) has been developed by the Institut Français du Pétrole for soil reconnaissance at great water depths where current investigation techniques are insuitable.

Implementing the self-boring process (originally developed in France by the Laboratoires des Ponts et Chaussées), the offshore self-boring pressuremeter mainly consists of :

- a framework with its equipment sitting on the seabed and designed to make a pressuremeter probe 160 mm in diameter and 6 meters long penetrate into the soil (without disturbing);
- a control cabin located on the support vessel;
- an umbilical cable providing the electric power supply and data transmission.

The offshore self-boring pressuremeter is suited for the reconnaissance of fine sediments (sands, silts, clays) such as cover a large portion of the seabed, especially beyond the continental shelf.

The prototype that has been developed has the following characteristics:

- application at water depths of 300 and even 1000 meters;
- penetration of the pressuremeter probe up to 60 meters into the seabed;

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- implementation from a nonspecialized ship, such as a supply-boat, equipped with an A - frame or gantry crane for removing the device over the shipboard
- remote control and automatic data acquisition.

The offshore self-boring pressuremeter has been very thoroughly tested successively onshore, at a shallow water depth and then at great water depth, with excellent operational reliability. The advanced technology of the offshore self-boring pressuremeter, its remote-controlled operating and automatic data acquisition are such that it is the prototype of a new generation of seabed reconnaissance techniques at great water depths.

INTRODUCTION

Among soil exploration processes and techniques on the continental shelf, drilling with coring and or in-situ measurements through the drill string is incontestably the most widespread process [1] [2]. The disadvantages and limitations of this technique, however, are well known, and the need to improve information about soils before the installation of offshore structures is constantly enunciated by all specialists.

The increasing difficulty of taking representative core samples as the water depth increases has led to greater and greater recourse to in-situ measurements so as to determine actual soil properties as accurately as possible under eventual conditions of loading.

From this standpoint the pressuremeter is incontestably the most appropriate technique [3]. However, the conditions under which the probe is implemented have a considerable influence on the results obtained, thus confirming the necessity of having the probe penetrate into the soil without disturbing it. The self-boring process, originally developed by the Laboratoires des Ponts et Chaussées [4] for onshore applications, meets this objective.

The Offshore Self-Boring Pressuremeter (PAM) (Fig. 1) developed by the Institut Français du Pétrole, with the assistance of the Laboratoires des Ponts et Chaussées, implements a measuring probe from a seabed framework [5] [6] [7] [8] [9] to a maximum penetration depth into the soil of 60 meters and underneath water depths of 300 meters (and even water depths of 1000 meters). We will not go into detail about the description and operation of the probe because that is the subject of another paper at this Symposium [10]. The aim of this paper is

OFFSHORE SELF-BORING PRESSUREMETER FOR DEEP WATER

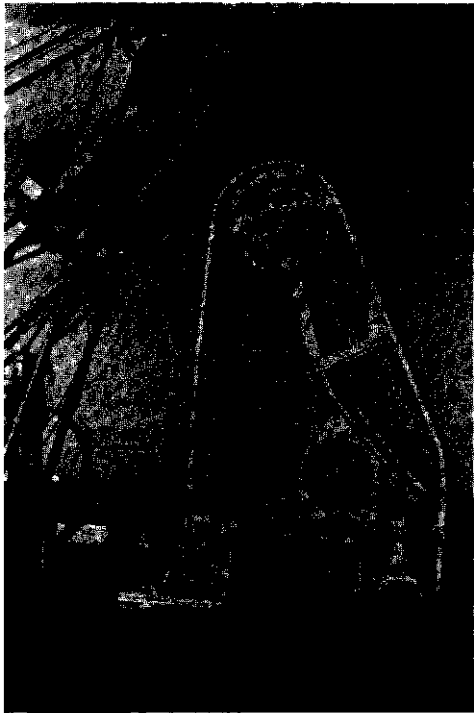


Fig. 1 - IFP Offshore Self-Boring Pressuremeter (PAM)

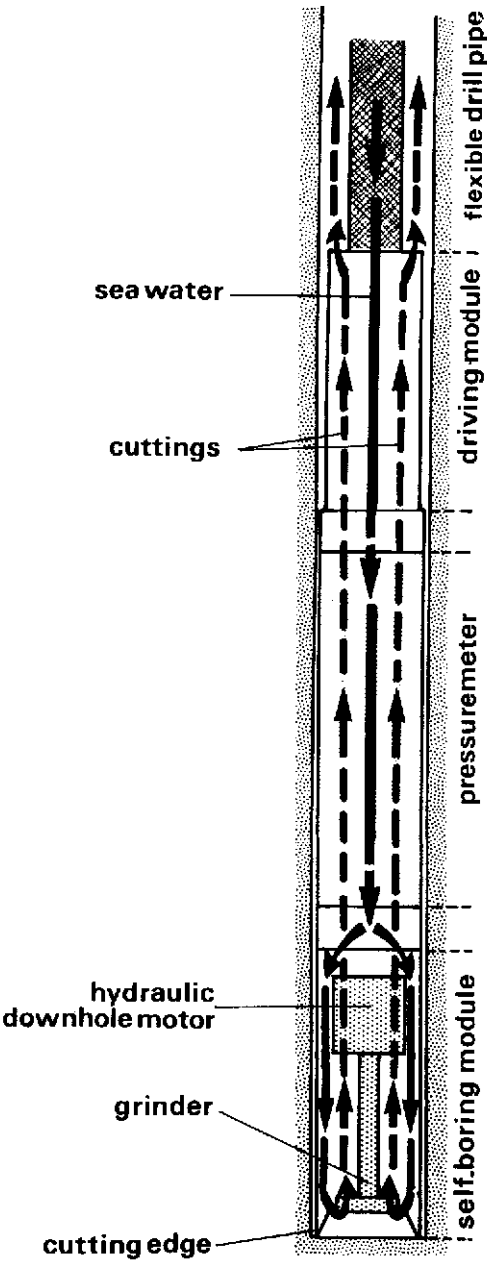


Fig 2 - Self-Boring Principle

- to review the reasons for using and the specifications of the PAM;
- to describe the main characteristics of the device and its implementation, and ;
- to give the tests results obtained with the device.

I. REASONS FOR USING AND SPECIFICATIONS OF THE PAM

The reasoning underlying the development of the offshore self-boring pressuremeter stems from the advantages of the self-boring process and of pressuremeter measurements. The specifications of the prototype device were defined on the basis of current and foreseeable needs in the reconnaissance of soils on the continental slope at water depths of several hundred meters.

1. Advantages of Self-Boring and of the Self-Boring Pressuremeter

The self-boring process (Fig.2) is a particularly reliable and well-proven technique for making in-situ measurements. The measuring probe is made to penetrate with minimum disturbance of the soil. The continuously-recorded self-boring logs give a good indication of the nature, properties and heterogeneities of the formations crossed through. The fact of making the probe penetrate without rotation and with very slight circulation flow rate does not disturb the walls of the borehole and thus almost entirely eliminates any danger of cave-ins, even in loose formations (sands).

The self-boring pressuremeter has a great many advantages compared with other commonly-used in-situ measuring techniques :

- the "pressure-deformation" curve that is recorded situates the stresses applied to the soil in relation to its limit properties (only values determined by a penetrometer or vane probe);
- the self-boring pressuremeter is the only in-situ measuring device capable of correctly obtaining the deformability properties of the soil (shear moduli at zero or very low deformation);
- formation layers are identified very reliably on the basis of pressuremeter measurements.
- the self-boring pressuremeter is capable of determining the behaviour of soils under the effect of cyclic loading which must necessarily be known for foundation design of offshore structures subjected to wave action.

2. Fields of application of the PAM

The self-boring pressuremeter is particularly well suited for investigating fine sediments, i.e. sands, silts and clays such as those that cover a very large portion of the seabed, especially beyond the continental shelf. It is probably the only reconnaissance device capable of providing accurate information on :

- soft or very soft soils ($C_u \leq 5$ kPa) encountered in deltaic zones and generally covering the continental slope and great ocean depths, and;
- loose carbonate sediments which are very sensitive to any disturbing. In such cases, all other coring or in-situ measuring techniques are incapable of providing meaningful results.

3. Water depths for implementing the prototype device.

The water depths at which petroleum activities are being carried out are constantly increasing, now reaching 1500 meters for exploration and more than 300 meters for production. The development outlook for the very near future is 700 to 800 meters in the Mediterranean and about 500 meters in other parts of the world. The prototype pressuremeter that has been developed meets the present and foreseeable needs. It can already be used at water depths of about 300 meters, and in the near future these depths will be increased to about 1000 meters.

4. Penetration of the probe into the soil and measurements

Production from oilfields beyond the continental shelf will entail the installation of floating structures (tension-leg, semi submersible platforms, etc.) or structures sitting on the seabed (guyed towers, gravity towers, etc.) as well as subsea wellheads and various other seabed installations. The fact that the self-boring pressuremeter probe can penetrate for about 60 meters into the seabed replies to most needs for making a detailed investigation of the soil both before the installation of planned structures and for estimating potential landslides risks on the continental slope.

A device for cyclic pressuremeter testing will be developed in 1982, and it will be of particular importance for several tens of meters of penetration for foundation and anchoring systems design.

OFFSHORE SELF-BORING PRESSUREMETER FOR DEEP WATER

The interest of the prototype device for analyzing landslides risks on the continental slope requires a pore-pressure measuring device to be adapted to the pressuremeter probe.

5. Implementing the PAM

The technique of drilling, with coring or in-situ measurements, implies the availability of a drilling ship having suitable dimensions for the ocean conditions involved. The offshore self-boring pressuremeter on the other hand, has the advantage of not requiring the use of a specialized vessel.

6. Remote controlling the device, data acquisition and processing

All the functions of the device are remote controlled from the surface, either manually for preliminary operations or automatically by means of a computer for the execution of borings and measurements. The prototype device has the advantage of making real-time recordings of :

- self-boring parameters as the probe penetrates into the soil, so that the pressuremeter measurement depths can be chosen judiciously;
- monotonic or cyclic "pressure-volume" pressuremeter curves.

The quick processing of the data aboard ship enables an almost immediate evaluation to be made of the properties of the formations encountered as well as of the preliminary sizing of the planned foundations.

II. DESCRIPTION OF THE OFFSHORE SELF-BORING PRESSUREMETER

The offshore self-boring pressuremeter (Fig. 3) consist of :

- a framework and its equipment on the seabed for implementing the measuring probe;
- a control cabin on the support vessel;
- an umbilical cable connecting the cabin and the framework;
- instrumentation with remote controls and data acquisition.

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1. Seabed framework and its equipment.

A tripod frame sits on the seabed, held up by three large bed footings (Fig. 4)

It has the following general characteristics :

- height = 8,5 meters
- transportation width = 2,5 meters
- ground coverage in working position = 5,8 x 6,8 meters
- approximate weight in air = 16 tons
in water = 14 tons

The bed footings having a surface area of 20 m² ensure the stability of the device on very soft soils. After use, the disengagement of the footings can be facilitated by water injection under pressure, if necessary.

The storage winch, with an outside diameter of 3.8 meters, holds 70 meters of flexible drill stem 135 mm in diameter. This powered drum is also capable of unwinding the flexible drill stem at varying speeds depending on the penetration of the probe into the seabed.

The pulley, 2 meters in diameter and situated at the apex of the tripod, serves as a lazy pulley for the flexible drill stem.

The guide tube has three main functions :

- correcting the verticalness within a range of $\pm 3^\circ$ as a function of the seabed slope;
- making a telescopic tube penetrate about 1 meter into the seabed to cross through the water-soil interface;
- compensating for the reaction forces of the probe packer at the beginning of boring.

The measuring probe, 6 meters long and 160 mm in diameter, hanging from the end of a flexible drill stem, is described elsewhere [10]. Let us merely say that it contains three modules (Fig. 5) :

- the self-boring module;
- the driving module which comes in two variants depending on the soil properties expected to be found :

. a drill-collar weighing about 8 kN which can be used in very soft soils;

OFFSHORE SELF-BORING PRESSUREMETER FOR DEEP WATER

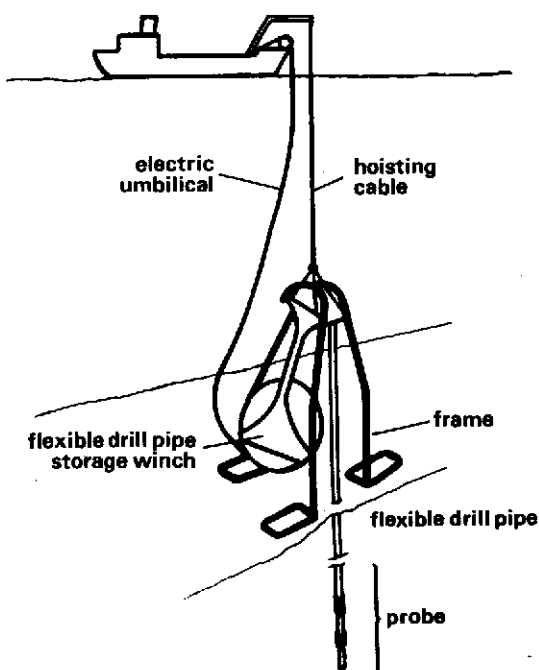


Fig. 3 - Diagram of implementation of PAM

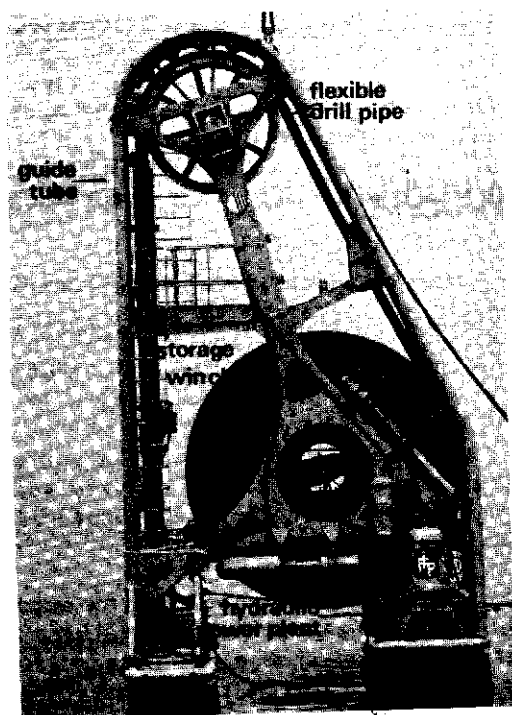


Fig. 4 - Frame of PAM

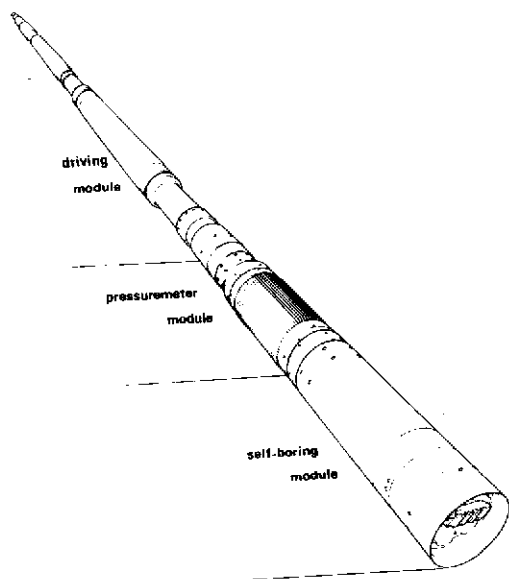
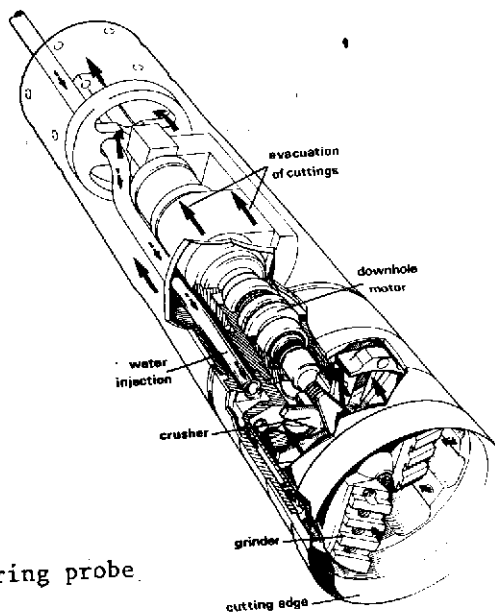


Fig. 5 - LPC/IFP offshore self-boring probe



OFFSHORE SELF-BORING PRESSUREMETER FOR DEEP WATER

- . a jack-packer which ensures a sequential penetration of the probe into the ground, while the inflated packer, locked in the borehole, takes up the reaction (maximum force of 35 kN) in medium stiff to stiff formations;
- the pressuremeter module consisting of an inflatable cylindrical cell 160 mm in diameter and with a slenderness ratio of 2.

The three electro-hydraulic power plants operating in equipressure and controlling all the functions of the device (fig. 6) :

- the 18 kVA power plant (1) which handles the actual function of the framework :
 - . driving the storage drum for the flexible drill stem,
 - . correcting the verticalness of the guide tube, and
 - . driving and extracting the telescopic tube ;
- the 18 kVA power plant (2) which controls the functions of the self-boring probe :
 - . driving the hydraulic motor of the grinder, and
 - . driving the hydraulic motor of the circulation pump during drilling;
- the 1 kVA power plant (3) which controls the driving of the probe jack.

The two "pressure-volume" controllers (known as the CPV from its initials in French) or volumetric pumps with a varying and perfectly controlled flow rate and which are identical in design, handle the expansion of both the pressuremeter cell up to 25 % (i.e. 1.6 liter) of its initial volume and the packer up to 40 % (i.e. 10 liters) of its initial volume.

2. Control cabin

The control cabin situated on the support vessel and equipped with all the electric equipment required for operating the PAM includes (Fig. 7) :

- a power console supplied with 3-phase 380 or 440 V;
- a frequency convertor for controlling at will the penetration of the probe jack (rate of penetration) and the CPV flow rates;
- a manual control panel and a display unit for the different functions of the framework and probe;

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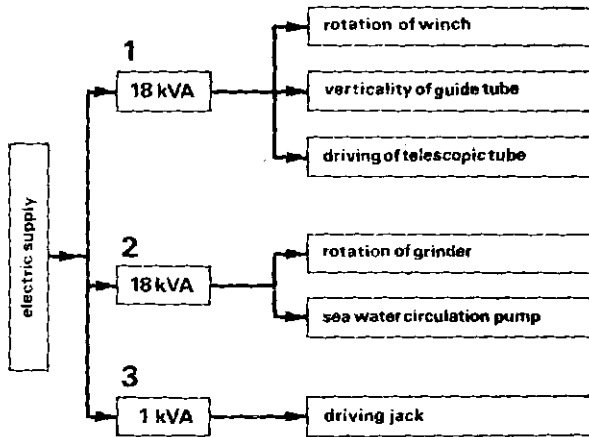


Fig. 6 - Hydraulic functions of PAM



Fig. 7 - Control room of PAM

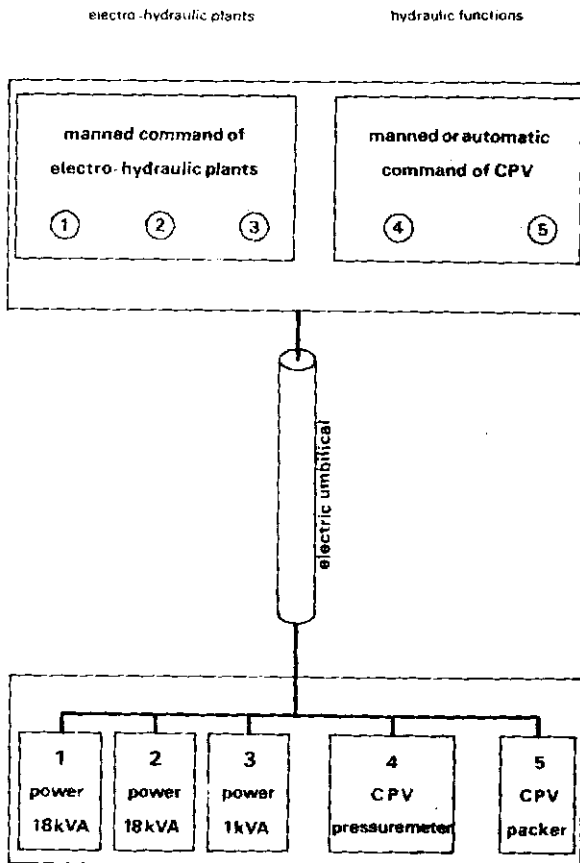


Fig. 8 - Schematic electrical diagram of PAM

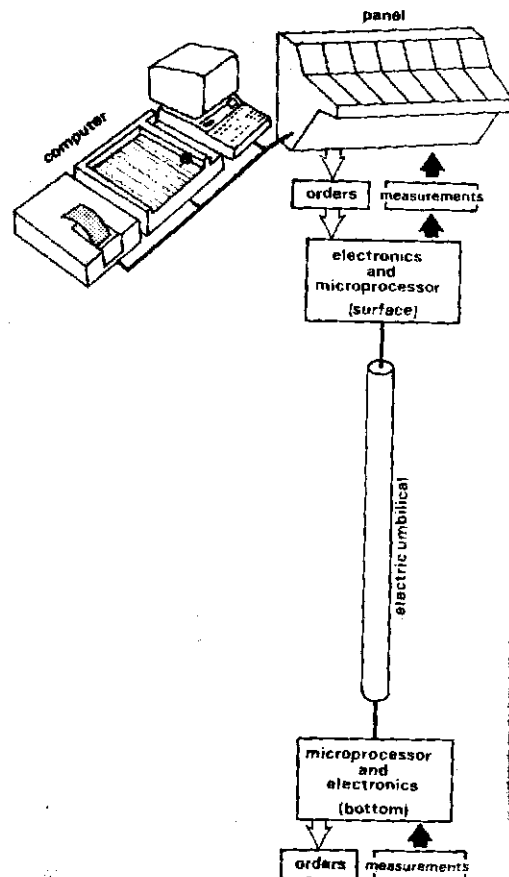


Fig. 9 - Diagram of seabed - vessel transmissions

OFFSHORE SELF-BORING PRESSUREMETER FOR DEEP WATER

- a real-time computer with its peripheral equipment (printer, plotting table) for the automatic control of the device and data acquisition.

3. Electric umbilical

The electric umbilical cable is 63 mm in diameter and handles the following two functions :

- electric power supply (fig.8) for the two 18 kVA hydraulic power supplies and for the motors whose speed is controlled from the surface;
- coaxial transmission of the control and measurement signals (fig.9)

The cable is protected by two crossed armoring systems made of synthetic fibers and is covered by a neoprene sheet. The cable now used is 400 meters long so that the device can be used at water depths of about 300 meters. Project is now being done on adapting the prototype device so that it can be implemented with a cable about 1000 meters long.

4. Instrumentation and seabed-surface transmission

The instrumentation of the prototype device includes some 20 electro distributors for the hydraulic control of the different functions as well as some 30 sensors controlling the operation of the main components or data acquisition.

Two microprocessors, one situated on the seabed and the other on the surface, transmit the control and data-acquisition signals via the coaxial cable. A computer controls all the penetration operations of the probe, the execution of the pressuremeter tests and data acquisition (self-boring parameters and "pressure-volume" pressuremeter curves) recorded simultaneously on cassettes and on a printer (or plotting table (fig.9).

OFFSHORE SELF-BORING PRESSUREMETER FOR DEEP WATER

III. OPERATION OF THE OFFSHORE SELF-BORING PRESSUREMETER

Implementation of the device includes lowering the framework to the seabed and then executing the boring.

1. Implementing the framework on the seabed

The device is commonly operated on a non-specialized ship (of the supply-boat type) equipped with (fig. 10) :

- A-frame or gantry crane with a capacity of 20 to 30 tons to remove it over the ship board. This operation assumes that wave height is no more than 1 or 2 meters depending on the stability of the ship used; and
- a winch with a hoisting capacity of about 30 tons so the framework can be hoisted back up (if the footings should sink into the seabed).

The framework is lowered by means of a carrying cable which is separate from the self-supporting umbilical cable. The positioning of the ship may have a tolerance of movement of several meters due to flexibility of the seabed-surface links.

A procedure for casting off the umbilical cable enables the device to be temporarily abandoned if meteorological conditions so require.

The device can also be implemented from a drilling ship equipped with a central moonpool at least 7 meters wide.

2. Stability of the framework on the seabed

The surface area of the footings (20m²) provides stability for the device on very soft soils having a surface cohesion higher than about 1.5 kPa.

Furthermore, the stability of the device under the effect of currents is ensured if the velocity of these currents is no more than 2 knots.

The guide tube is used to place the probe in a vertical position when the seabed slope is no greater than about 5 %.

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3. Executing a boring

Any implementation of the pressuremeter is preceded (before lowering the device to the seabed) by calibrations so as to make further corrections in the measurements performed in the soil :

- determining the correction curve $(p, \frac{\Delta V}{V})_1$ of the pressuremeter measuring circuit obtained with zero expansion of the pressuremeter measuring cell (volume correction);
- determining the rigidity curve $(p, \frac{\Delta V}{V})_2$ of the pressuremeter membrane (cell) expanded freely in air to $\frac{\Delta V}{V} = 25\%$. (pressure correction)

Penetration of the probe into the ground, between the depths chosen for pressuremeter testing, takes place at a velocity of between 5 and 25 cm/min depending on the nature and properties of the formation:

- either continuously by means of a drill-collar module which can be used in soft soils, or
 - by 0.5 meter sequences using the "jack-packer" module according to the procedure described elsewhere in sands and stiff clays [10].
- The tension of the flexible drill stem measured at all times by two sensors controls the rotation of the storage drum as the probe penetrates into the formation.

The penetration of the probe is accompanied by the recording of self-boring parameters every 10 cm to provide information on the properties of the formations crossed through (Fig.11) :

- driving force, Q
- pressure inside the pressuremeter cell at zero expansion, p_i
- rate of penetration
- torque on grinder
- circulation water pressure

The pressuremeter test with simultaneous recording of the "pressure-volume" curve is performed at each previously determined level (every meter or 2 meters) or at a level chosen by the operator as a function of information deduced from self-boring parameters (Fig. 12 and 13). Pressure and volume corrections are applied to the pressuremeter curve in delayed time.

The time required for making a probe covering about 50 meters, with one pressuremeter test every meter or every two meters, is about 24 to 36 hours after the device has been set on the seabed.

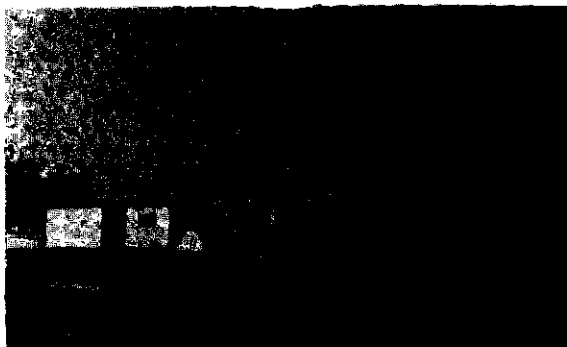


Fig. 10 - Implementation of PAM in the Mediterranean

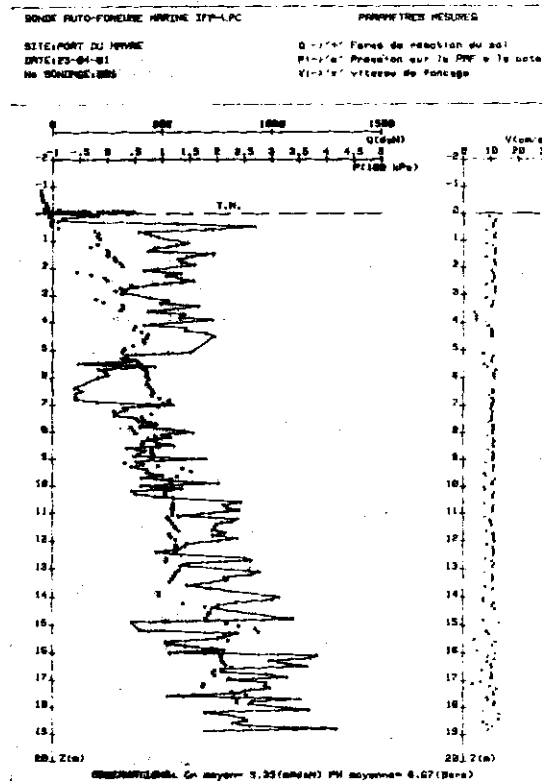


Fig. 11 - Self-boring parameters recorded in a silty sand (Le Havre)

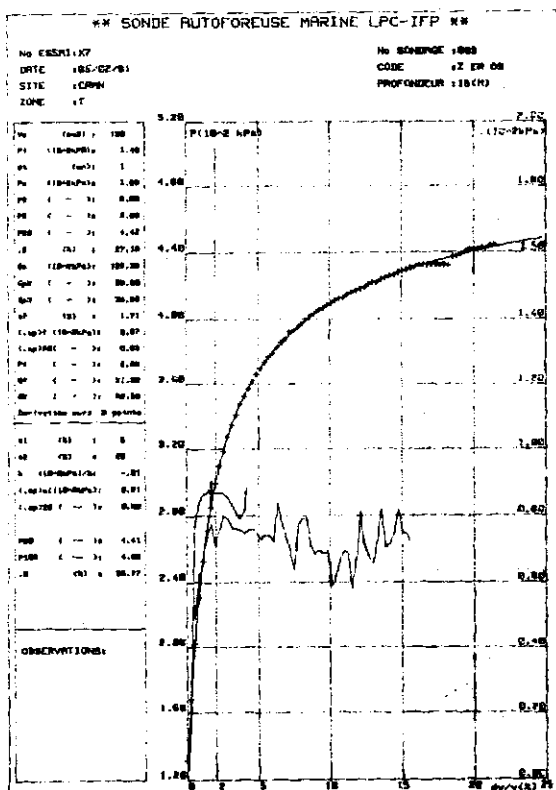


Fig. 12 - Pressuremeter curve in a clay (Cran)

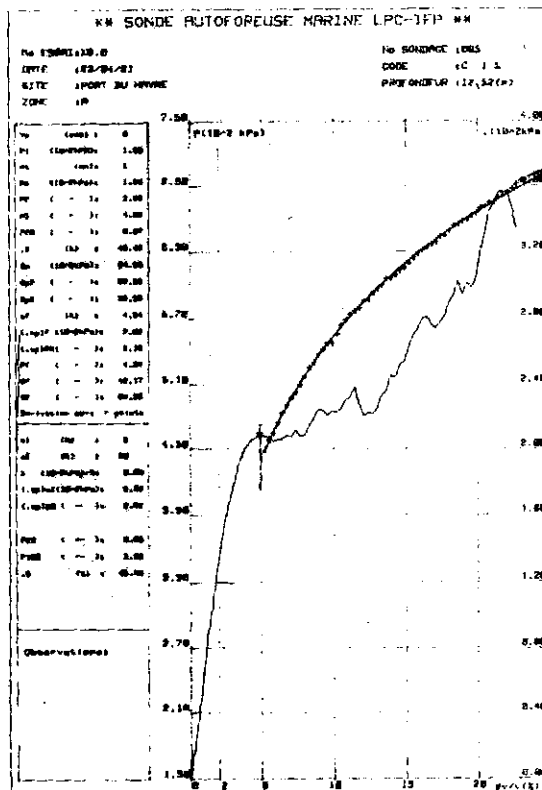


Fig. 13 - Pressuremeter curve in a silty sand (Le Havre)

OFFSHORE SELF-BORING PRESSUREMETER FOR DEEP WATER

IV. TESTS OF THE OFFSHORE SELF-BORING PRESSUREMETER

The offshore self-boring pressuremeter has been systematically tested and experimented. Preliminary tests of the main components (offshore self-boring probe, electro-hydraulic power plants, seabed-surface transmissions, pressure-volume controllers, different sensors, etc.) served to prove their perfect reliability. The device was then tested, first on land and then at sea.

1. Onshore tests of the device

The first test phase for the device was carried out at a well-known onshore site at Cran (near Redon, France), in February 1981, for the purpose of checking how the different components operated and of testing the performances of the probe with two variants of the driving module ("drill collar" and "jack-packer") on a site having thoroughly known properties.

Five borings into normally consolidated clay ($20 < C_u < 50$ kPa) were carried out to a maximum depth of 17 meters (bottom limit of gravelly soil) with one pressuremeter test every meter or every two meters (Fig.12). The results obtained are in very good agreement with the ones previously deduced from prior tests at the same site with the Laboratoires des Ponts et Chaussées PAF 76 self-boring pressuremeter.

2. Testing the device at shallow water

The second phase of testing the offshore self-boring pressuremeter was performed in April 1981 in a basin in the port of Le Havre with a water depth of about 10 meters. The device was implemented from a dock by means of a crane. The aim of this phase was :

- to confirm the underwater operational reliability of the device;
- to test the procedures for controlling the installation of the device on the seabed;
- to check the performances of the self-boring probe in a compact silty sand.

Five borings were performed with pressuremeter tests every meter or two meters. The level of pebbles was attained at a depth of 19 meters after about 15 hours of operating (Fig. 11 and 13).

3. Offshore Tests

The third phase of experimenting the pressuremeter took place in the Mediterranean using the dynamically-positioned ship Castor 02 which is 45 meters long and is equipped with mobile A-frame for swinging the frame out over the stern of the ship (Fig. 10). Because of the particularly bad meteorological conditions, two experimental campaigns were carried out in November-December 1981 and February 1982. These experiments served to test the operating of the device at different water depths of up to 300 meters.

Three boreholes involving a total of 28 pressuremeter tests were performed in the port of Fos-sur-Mer in sandy-silty formations running down to the top of a pebbly formation (depth of 12.5 meters under the sea-floor).

A borehole was performed in the Gulf of Lion about 12 km SE of Sete at a water depth of 50 m, with a penetration depth of 29.1 m in a normally consolidated clayey formation (Fig. 14). Total time required for lowering the framework and setting it on the seabed was about one hour. Penetration time to a depth of 29.1 m under the sea-floor and with 23 pressuremeter tests was 17 hours. The probe penetrated into the seabed under the effect of gravity or helped along by the jack-packer system, depending on the depth. The particularly reliable pressuremeter measurements results bear witness to the excellent behavior of the device.

The pressuremeter was then submerged at gradually increasing water depths of up to 270m. The tests and checks performed at water depths of 100, 200 and 270m gave proof of the entirely satisfactory behavior of the device (storage winch for the flexible stem, water circulation during penetration, grinder etc.) and of the measuring system.

The results of the different experiments performed lead to the conclusion that, as of now, the pressuremeter is capable of performing normal operations at water depths of 300 m.

CONCLUSIONS

1. For many years the need to improve man's knowledge of offshore soil for the installation of petroleum structures has been felt very keenly by all experts. The offshore self-boring pressuremeter (PAM) which has been developed and tested with success by the Institut Français du Pétrole for implementation at great water depth meets this need.
2. The prototype device developed for water depths of from 300 to 1000 meters has a great many advantages compared with techniques that

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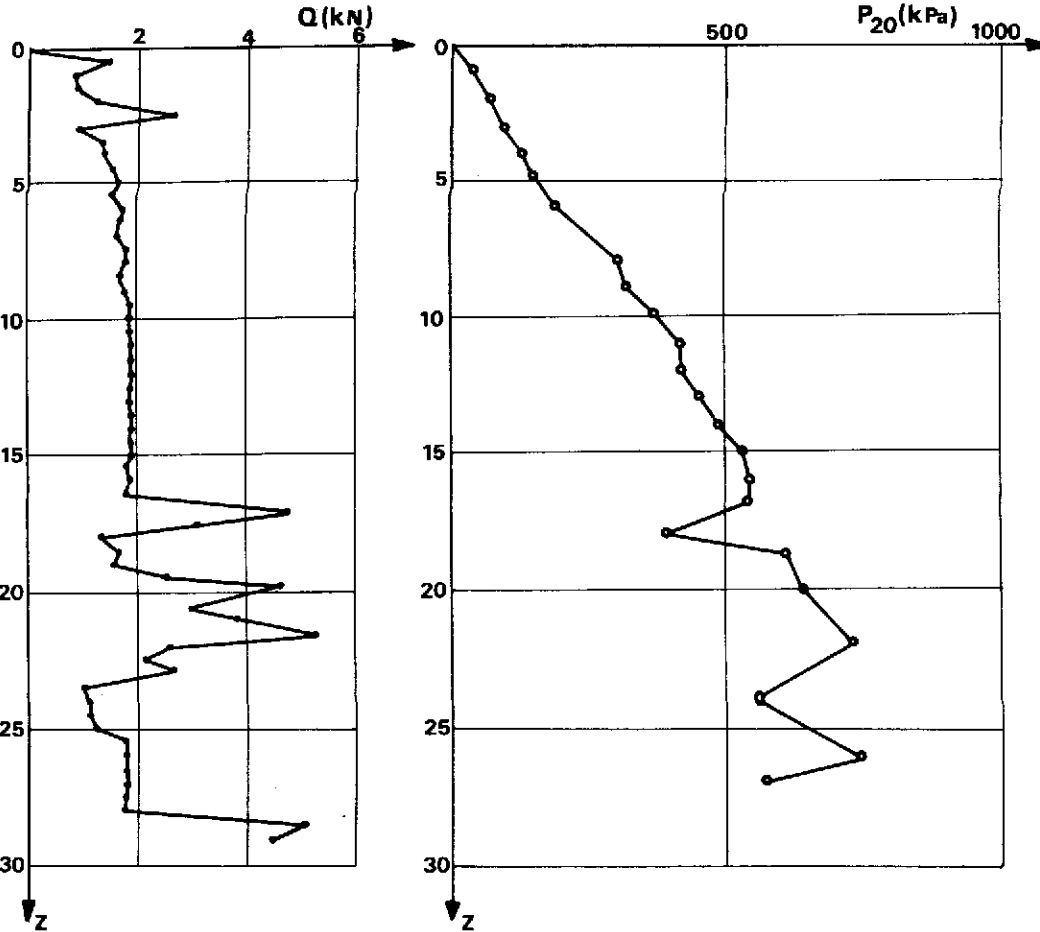


Fig. 14 - Borehole in the normally consolidated clay of Lion Gulf (50 m water depth)

OFFSHORE SELF-BORING PRESSUREMETER FOR DEEP WATER

are currently in use :

- from the standpoint of understanding soil properties, the self-boring pressuremeter provides representative data on the geotechnical properties of soils and serves to determine the behaviour of soils under cyclic loading, and this is of great importance for foundation design of offshore structures.

- from the standpoint of implementation, the device does not require the use of a specialized vessel (drill ship).

3. The different tests of the device on land, at shallow water depth for about 100 hours and at 50 meters of water depth at sea have given proof of the operational reliability of the device and the quality of the measurements obtained from the dual standpoint of self-boring parameters and pressuremeter measurements.

4. The characteristics of the prototype offshore self-boring pressuremeter (water depths of application, operational remote controls, automatic data acquisition, etc.) make this prototype a new-generation device for soil reconnaissance techniques at great water depth.

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SESSION 3

ADDITIONAL CONTRIBUTIONS AND DISCUSSION

DISCUSSION

M. JAMIOLKOWSKI- (*Technical University of Turin*) to Mr. ST. JOHN

Do you suggest that the unloading range Δp should be less than $2C_u$?

H. ST. JOHN- (*Building Research Establishment*)

In actual fact, the amplitude of the stress variations during unloading/reloading cycles must be limited to $2C_u$ to prevent soil failure in extension.

CONTRIBUTION

HYPERBOLIC MODEL FOR THE PRESSUREMETER CURVE

J.L. BRIAUD- (*Texas A & M University*)

Using a preboring pressuremeter the following test was performed. A strain controlled test was performed with an unload-reload cycle as shown on figure 1 ; note that the strain controlled test allowed to obtain a large number of data point on the curve. Point J was then considered as the origin and secant moduli corresponding to the slopes JK, JL, JM,..., were calculated for strain increments ϵ corresponding to injected volumes between J and K, J and L, J and M, ...

SESSION 3 ADDITIONAL CONTRIBUTIONS AND DISCUSSION

Figure 2 presents the plot of the inverse of the secant modulus versus the strain increments.

A similar procedure was applied for the unloading part of the cycle by taking C as the origin. The two straight line regressions of figure 2 show that the hyperbolic model is predicting reasonably well the shape of the cycle. Indeed hyperbolic model

$$\sigma = \frac{\epsilon}{a+b\epsilon}$$

$$\text{or} \quad \frac{\epsilon}{\sigma} = a+b\epsilon$$

$$\text{or} \quad \frac{1}{E} = a+b\epsilon$$

Note that the hyperbolic model does not fit as well at lower strain values. However by extension of the $\frac{1}{E}$ vs ϵ plot a zero strain modulus can be obtained for the unload part of the cycle E_u (Fig. 2) and for the reload part of the cycle E_R . The modulus E_u is measured at the radial stress p_u (Fig. 1) and E_R at p_R . Then, using the commonly used Jambu's formula one can write :

$$E_R = K \left(\frac{p_R}{p_a} \right)^n$$

$$E_u = K \left(\frac{p_u}{p_a} \right)^n$$

$$\text{and} \quad \frac{E_u}{E_R} = \left(\frac{p_u}{p_R} \right)^n$$

The stress dependency exponent n can then be obtained. A number of assumptions are made in the above procedure which at the present time make of n an index more than an exact parameter which could be used in finite element method for example.

C.P. WROTH- (*University of Oxford*)

(a) We must be careful about this because effective stresses are considered in these models, whereas the pressuremeter measures total stresses.

SESSION 3 ADDITIONAL CONTRIBUTIONS AND DISCUSSION

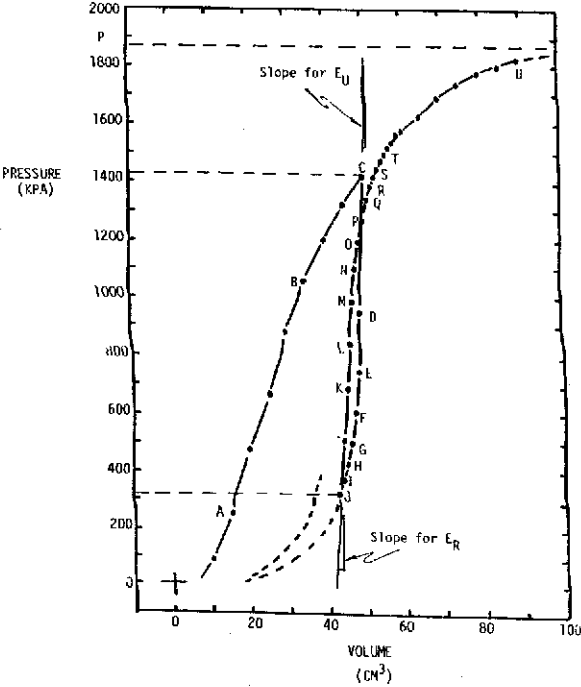


Figure 1

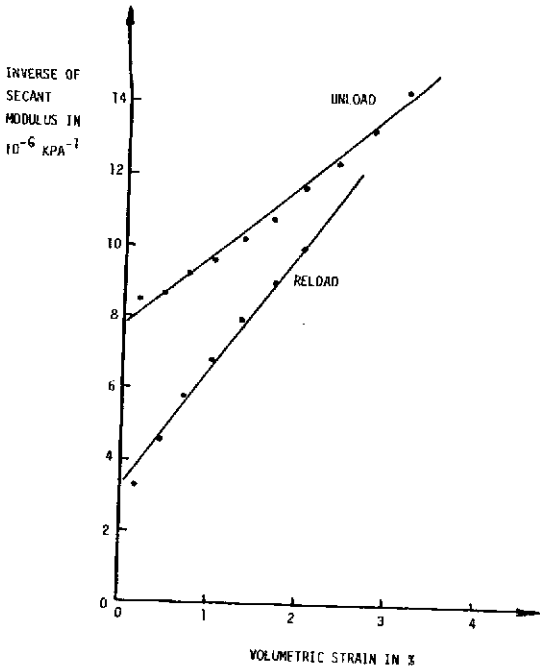


Figure 2

SESSION 3 ADDITIONAL CONTRIBUTIONS AND DISCUSSION

(b) The measurement of the shear modulus according to unloading/reloading loops is very sensitive to the amplitude of the variation in the applied deformation.

(These considerations are discussed in Mr. Wroth's paper "British experience with the self-boring pressuremeter" submitted after the Symposium).

COMMENT by

G. SANGLERAT- (*Sols Investigations - Lyon*)

On the LPC-TLM pressio-penetrometer

This measurement instrument is extremely interesting. In fact, continuous measurement using the penetrometer helps to pick the right measurement levels for the pressuremeter.

COMMENT by

A. VAN WAMBEKE- (*Ecole Royale Militaire, Brussels*)

On the use of LPC-TLM pressio-penetrometer.

The limitation of pressuremeter measurement locations may be dangerous. Systematic measurement is recommended using the pressuremeter.

J.F. JEZEQUEL- (*Laboratoire des Ponts et Chaussées de St-Brieuc*)

In reply to Mr. VAN WAMBEKE

Of course the ideal situation is to obtain reliable data concerning the entire profile tested in both hard and soft layers. In practice however, time is often lacking and so more attention has to be paid to the tests on soft horizons rather than on hard ones. This can be tolerated more easily if one has the good luck to have both continuous (penetrometer) and spot (pressuremeter) measurements as is the case for the device shown here, i.e. the LPC-TLM pressio-penetrometer.

The pore pressure measurement cell is located between the penetrometer tip and the pressuremeter cell. Hence the measurement is independent of the pressuremeter test. It serves to determine the existence of pore overpressures in the soil, and also to have an idea of the nature of the soil at the level of each pressuremeter

SESSION 3 ADDITIONAL CONTRIBUTIONS AND DISCUSSION

test. In fact, when instrument driving is interrupted, and during the pressuremeter test, the initial value and the variation in over-pressure caused by driving are recorded. They obviously depend on the type of soil. The measurement cell also serves subsequently for permeability measurements.

COMMENT by

J.L. BRIAUD- (*Texas A & M University*)

Just as the offshore self-boring pressuremeter tests in shallow water depths have pointed out the change in shear strength of Flanders clay due to its unloading, measurements on samples taken with the Texas A and M University pressure corer have revealed the importance of in situ conditions on the value of C_u . The undrained cohesion C_u before stress relaxation falls routinely to $0.7 C_u$ stress relaxation.

CONTRIBUTION

EXAMPLES OF OFFSHORE APPLICATIONS OF THE PRESSUREMETER

J.P. MIZIKOS- (*Société Nationale Elf-Aquitaine*)

My comments are related to two examples of offshore applications of the pressuremeter.

(1) The first example concerns a site with alternating under-consolidated muds and sands, where a jack-up platform was to be installed, resting on the seabed on caissons.

Various tests were performed with the Ménard pressuremeter to evaluate the bearing capacity of the sands.

Existing correlations between pressuremeter and conventional data proved unsatisfactory. We therefore resorted to an original physical analysis using the entire pressuremeter curve, and this yielded good forecasts.

Thus it seems a good idea to get out of the rut of correlations and insufficiently checked theoretical interpretations and to rely more on physical logic.

(2) The second example concerns a North Sea gravity platform (see another paper at this Symposium) under which pressuremeter cells are placed.

SESSION 3 ADDITIONAL CONTRIBUTIONS AND DISCUSSION

After six months of rest, the following was observed in the initial state :

$$p_t - u_0 = 200 \text{ kPa}$$

and at failure :

$$u = \text{constant and } p_t - u = 600 \text{ kPa,}$$

where p_t = total stress, and u = pore pressure.

This meant a higher effective pressure, which is absurd. The measured pore pressure is thus not representative of reality. Only the total pressures can be measured with the pressuremeter.

It has turned out that the knowledge of effective stresses is very important for offshore geotechnical problems, such as driving, cyclic loadings, etc. Thus the pressuremeter does not meet offshore requirements, especially for axial capacity design of offshore piles.

F. BAGUELIN- (*Laboratoire Central des Ponts et Chaussées*) in
reply to Mr. MIZIKOS

To talk of the rut of correlations is going too far. It must be remembered that the Ménard calculation method has been tried and proven, and that the accuracy of 30% obtained in forecasting is acceptable. In any case, the comparison between theoretical results from experimental measurements and real observations of foundation behavior remains absolutely indispensable.

CONTRIBUTION

FORCE/DISPLACEMENT CURVES OF TWO PILES FROM SELF-BORING PRESSUREMETER TEST RESULT

R. FRANK- (*Laboratoire Central des Ponts et Chaussées*)

Some previous papers and statements have dealt with the problem of pile design, and the use of pressuremeter characteristics in analyses by finite elements. I should like to show briefly the pullout load/displacement curves obtained for two closed tubular piles, driven into marine soils, by using the parameters G_0 and P_{20} measured using the self-boring pressuremeter (G_0 = initial shear modulus, and P_{20} = pressure for $AV/V_0 = 20\%$). Further details may be found in the paper by F. Baguelin, R. Frank and J.F. Jézéquel at the Second International Conference on "Numerical Methods in Offshore Piling" (Austin, Texas, USA, 29/30 April 1982).

SESSION 3 ADDITIONAL CONTRIBUTIONS AND DISCUSSION

Figures 1 and 2 show three curves in each of the two following cases :

- . the experimental curve obtained by a static pullout test with load stages maintained for one hour ;

- . the curve calculated by finite elements, in a heterogeneous elastic soil ($E = 3 G_0$ at each level, $\nu = 0.49$), and with special contact elements between the pile and the soil, designed to limit friction to the value q_s (determined at each level by the value of p_{20}) ;

- . the curve calculated by integration, along the shaft, of the lateral friction mobilization curves (analysis by load transfer functions) determined as indicated in figure 3.

In the case of the Cran soft clay site, the results obtained are excellent, especially in the vicinity of the working load Q_a (Fig.1).

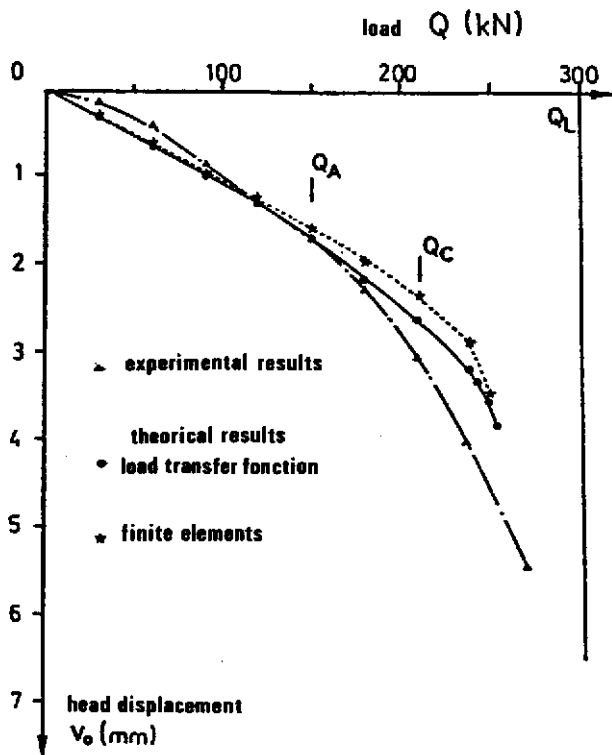


Figure 1 - Comparison of the experimental and theoretical load-displacement curves for the pulling-out test at Cran.

SESSION 3 ADDITIONAL CONTRIBUTIONS AND DISCUSSION

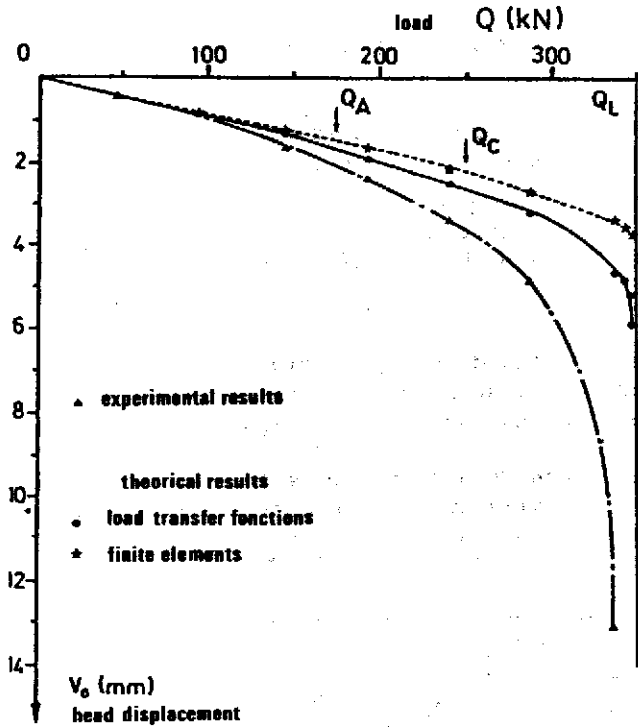


Figure 2 - Comparison of the experimental and theoretical load-displacement curves for the pulling-out test at Plancoët.

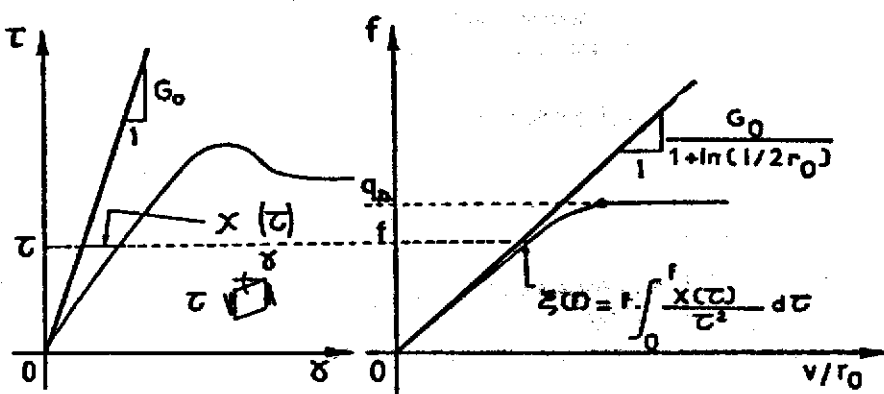


Figure 3 - Shear curve (τ, γ) of a soil element and determination of the shaft friction curve $(f, \frac{v}{r_0})$

SESSION 3 ADDITIONAL CONTRIBUTIONS AND DISCUSSION

In the case of Plancoët (sandy silts, loose sands and sandy clays), agreement is not so good, and displacement subject to the working load is underestimated by about 25 or 30 % (Fig. 2). In the case of sensitive soils, the results can be explained by greater disturbance around the pile than around the pressuremeter probe inserted by self-boring.

COMMENTS by

A. VAN WAMBEKE- (*Ecole Royale Militaire - Brussels*), about :

THE VARIETY OF PRESSUREMETER APPARATUS AND THE NEED TO SPECIFY THE TYPE OF INSTRUMENT USED AND THE PROCEDURE APPLIED

The success of what can be called the "pressuremeter method" stems from the initial uniformity of the instruments and methods involved.

Thanks to this uniformity, and certainly also thanks to the inventive dynamism of its promoter, the pressuremeter has rapidly offered the practicing geotechnician a whole body of reference geotechnical parameters (essentially but not exclusively the limit pressure and the pressuremeter modulus) as well as calculation methods. These methods have been confirmed by extensive use and by continual experimental checking, which have helped to solve most of the fundamental problems faced by geotechniques applied in a semi-pragmatic approach, which has largely proved its efficiency.

The present situation is different.

The success of the pressuremeter and the value of its applications, especially offshore, have given rise to a large number of new and innovative instruments. This proliferation is the result of the prodigious advance of electronics and automation, and also of the availability of large credits granted to develop a reliable and promising method.

This virtually explosive development, which is certainly laudable, also incurs certain risks.

The measured pressuremeter characteristics closely depend on the type of instrument (single- or three-cell probe with a given length and diameter, probe insertion method, etc) and on the procedure followed (controlled pressure or controlled deformation test, number and duration of load stages). One can thus anticipate a dispersion for which the prospected soil itself is not responsible, and which could lead to increasing inaccuracies in the use of the formulas, requiring the use of more generous and hence uneconomical safety factors.

SESSION 3 ADDITIONAL CONTRIBUTIONS AND DISCUSSION

One might imagine a radical remedy for this risky situation, i.e. standardization. We do not believe in it. First, because it is liable to be rigid and constrictive. Secondly because, for the case at hand, it would be perfectly inoperative. It is highly unlikely that the promoters of a sophisticated instrument, developed at considerable cost, will renounce its use to satisfy a requirement that is imposed from outside.

We can see a reasonable solution in two directions. To begin with, the need to define reference characteristics, obtained in clearly defined procedure and equipment conditions. We also believe that the adoption of the original pressuremeter test as a reference test would largely be justified chronologically and logically.

This can also be achieved by a patient and stubborn search for existing correlations between the reference characteristics and those obtained using any other type of instrument and method.

Another temptation of which the risks deserve to be assessed is that of excessively automating the interpretation of pressuremeter test results. Everything is ready to design a foundation, formulas included, without the need for the engineer's critical approach.

This is extremely dangerous. The engineer's intervention is indispensable between, on the one hand, primary interpretation, which should lead not only to the determination of the pressuremeter characteristics, but also to the production of complete pressuremeter curves, and, on the other, the practical use of these characteristics.

SESSION 4

Chairman : L. C. REESE
University of Texas, Austin, Texas, USA

The pressuremeter and foundation engineering

APPLICATION OF MÉNARD PRESSUREMETER TESTS RESULTS TO FOUNDATION DESIGN

SUMMARY

By taking into consideration the various data that a pressuremeter test can yield to derive rules for foundation design Louis Menard made a great achievement. Not only could he predict the ultimate bearing capacity of any type of full scale foundation (footings, piles, etc . . .) but also he was able to assess vertical settlements and horizontal displacements of these foundations. Then there was no limit to the scope of application of these design rules based on pressuremeter tests results : they became of great assistance in the design of sheet pile walls, slurry or diaphragm walls, tunnels, penstocks, etc . . . Their use extended to the off-shore field as well.

I. GENERAL

One must remember the state of the art of Soil Mechanics at the time Louis Menard started his professional activity, in the late 50's. This was highlighted at the tenth Rankine Lecture (Roscoe 1970) : "The designer was forced to assume that the foundation did not move until the soil failed, and then the movements were catastrophic. Even for failure conditions the current theories were lamentably restricted". "Load deformation curves were never discussed, except of course in the section on long term one dimensional settlements or when the soil was assumed to be a bed of elastic springs". The great achievement of Menard was to develop a true scale loading test, that J. Leblanc described earlier in this Symposium, and to formulate specific rules for foundation design. These rules were based on the various theories on hand at that time and on in-house experiments on full scale foundations.

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These rules used the data derived from the pressuremeter test namely the shear modulus G , the limit pressure and the so-called creep pressure (or yield pressure).

Naturally none of the theories we mentioned above, that is the theory of purely elastic soils (Lamé,1852) and the theory of plasticity (Hill,1950) were as refined as they are now. The theory of elastic-plastic behaviour of soil was in its infancy. He used them all in his first analysis (Menard 1955, Menard 1957) based on the assumptions of figure 1. Then it took him some time to be able to sort out the various discrepancies which appeared between the results derived from the pressuremeter test data using conventional formulae and the observation of full scale foundations. This is why his first paper on settlement calculation only appeared in the early 60's (Menard et Rousseau, 1962). Immediately after came the paper dealing with the bearing capacity rules (Menard, 1963). We shall start with the latter.

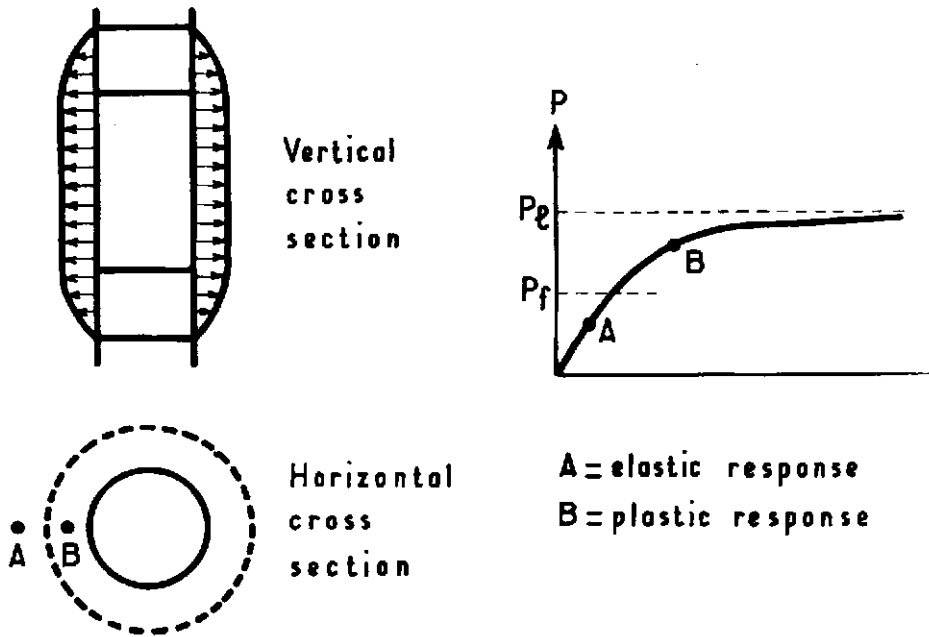


Fig. 1 Assumptions of soil behaviour

APPLICATION OF MENARD PRESSUREMETER

II. BEARING CAPACITY RULES

Menard derived his bearing capacity rules from the comparison of the behaviour of the soil around a pressuremeter probe and around a foundation. This led him to write :

$$q_l - q_o = k (p_l - p_o) \tag{1}$$

- where q_l is the ultimate bearing capacity of the foundation element
- q_o is the vertical pressure at rest
- p_l is the limit pressure measured by the pressuremeter
- p_o is the horizontal pressure at rest
- k is the bearing factor.

This factor is a function of only 3 parameters : the relative depth of the foundation, the classification of soil (clay, silt, sand) and the shape of the foundation (isolated or strip footings, piles or diaphragm walls, etc . . .).

Equation (1) now seems naturally self evident. This equation incorporates the elastic and plastic response of the soil. Consequently k values increase with depth until such a depth where there is no more surface effect : the sheared zones (around the pile for instance) develop against the soil which shows an elastic response, exactly like around a pressuremeter probe (fig. 2). Menard was among the first to point out the importance of critical depth.

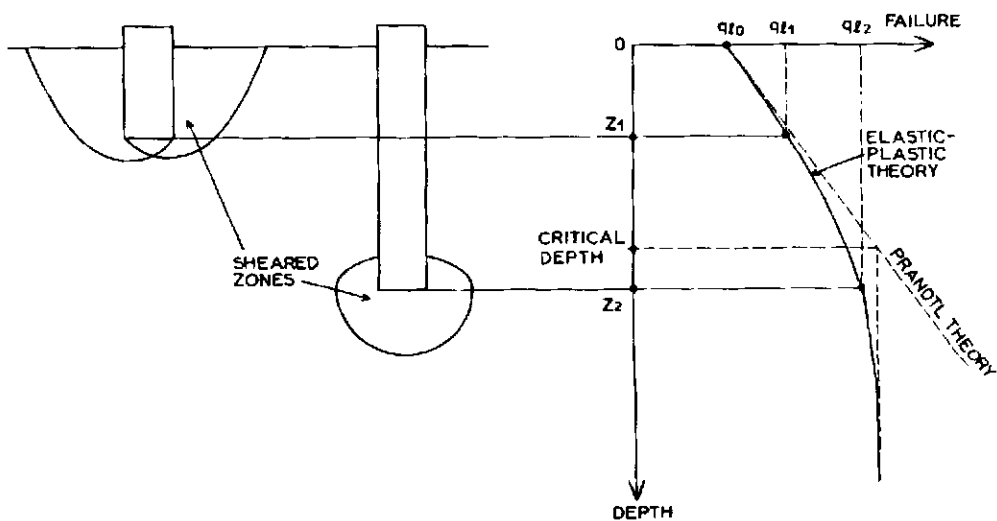
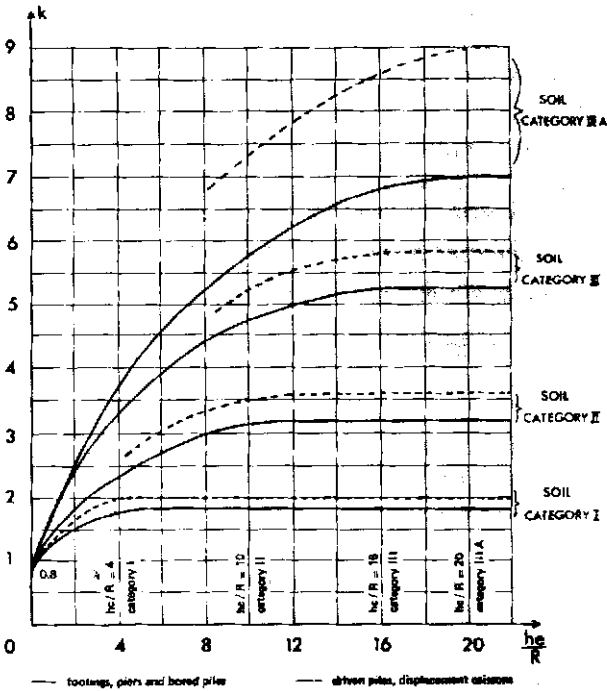


Fig. 2

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Typical variations of k values are given in figure 3.

Fig. 3



Values of k could simply be derived by comparing pressuremeter loading tests and foundation loading tests. However Menard had also a theoretical approach to this problem (Menard and Barraud, 1959) and he was impressed by the similarity between the behaviour of the soil around a pile base and around an expanding sphere (Skempton et al. 1973) :

$$q_l - q_o = k' (p_{l(s)} - p_o)$$

where $p_{l(s)}$ is the limit pressure obtained during the expansion of a sphere and k' is a bearing factor related to a spherical probe. Further, we have a relationship between the limit pressures obtained during expansion of spherical and cylindrical cavities :

$$p_{l(s)} - p_o = k'' (p_l - p_o) \tag{2}$$

and finally :

$$q_l - q_o = k' \cdot k'' (p_l - p_o)$$

with $k = k' \cdot k''$

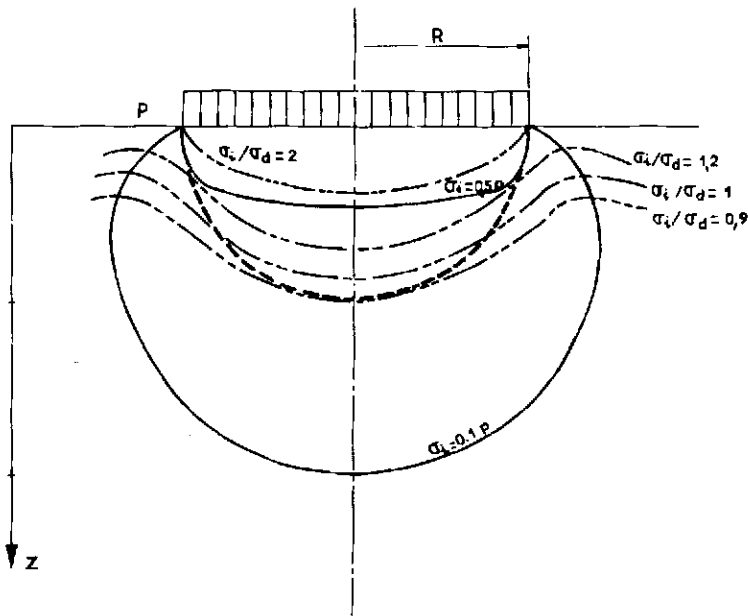
APPLICATION OF MENARD PRESSUREMETER

Since then many research workers have pinpointed this practical approach (Zanghellini, 1973, Foray & Puech, 1976) and, even if some recent loading tests on fully instrumented piles has shown a trend to alter some k values (Bustamente et al, 1981) like the ratio between q_i and q_c , the rules developed by Menard have found a wide acceptance.

To make this chapter complete we must also mention the experimental rules for skin friction on the various type of piles.

III. VERTICAL SETTLEMENTS

By dividing the stress and strain tensors at each point in a semi-continuous medium loaded by a footing into isotropic and deviatoric tensors (fig. 4) Menard was able to show that settlements could be calculated by using a G modulus measured by a vertical cylindrical probe.



$$w = w_i + w_d = \frac{2pR(1 + \gamma)}{9K} + \frac{pR(2 - \gamma)}{3G}$$

Fig. 4 Contours of the isotropic tensor σ_i and of the ratio σ_i/σ_d (where σ_d is the deviatoric tensor) for a circular, uniformly loaded foundation (Poisson's ratio = 0.3) (after Cordary, not published).

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To take into account the remoulding of the borehole walls the formula of Menard differs from the theoretical value (Cordary et al, 1981) but its results have been proved to be accurate (Amar et al, 1977) as long as stresses were not including plasticity.

The same type of approach could be also used to assess the vertical settlement of piles (Gambin, 1963, Marchal, 1971) and this method has been successfully extended to C.P.T. results (Verbrugges, 1981). Some recent observations confirm the interest of this method (Gambin, 1977, Van Wambeke, 1980, Sellgren, 1981) which involves plastic deformations along the pile shaft (fig. 5).

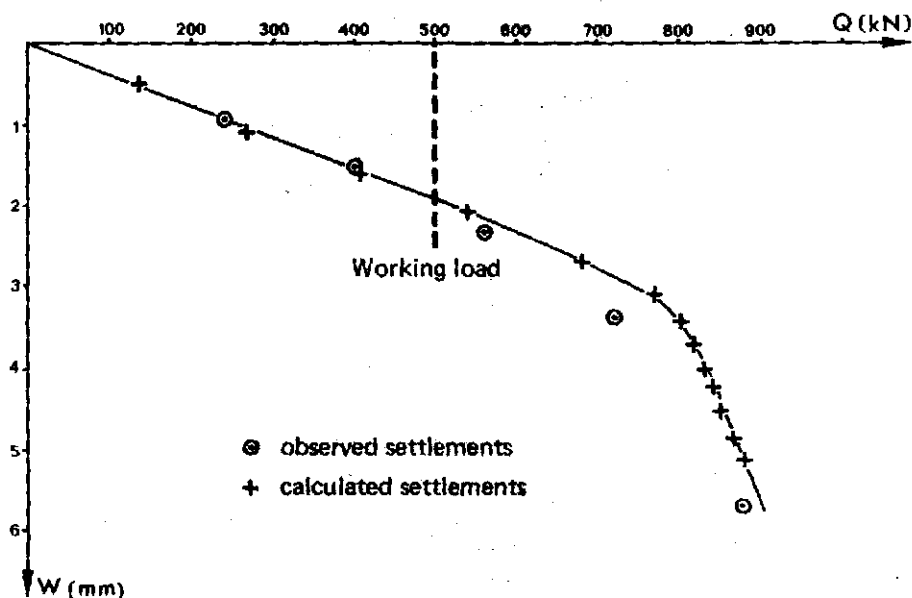


Fig. 5 Pile settlement as calculated and observed

IV. HORIZONTAL DISPLACEMENT

The author (Gambin, 1979) recently tried to review all the work of Menard in this field, under both theoretical and experimental aspects, limited to pile design.

Soil response is variable according to the relative stiffness of the foundation compared with the supporting soil. This method takes into consideration a modulus of reaction of the soil for each layer and looks at the displacement depending on the elasticity of the piles.

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Displacement of sheet pile walls has been analyzed in various papers (Menard et al, 1969, Bourdon, 1980 b). Complete design of diaphragm walls was recently proposed in terms of pressuremeter data (Bourdon 1980 a).

The Port of Paris Authority for instance has been using computer programmes based on this work to design its dolphins and bulkheads for more than 10 years, resulting in safer and more economical solutions.

In all these cases the $p.y$ curves are limited to 2 straight sections (fig. 6).

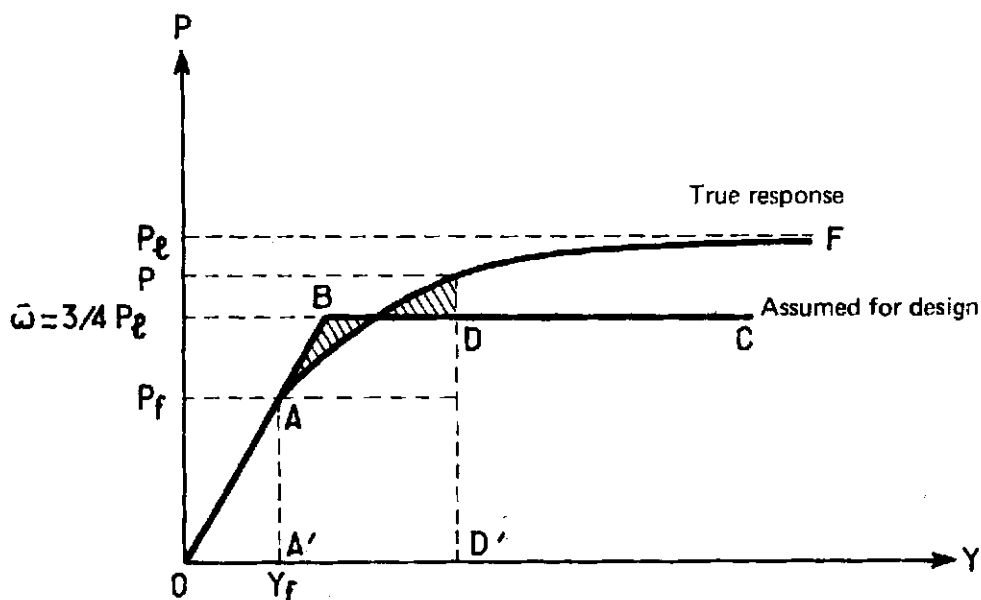


Fig. 6

V. APPLICATION TO OFF-SHORE PROJECTS

Since 1965 pressuremeter tests results have been widely used in the design of off-shore foundations. J. Leblanc has shown earlier in this symposium that pressuremeter tests can be carried out in marine environment and that in 1970 more than 4000 tests had already been run, the probe being placed by a free falling underwater device or by vibro-driving or by more conventionnal means. Jezequel, Lamy and Perrier, also submitted another piece of equipment which consists of a cone penetrometer, a pressuremeter and a pore water pressure sensor, installed by a vibro-hammer and connected to various automatic recording and processing modules. This equipment has been in service for 2 years and apart from running in problems it has received acclaim from engineers and clients.

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The range of projects includes :

- berthing facilities (anchored bulkheads, dolphins, gravity piers, etc . . .)
- jack-up barges and fixed platforms for the oil industry, anchors, etc . . .

However equation (1), which applies to the bearing capacity of a footing must be replaced by equation (2) for the pull out capacity of an anchor.

To date all the off-shore foundations designed using pressuremeter results have not undergone any excessive settlements over and above the design predictions.

VI. CONCLUSION

The use of pressuremeter test results to design off-shore foundations has been proved to be very valuable. This is simply due to the fact that a pressuremeter test is a true scale loading test and can be interpreted using elastic-plastic theory of soil behaviour.

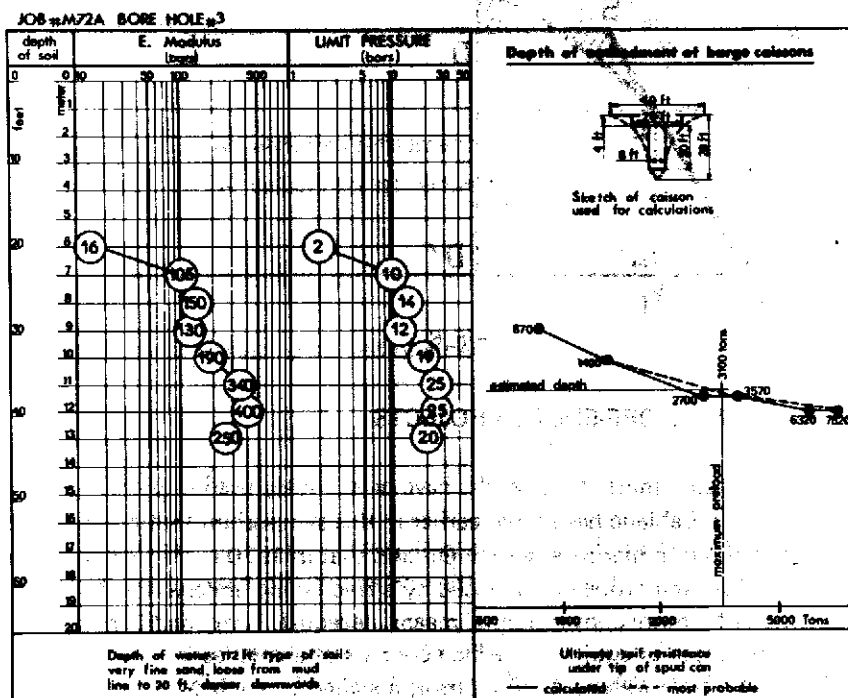


Fig. 7 Example of prediction of jack-up barge spud penetration, a good opportunity to check the validity of the Menard design rules.

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RULES OF FOUNDATION DESIGN USING SELF-BORING PRESSUREMETER TEST RESULTS

ABSTRACT

The results of tests carried out with the self-boring pressuremeter can be interpreted in accordance with the theories of pressiometric test in order to obtain elementary mechanical parameters : horizontal natural pressure, shear moduli, shear strengths, under undrained conditions (clay) or drained conditions (sand). These parameters can then be used in design models corresponding to various problems (foundations, retaining structures, unstable slopes, etc.).

Another approach is to use the test parameters directly. To this end, a reference test has been defined and semi-empirical rules of procedure have been established for design calculations of foundations : determination of punching pressure, settlement, and in the case of piles, shaft friction or reaction to horizontal efforts.

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RULES OF FOUNDATION DESIGN USING SELF-BORING PRESSUREMETER

A variety of types of test can be carried out with the self-boring pressuremeter : at different rates of expansion or of cyclic type for example, but for the purposes of ordinary foundation surveys, a so-called reference test has been defined ; it involves incremental monotonic loading and an imposed strain rate. See the communication of M. Jézéquel at this Symposium : "The Self-Boring Pressuremeter (SBP)". A reconnaissance program is generally organized, involving an extensive series of reference tests and a few special tests adapted to the problem and carried out at judiciously chosen locations. The test results can be applied of in two different ways :

1) They may be interpreted in accordance with pressiometric test theory so as to obtain elementary mechanical parameters : natural horizontal pressure p_0 , shear moduli G , or shear strengths. These parameters are then introduced into design models just like other soil parameters derived from other tests or measurements.

2) They may be introduced into semi-empirical design formulae, directly giving an evaluation of the parameters characterizing the behaviour of the structure. At the present time, such rules have been defined for foundations and for the reference test.

In what follows we present procedures for making use of the test results which are applicable to these two approaches, taking the reference test as the starting point for the sake of convenience, though the theory may be applied to a much wider range of tests.

1. INTERPRETATION OF THE PRESSUREMETER TEST AND DETERMINATION OF ELEMENTARY MECHANICAL PARAMETERS

We shall examine successively :

- the measurement of the natural horizontal pressure p_0 ,
- for fine soils, the determination of the undrained shear curve and moduli G_0 , and undrained cohesion c_u ,
- for granular soils (or where applicable, fine soils subjected to very slow special tests of the drained type), the determination of moduli and certain parameters of strain.

1.1. Natural horizontal pressure p_0

Ideally, this is the starting point of the pressio-metric curve (fig. 1).

However, experience shows that even if the self-boring procedure is carried out correctly, a certain disturbance of the soil is unavoidable, though it is true that it is in no way comparable in extent to that occurring with conventional procedures of inserting measuring instruments or core samplers in the soil.

To obtain an accurate measurement of p_0 it is necessary to wait for the soil to relax completely; this may take an hour or more.

In the reference test, the relaxation time is limited to 10 minutes. This gives an approximate value of p_0 , which is generally sufficient for ordinary purposes.

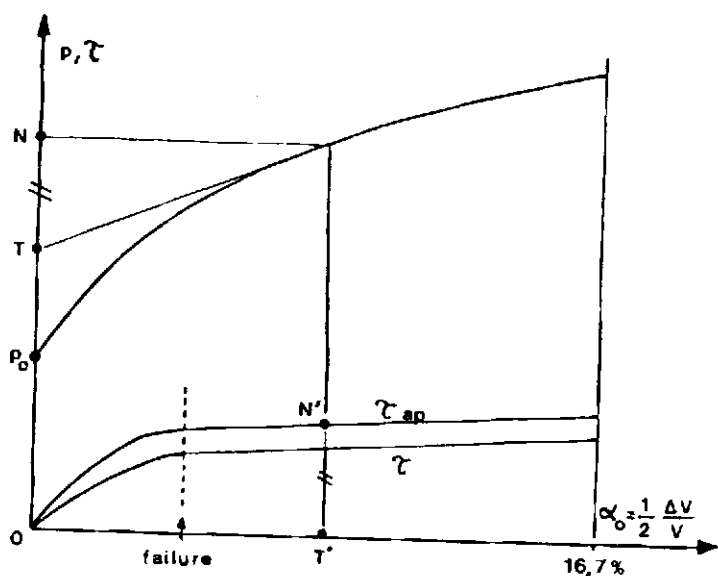


Figure 1 - Pressiometric curve and shear curves in sand.

RULES OF FOUNDATION DESIGN USING SELF-BORING PRESSUREMETER

1.2. Undrained test

The interpretation of an undrained pressuremeter test, that is to say a test in which the volume of soil does not vary, was formulated simultaneously by three groups of research workers : Baquelin, Jézéquel, Lemée and Le Méhauté (1972) in France ; Ladanyi (1972) in Canada and Palmer (1972) in Great Britain. We consider a homogeneous intact soil (constant initial pressure equal to p_0 ; unique shear curve).

Using the variable $\alpha_0 = \frac{1}{2} \frac{\Delta V}{V} (= \frac{1}{2} \frac{V-V_0}{V_0})$ we find (fig. 1) that the shear τ mobilized in contact with the probe is equal to the sub-tangent TM of the pressiometric curve (p, α_0) :

$$\tau = \alpha_0 \frac{dp}{d\alpha_0} \quad (1)$$

With the usual variable $x = \frac{\Delta V}{V_0}$ we find :

$$\tau = \frac{\Delta V}{V_0} \frac{dp}{d(\Delta V/V_0)} (1 + \frac{\Delta V}{V_0}) = x \frac{dp}{dx} (1+x) \quad (2)$$

With small strains ($x \ll 1$, $\alpha \approx x$) expression (2) becomes simpler and takes the form of (1), and the shear strain γ (diameter of the Mohr's circle of strain) is equal to :

$$\gamma = 2x \quad (3)$$

Consequently the curve (τ, x) or (τ, α_0) is the shear curve of the soil corresponding to the stress involved in the pressiometric test (horizontal shearing without rotation of principal stresses) ; hence it is possible to determine several characteristic parameters :

- $G_u = G_0$ initial undrained shear modulus
- c_u undrained cohesion, peak value
- x_f or α_f peak value of failure strain

- other values of undrained cohesion : after peak, or at large strain (for example for $\alpha_f = 10\%$ or 20%).

RULES OF FOUNDATION DESIGN USING SELF-BORING PRESSUREMETER

The theory of the undrained test is applicable to fine soils : clay, silt, subjected to rapid tests, and especially to the reference test.

It should be noted that special tests with measurement of pore pressure u generated in contact with the membrane can be performed (see for example Baguelin, Jézéquel and Le Méhauté, 1973 ; or Baguelin, Jézéquel and Shields, 1978). The test can then be interpreted in terms of effective stresses, and indications can be obtained as to the stress paths in a Mohr's diagram, for instance.

1.3. Drained test

A test is of this type when no pore pressure is generated ($u = 0$). This is the case when reference tests are performed in clean sand ; it may also occur in the case of slow or very slow tests performed in soils which contain a high proportion of fines, even clay or silt.

The theory of the drained test, to which several research workers have contributed, from Lamé (1852) up to the present day (see Baguelin, Jézéquel and Shields, 1978, for a complete presentation) provides the following principal results :

The first result is that the initial slope of the p-siometric curve ($p, x = \frac{\Delta V}{V_0}$) or ($p, \alpha_0 = \frac{1}{2} \frac{\Delta V}{V}$) is equal to $2 G_0$:

$$2 G_0 = \left(\frac{dp}{dx} \right)_{x=0} \quad (4)$$

Under small strains, this relation may be generalized:

$$2 G(x) = \frac{dp}{dx} \quad (5)$$

but it should be noted that the variable x does not exactly represent the shear strain γ , the diameter of the Mohr's circle of strain, because of the possible variation of volumetric strain μ ($x = \mu + \gamma$).

In fact, the complete interpretation of the test requires a knowledge of the volumetric strain μ induced during the test in the soil in contact with the membrane. Using the variable μ' equal to :

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$$\mu' = \frac{\mu}{1+\mu} \quad (6)$$

(in small strains $\mu' = \mu$) we find the following relation concerning the shear mobilized at the wall of the probe :

$$\tau = (\alpha_o - \mu') \frac{dp}{d\alpha_o} \quad (7)$$

If we call the apparent shear, τ_{ap} , the value :

$$\tau_{ap} = \alpha_o \frac{dp}{d\alpha_o} \quad (8)$$

which corresponds to the sub-tangent NT of the pressiometric curve (fig. 1), we see that the actual shear, τ , differs from τ_{ap} because of variations of volumetric strain μ' :

$$\frac{\tau}{\tau_{ap}} = \frac{1 - \mu'}{\alpha_o} \quad (9)$$

Conversely, if we know accurately, from triaxial tests for example, the parameters of failure c' and ϕ' , whether peak values or steady values, we can make use of relations (8) and (9) to evaluate the volumetric strain μ' attained either at peak or steadily ; the actual shear τ is obtained from c' and ϕ' , and the appropriate values of the pressure p , determined from the characteristic deformations of the curve (τ_{ap} , α_o), applying the relation :

$$\tau = \frac{1}{2} \left[1 - \operatorname{tg}^2 \left(\frac{\pi}{4} - \frac{\phi'}{2} \right) \right] \left(p + \frac{c'}{\operatorname{tg} \phi'} \right) \quad (10)$$

The value obtained can be compared with the value τ_{ap} in accordance with relation (9), and hence we get μ' . The method is however sensitive to uncertainties on c' and ϕ' . It can give an indication of the dilatant or loose nature of the sand.

The various parameters p_o , G_o or G_x , c_u , τ_{ap} or μ thus determined can be used, where necessary, along with other soil parameters obtained from other tests, in design calculation models. We may cite as a recent example a design calculation using the method of finite elements applied to a very high cast-in-situ quay wall recently built at Le

RULES OF FOUNDATION DESIGN USING SELF-BORING PRESSUREMETER

Havre (Blivet, Bonafous, Frank, Josseume, 1981) ; but applications in many other fields may be envisaged (other retaining structures, foundations, unstable slopes, etc.).

II. SEMI-EMPIRICAL USE OF THE REFERENCE TEST FOR THE DESIGN OF FOUNDATIONS

Firstly, let us recall (fig. 2) the usual parameters derived from the reference test (cf. the communication of J.F. Jézéquel at this Symposium : "The Self-Boring Pressuremeter (SBP)" :

- p_0 = initial pressure
- p_2 = pressure at 2 % strain ($x = \Delta V/V_0 = 2 \%$)
- p_5 = pressure at 5 % strain ($x = \Delta V/V_0 = 5 \%$)
- p_{20} = pressure at 20% strain ($x = \Delta V/V_0 = 20 \%$)
- G_{p2} = secant modulus at 2 % strain ($x = 2 \%$)
- G_{p5} = secant modulus at 5 % strain ($x = 5 \%$)
- $\beta = \frac{p_{20} - p_5}{p_{20} - p_0}$ = soil type index

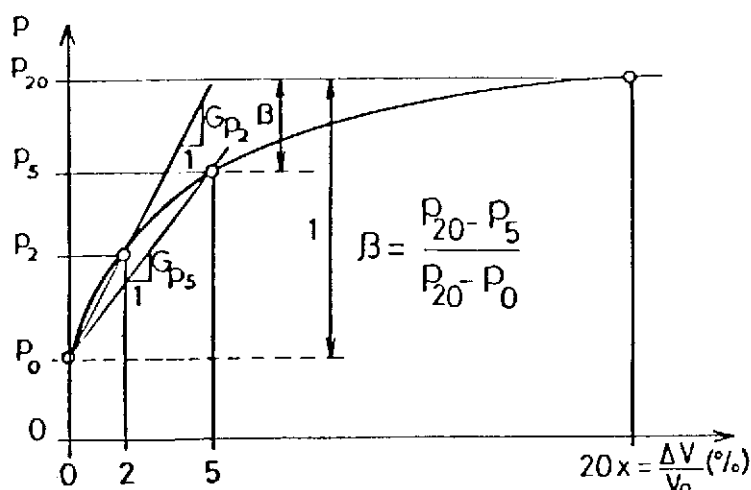


Figure 2 - Pressiometric reference test : pressiometric curve and characteristic parameters.

RULES OF FOUNDATION DESIGN USING SELF-BORING PRESSUREMETER

The parameter β reflects the more or less stretched nature of the pressiometric curve, and is related to the type of soil. The bend of the curves in clays is pronounced, and β is low ; the pressiometric curves of sands are on the contrary stretched, and β is high, the more so in proportion as the soil is frictional and compact (Baguelin, Jézéquel, Le Méhauté, 1979). The table (fig. 3) shows the ranges of characteristic values for different soils.

Soil Category	Range of β in %	Average β in %	Nature of soil
1	< 25		sensitive clay
2	25 < β < 40	32.5	clay
3	35 < β < 50	42.5	silt and compressible sand
4	45 < β < 50	52.5	dilatant sand
5	β > 60		very dilatant sand

Figure 3 - Characteristic values of the soil type index β .

Semi-empirical rules have been developed for the design of foundations subjected to vertical or horizontal loads.

2.1. Foundations subject to vertical loads

In a manner analogous to the pressiometric rules established by Ménard and his co-workers (Ménard, 1963 ; Centre d'Etudes Ménard, 1967) for the conventional test, we have developed semi-empirical rules for the calculation of the limit load of foundations (limit of pressure of punching at the base, and shaft friction), as well as for the calculation of the settlement of shallow foundations.

• Limit punching pressure at the base of foundations

We have :

$$q^*_1 = \Psi \cdot p^*_{20}$$

where :

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- q_{ℓ}^* = net punching pressure under the foundation base = $q_1 - q_0$
- q_{ℓ} = total punching pressure
- q_0 = overburden pressure at foundation level
- p_{20}^* = net pressure at 20 % strain in the reference pressiometric test = $p_{20} - p_0$
- Ψ = coefficient of bearing capacity, function of β and of the embedment of the foundation D_e/B , given by the chart (fig. 4) for strip footing.

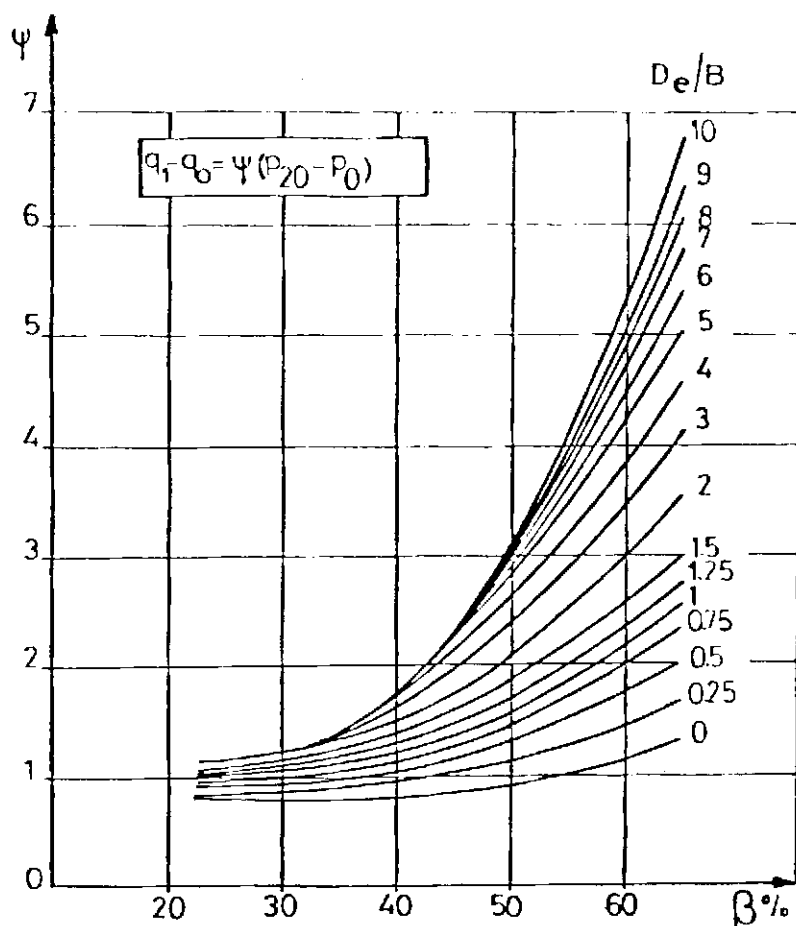


Figure 4- Chart giving the coefficient of bearing capacity Ψ for strip footing.

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This formula is established for an homogeneous soil. If the properties vary, we determine the characteristic values of p_{20}^* and D/B in accordance with rules analogous to those proposed by Ménard.

• Lateral friction of piles

We may apply the usual rules of estimation of the lateral friction f_l on the basis of the limit pressure p_l^* of the Ménard test, using the following approximate correspondence between p_l^* and p_{20}^* and β ; a relation valid only for this purpose :

$$p_l^* = p_{20}^* \left(\frac{5}{3} \beta + 0.5 \right)$$

Figure 5 is thus obtained from the chart given by Baguelin, Jézéquel and Shields, 1978.

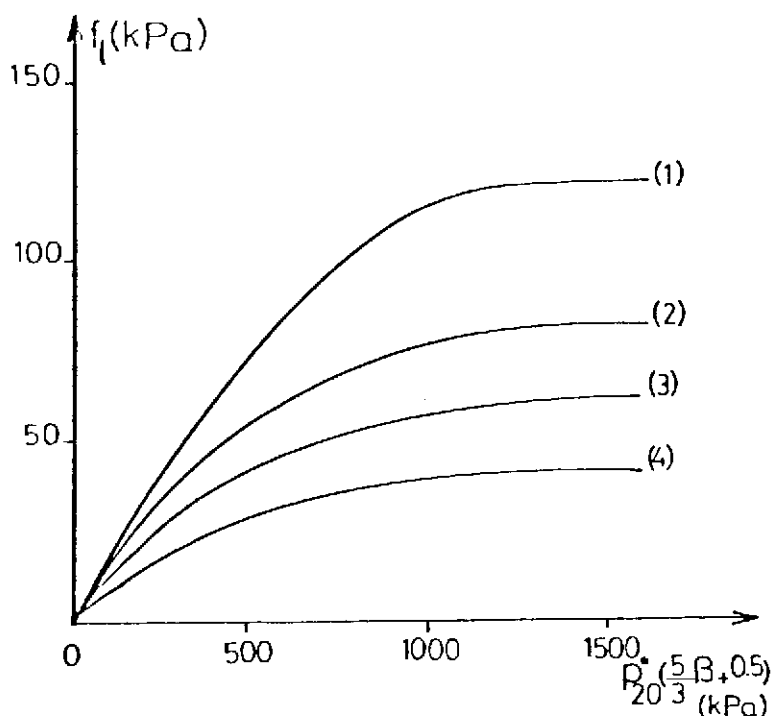


Figure 5 - Friction on the sides of a pile.
 (1) Concrete displacement piles in granular soil.
 (3) Steel displacement piles in cohesive soils.
 (4) Steel non-displacement piles in any soil.
 (2) Other soil and installation procedures.

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• Settlement

The following formula is given for long term settlement, s , of shallow foundations :

$$s = \frac{q^*}{10} \left[\frac{2B_o}{G_{p2}} \left(\lambda_1 \cdot \frac{B}{B_o} \right)^{(1-\beta)} + \lambda_2 \cdot \frac{B}{G_{p5}} \right] \quad B > B_o$$

where :

- s = final settlement of the foundation
- q^* = net pressure exerted by the foundation ($=q-q_o$)
- B = width of the foundation
- B_o = reference width = 0.60 m
- λ_1, λ_2 = shape coefficients, function of the ratio $\frac{L}{B}$ (length/width) of the foundation, given in figure 6.
- G_{p2}, G_{p5} = secant pressiometric moduli at 2 % and 5 % strain
- β = soil type index

L/B	1 foundation		2	3	5	20
	circular	square				
λ_1	1	1,12	1,53	1,78	2,14	2,65
λ_2	1	1,1	1,2	1,30	1,40	1,50

Figure 6 - Coefficients of form λ_1 et λ_2 .

According to the distinction introduced by Ménard, the first term represents the settlement due to deviatoric strains, and the second term the settlement due mainly to consolidation strains. When properties vary with depth, these terms are differently modulated, in accordance with the rules proposed by Ménard.

In fact, this formula is derived from the formula of

settlement of foundations on an elastic medium. Various corrections have been made to it, in the light of experience : non-linearity of the influence of the width B , which is especially sensitive in the case of sandy soils ; the choice of secant moduli, in particular the modulus $G_{0.5}$ makes it possible to allow approximately for the importance of volumetric strains, which are greater for clay than for sand.

In establishing these rules, account has been taken of experimental correlations observed between the parameters of conventional and self-boring pressiometric tests, together with observations of actual foundations.

2.2. Piles subjected to horizontal loads

It is customary to deal with these problems by using the concept of lateral reaction, according to which, at a given point along the length of the pile, the mean pressure p , or reaction, depends solely on the horizontal displacement, y , of the pile at that point.

A satisfactory similitude has been observed experimentally (Baguelin, Jézéquel, 1972 ; Baguelin, Jézéquel, Shields, 1978) between the pressiometric curve of the reference test and the reaction curve of a pile subjected to monotonic and relatively rapid horizontal loading (from a few minutes to a few hours).

More precisely, we may equate the following curves at a given depth (fig. 7).

- Reference pressiometric test :

$$\text{curve } p^*, x = \frac{1}{2} \Delta V/V_0$$

where $p^* = p - p_0$

- Lateral reaction : curve $(p, y/R)$

where p = reaction at the point considered

y = horizontal displacement of the pile at the point considered

R = radius of pile.

More complex loadings (repeated or permanent) increase the displacement y for a given level of reaction, but pro-

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bably also reduce the limit value of the reaction. These points are currently the subject of experimental research on actual piles. Provisionally, we may adopt the following simple and safe rules for repeated or permanent loadings (fig. 7) :

- confine the limit reaction pressure p_u to :
 - . 50 % of the pressure p_{20}^* for clay or silty soils ;
 - . 75 % of p_{20}^* for sandy soils.
- before this, double the displacements allowed for in the case of rapid monotonic loadings.

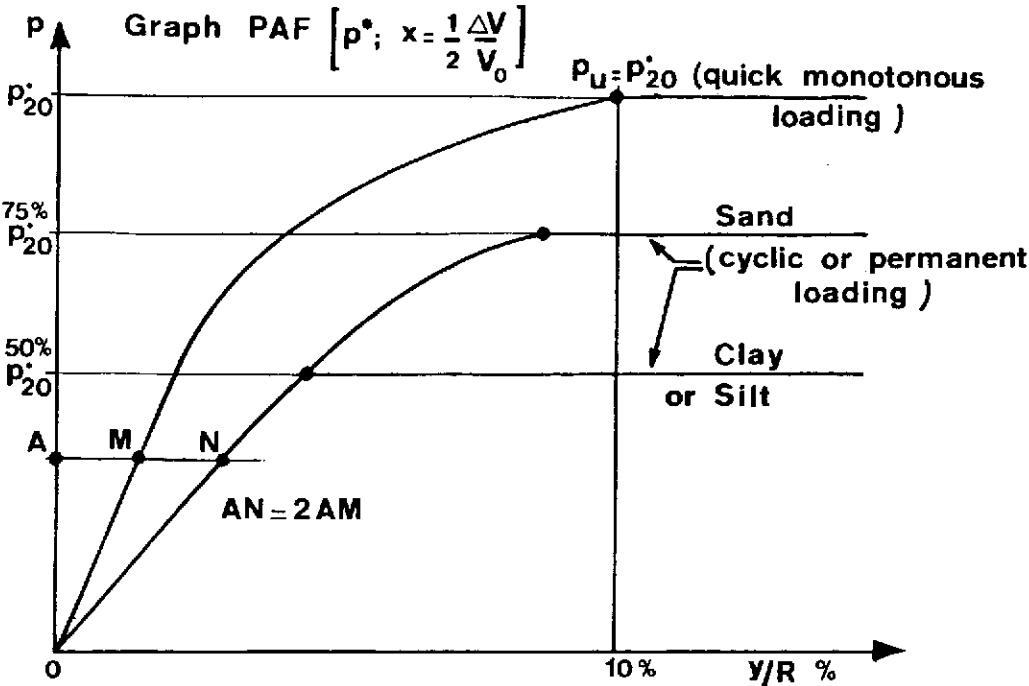


Figure 7 - Determination of lateral reaction curves $p, y/R$, of piles from the results of the pressiometric reference test.

CONCLUSION

The self-boring pressuremeter, supplying as it does curves of tests performed on an almost intact soil and hence lending themselves to interpretation in accordance with theory, can be used in soil reconnaissance to determine :

- either the elementary mechanical parameters to be introduced into design calculation models ;

- or standardized test parameters to be used in semi-empirical formulae directly giving the characteristic parameters of behaviour of structures ; this is done in the case of foundations at the present time.

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TENSION PILE DESIGN USING SELF-BORING PRESSUREMETER TEST RESULTS

ABSTRACT

This paper is intended to show that, in clayey soils, the Self-Boring Pressuremeter offers representative *in situ* measurements of the basic parameters governing the behavior of pilings subject to axial, static and cyclic loading. An examination of the results of an experiment on an instrumented pile driven into Cran clay helps to justify the calculation methods using local reaction curves recommended by various authors. In this example, the validity of the plotting of monotonic (t, z) curves from pressuremeter parameters is demonstrated. Several series of cyclic tests with the PAF76 are presented, showing the possibility of taking reliable measurements of cyclic shear moduli for clay, in order to design pilings subject to cyclic loads.

INTRODUCTION

1 With the steady increase in the water depth of petroleum production, the development of a complete engineering design method for offshore pilings based on the results of *in situ* tests appears more than ever necessary. On the one hand, we are witnessing changes in the types of structure installed. Floating structures stabilized by tension-leg anchoring methods will be required to play an increasingly important role. The anchoring point, generally consisting of piles or groups of piles, is subject to a combination of ascending vertical forces and horizontal forces of variable intensity, in which

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the cyclic component is predominant. Furthermore, the conventional methods of offshore geotechnique are unsuitable in this case. The use of empirical formulas and numerical models is faced with the problem of determining the representative parameters of the soil. Samples taken by drilling and wireline coring are increasingly disturbed with greater water depth. Conventional *in situ* tests (Cone Penetration Test, Vane Test) are unable to reveal the complexity of foundation mechanisms.

2 Since 1978, the Association de Recherche en Mécanique des Sols Marins, a French group of research institutions, oil companies and oil-related contractors, undertook a vast research program on the behavior of tension piles [1]. This program included two complementary aspects:

- static and cyclic loading tests on instrumented driven piles,
- the development of methods to forecast pile capacity under axial loads.

In the latter area, a special effort was made concerning the use of *in situ* test data obtained using the Self-Boring Pressuremeter.

3 This paper is intended to show that, at least in a clay medium, the Self-Boring Pressuremeter serves to perform representative *in situ* measurements of the basic parameters governing the behavior of piles subject to static and cyclic axial loading. It is based both on experimental data gathered on an instrumented pile, and on the results of a series of static and cyclic tests with the PAF76, conducted in the immediate vicinity of the pile in Cran clay. Three main aspects are stressed:

- soil/pile interaction mechanisms in a clay medium,
- plotting of t/z friction curves in monotonic axial loading,
- determination of representative cyclic friction moduli using the PAF76.

THE CRAN EXPERIMENT SITE

The Cran site, located in the Morbihan (France), on the edge of the Vilaine, is a testing site of the Laboratoires des Ponts et Chaussées (French Highways Department), consisting of fine clay sediments of average consistency already analyzed by thorough reconnaissance:

- laboratory tests (triaxial, consolidation measurements, etc.),
- statif penetrometer (CPT),
- vane probe,
- Ménard pressuremeter, PAF76 Self-Boring Pressuremeter, Offshore Self-Boring Pressuremeter (PAM) [2].

In the total clay thickness of 17 m, four layers may be distinguished:

- 0 to 2 m: a weathered surface layer, highly overconsolidated,
- 2 to 8 m: a layer of soft clay of which the cohesion varies linearly with depth,
- 8 to 11m: a stiffer layer on the average, but rather heterogeneous,
- 11 to 17m: a very homogeneous layer.

The main geotechnical characteristics of the Cran site are shown in Figure 1. Note the high value of the plasticity index ($PI = 80$), slight apparent overconsolidation ($1.5 < OCR < 2$), and the low values of shear strength ($C_u \leq 60$ kPa).

THE CRAN EXPERIMENTAL PILE

Pile and instrumentation

The Cran pile is a metal tubular pile, closed at the base, 10 3/4 inches in diameter and 17 m long. It is instrumented along its entire height with strain gauges and measurement cells for total stresses and pore pressures. Three piezometric columns were also installed in the surrounding formation at different distances from the pile. For further details concerning the instrumentation employed and the operating procedures, see the previous article by Puech and Jézéquel [3].

Test program

The pile was installed by driving and monitored throughout the re-consolidation period. Several series of pullout tests were then conducted:

- static loadings in intervals of one hour consisting of reference tests (type S),

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- long-term cyclic loadings ($N \geq 1000$) between two constant values of the head load (type C),

- so-called 'storm' cyclic loadings between two variable head load values, of which the mean is constant (type T).

The cycle period was 14 seconds.

All the results obtained in the pullout tests are confidential. We shall restrict ourselves to furnishing the information necessary to illustrate the thesis of this paper.

Soil/pile interaction

An understanding of the basic mechanisms of the pile in lateral friction necessarily implies the study of the laws of lateral friction mobilization. For the static test S3, Figure 2 shows the lateral friction mobilization curves at different depths. It may be observed that maximum lateral friction is mobilized in each soil layer or sub-layer for the same value of local pile movement. The value of this maximum friction depends directly on the shear strength of each clay layer.

In addition, the variations in radial stresses measured on the pile shaft during pullout remained very low in absolute value, whatever the loading mode. As an illustration, the variations in pore pressures and total stresses are given, measured at 5.30 m below the surface in two static and cyclic tests (Fig. 3). The ratio of radial over-stresses to tangential stress along the pile is generally under 0.5.

The mobilization of lateral friction on the pile thus seems to be a pure shear mechanism along the shaft. The absence of any change in radial stresses shows that the overall formation reacts very little to the pile loading. The mobilization of lateral friction appears to be an essentially local mechanism, not affecting the bulk of the soil. This result, which is basic for its extremely important practical implications, was suggested by different authors, especially Baguelin and Frank, on the basis of calculations by finite elements for model piles in media displaying simple rheological behavior [4]. It provides a justification for the axial capacity design methods of piles in a cohesive medium based exclusively on the knowledge of the local friction/movement relationships, called the t/z curves. These (t/z) curves can also be determined simply from the local characteristics of the foundation soil.

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DESIGN PRINCIPLE FOR AXIALLY-LOADED PILES USING SELF-BORING PRESSUREMETER TEST RESULTS

The curves of lateral friction mobilization along the pile, like the experimental curves presented in Figure 2, appear to be essentially bilinear, with a rising linear portion characterized by the value of the friction modulus P_0 , and a friction level characterized by the limit value F_L .

Baguelin and Frank [4] showed that modulus P_0 varies linearly with the soil shear modulus G , i.e. that the slope at the origin of the friction curve (F , V/R) is G/k . The coefficient k essentially depends on the pile's slenderness ratio ($l/2R$) and the Poisson coefficient of the soil. Theoretical values have been suggested by various authors, including Orsi (1978), Poulos-Davis (1968) and Randolph (1971). The non-linearities of the shear curve may be taken into account. However, experience shows that the resulting non-linearities in the friction curves are considerably attenuated and can be ignored.

The self-boring pressuremeter is a reconnaissance tool for determining the *in situ* parameters required to plot the friction curves:

- pressuremeter shear moduli,
- extreme lateral friction related to shear strength and pile surface condition.

Monotonic t/z curves

A large number of standard pressuremeter tests were conducted in Cran clay, particularly in the vicinity of the pile.

Table 1 gives the values of the moduli P_0 obtained in static test S3 for each section of the pile considered. These values are compared to the means of the pressuremeter moduli G_0 (initial tangent modulus) measured at the level considered in the standard tests. The ratio $k = G_0/P_0$ ranges from 2 to 4.1. The range of theoretical values lies between 2.5 and 5 for slenderness ratios between 0 and 50 [4].

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Table I
Comparison of friction moduli and pressuremeter moduli

Level (m)	Pile		PAF76	G_0/P_0
	P_0 (kPa)	$\frac{L}{2R}$	G_0 (kPa)	
2 to 4	550	11	2240	4.1
4 to 6	1520	18	3980	2.6
6 to 8	2270	26	5680	2.9
8 to 11	3890	37	7920	2.0
11 to 17	3280	51	12120	3.7

Table 2 shows a comparison of the values of the ratio α between the real extreme friction observed on the Cran pile, and the shear strengths given by the PAF76 and the vane probe. For the Self-Boring Pressuremeter, the value of shear strength is determined from the curves derived from the expansion curves. This means that the coefficients α thus determined lie around 0.5. This value is substantially lower than the values suggested by different authors on the basis of laboratory tests (Tomlinson, Caquot, API Recommendations).

Table II
Pile adhesion coefficient

Level (m)	Pile	PAF76		Vane	
	F_L (kPa)	τ_f (kPa)	α	C_u (kPa)	α
2 to 4	5	16	0.31	18	0.28
4 to 6	10.5	22	0.48	26	0.40
6 to 8	13	29	0.45	37	0.35
8 to 11	21.5	34.5	0.62	52	0.41
11 to 17	24	51	0.47	48.5	0.49

CYCLIC TESTS USING THE SELF-BORING PRESSUREMETER

The capacity of the PAF76 Self-Boring Pressuremeter to perform *in situ* cyclic soil loading tests is an accomplished fact today. Further details may be obtained from the article by Jézéquel and Le Méhauté [5] at this Symposium.

The technology of the instrument, and in particular its so-called controlled deformation pressure/volume controller' dilatation system are ideal for the performance of cyclic tests:

- accurate measurement of pressures in the cell,
- control of cell deformation, even at very low values,
- possibility of complete automation of the system necessary for the performance of a large number of cycles.

The Offshore Self-Boring Pressuremeter (PAM), specially developed by IFP for offshore soil investigation, is equipped with a probe derived from the LPC probes, and thus offers the same advantages and possibilities as the PAF76. Its adaptation for the performance of cyclic pressuremeter tests like those described here is currently under way.

Cyclic test procedures

The measurement circuit and the performance of cyclic tests are described in [5]. The test procedures were determined to simulate shear mechanisms as accurately as possible at the soil/pile interface in service conditions. At the base of the pile, the average mobilized friction and its cyclic amplitude are low; in the middle, the average friction and cyclic amplitude are high, but maximum friction does not exceed limit friction; at the head of the pile, the pile movements may be greater than the movement corresponding to the mobilization of limit friction.

Consequently, except near the head, the deformation rates $\dot{\gamma}/R$ mobilized along the pile remain low. To be representative, simulation pressuremeter tests must be located within a range of pressures between p_0 , the soil pressure at rest, and p_f , the soil failure pressure corresponding to the shear strength τ_f (Fig. 4). Hence the cyclic deformation rates imposed on the soil by the pressuremeter cell must remain very low ($\Delta V/V_0 < 2\%$). The observation of this condition requires perfect control over the volume variations ΔV per cycle (a few cm^3 for low amplitude cycles), and is one of the major difficulties of this type of test.

The following pages discuss two types of test:

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- cyclic loading tests between two pressure limits P_M and p_m (type C tests),
- loading tests between two variable pressure limits, of which the mean is constant (type T tests, Fig. 4).

In all the tests, the mean pressure in the pressuremeter cell is equal to the pressure obtained for a monotonic volume dilatation of 0.5%. The volume deformation rate is 2% per minute.

Definition of parameters

The parameters involved are essentially the following (Fig.4 and 5):

- pressure p_f corresponding to complete mobilization of shear strength τ ,
- soil loading rate, expressed by the ratio $(p-p_0)/(p_f-p_0)$, where p is the maximum cyclic pressure in the pressuremeter cell for a given series of cycles,
- secant moduli G_M and G_m , corresponding respectively to the maximum and minimum pressure points for each cycle,
- cyclic modulus G_c relative to the N^{th} order cycle.

Cran cyclic tests

The characteristics of the type C or T tests conducted on the Cran site in the immediate vicinity of the experimental pile are summarized in Table III. Also shown are the characteristics of the monotonic reference tests. The number of cycles (N) applied is generally 300. The cycle period depends on the pressure amplitude and soil rigidity. Periods approaching one minute are obtained.

Table III
Characteristics of cyclic tests performed at Cran

z (m)	Test type	Cyclic test				N	Reference test	
		P_0 (kPa)	P_I (kPa)	P_{II} (kPa)	P_{III} (kPa)		P_0 (kPa)	P_f (kPa)
8	C	128	192			210	127	194
8	T	133	165	173	181	300	"	"
10	T	151	214	229	245	350	151	208
12	C	188	282			300	218	325
12	T	185	250	266	282	350	"	"
14	T	212	269	283	297	350	245	341
15							221	348
16	T	197	260	275	291	350		

Figure 6 shows the pressure/volume curve obtained in real time on the plotter of the measurement circuit in the type T test, performed at a depth of 8 m. Note the perfect control of the pressures P_M and P_m in each series of cycles, for very slight variations in these pressures in absolute value (a few kPa). Figures 7 and 8 illustrate the types of result obtained during this test run, with type C and T tests performed at a depth of 12 m.

The type C tests (Fig. 7) shown that:

- the volume deformation of the cell was not stabilized after a few hundred cycles,
- the secant modulus G_M varies little after 100 cycles,
- the cyclic modulus G_C is stabilized after a few dozen cycles.

The type T tests help to determine the variation in the moduli as a function of the different pressure levels. The following are recorded in these tests (Fig. 8):

- an increase in the volume deformation rate with increased pressure p_M (first part of test),
- a continuous decrease in the secant modulus G_M , even in the load removal phase (first and second parts of test),
- rapid stabilization of the cyclic modulus G_C , whatever the loading level,
- a significant decrease in the cyclic modulus G_C as the pressure p_M increases.

The variation in the stabilized cyclic modulus G_C as a function of shear level is shown in Figure 9 for all tests performed at the different depths.

PRESSUREMETER MODULI AND CYCLIC FRICTION MODULI

The cyclic pullout tests conducted on the Cran pile served to determine the friction moduli (Fig. 10). P_M and P_m are the secant moduli corresponding respectively to the maximum and minimum load points for each cycle. P_C is the cyclic modulus relative to the N^{th} cycle.

Figure 11 shows the variation in friction moduli recorded on the lower section of the pile in the storm test T2.2. The following may be noted:

- rapid stabilization of the moduli for each load increment,
- decrease in the cyclic modulus P_C with the load applied.

Figure 12, which shows the decrease in the friction modulus P_c as a function of mobilized friction rate (expressed by the ratio of maximum mobilized cyclic friction to limit friction measured in test S3: $F_{cyc}/F_{stat.lim.}$ should be compared with Figure 9, giving the variation in the cyclic pressuremeter modulus G_c with the soil loading rate. The ratio $k_c = G_c/P_c$ decreases from 2 to 1.5 as the friction rate increases from 0.5 to 1.

CONCLUSIONS

- 1 Static and cyclic loading tests conducted on a densely instrumented experimental pile driven into Cran clay showed that the mobilization of lateral friction in a clay medium is a local mechanism.
- 2 These observations serve to justify the axial capacity design approaches for piling based on the concept of local soil/pile interaction curves (t/z curves). These methods require knowledge of the shear curves of the material along the entire height of the pile.
- 3 The PAF76 Self-Boring Pressuremeter serves to obtain these curves by direct derivation of the expansion curve. The procedure for determining the monotonic t/z curves suggested by Baguelin *et al* from the parameters measured in the standard test (shear strength and initial tangent pressuremeter modulus) was compared with the experimental results, with a highly satisfactory degree of confidence.
- 4 Several series of cyclic tests conducted with the PAF76 in Cran clay, following the procedures described, showed that the Self-Boring Pressuremeter provided reliable and representative *in situ* measurements of the cyclic shear moduli of the material. The use of these data to forecast the behavior of piles subject to cyclic axial loads appears to be highly promising.
- 5 The Self-Boring Pressuremeter is emerging as the only tool capable of providing an *in situ* measurement of the static and cyclic parameters governing the behavior of piles subjected to lateral and axial forces. These parameters can be used directly in routine design engineering methods based on the use of local soil reaction curves (p/y and t/z curves). The short-term development of Offshore Self-Boring Pressuremeter tests should help to achieve considerable progress in offshore foundation design methods.

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The *in situ* pile tests and cyclic tests using the PAF76 were conducted by the Laboratoire des Ponts et Chaussées in Saint-Brieuc. The authors wish to emphasize the high quality of the work accomplished and the active participation of Mr. Jézéquel and Mr Le Méhauté in setting up the test programs.

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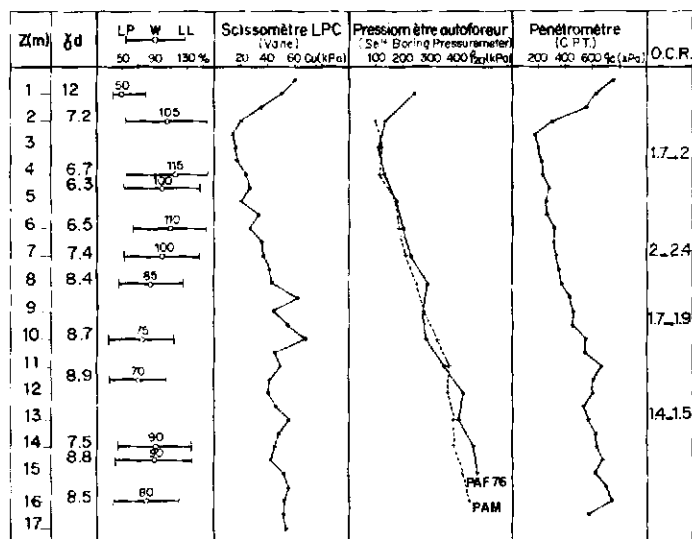


Figure 1

Geotechnical characteristics of Cran clay

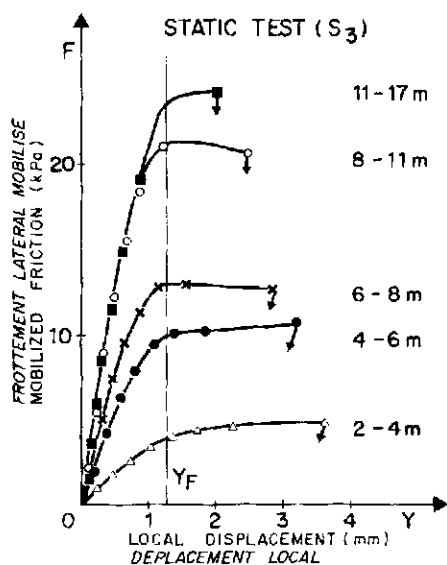


Figure 2

Pile lateral friction mobilization curves

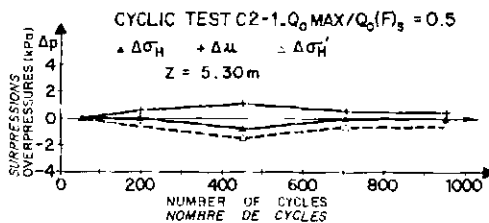
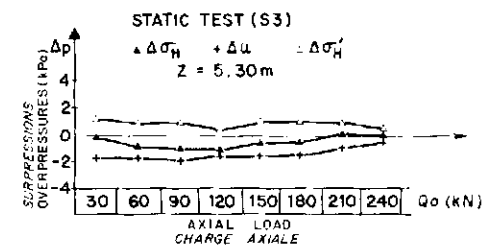


Figure 3

Overstresses measured on pile during two loading tests

TENSION PILE DESIGN USING SELF-BORING PRESSUREMETER TEST RESULTS

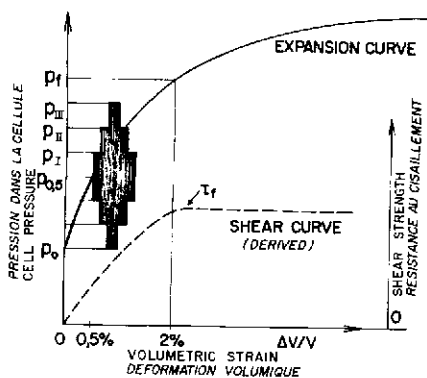


Figure 4

Schema of a type T cyclic
pressuremeter test

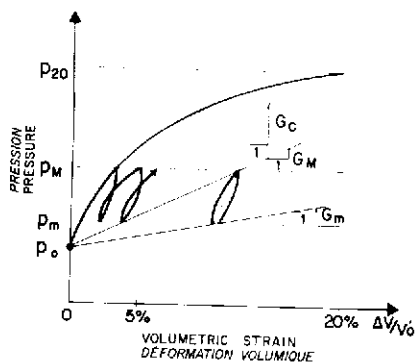


Figure 5

Definition of cyclic pressuremeter parameters (type C test)

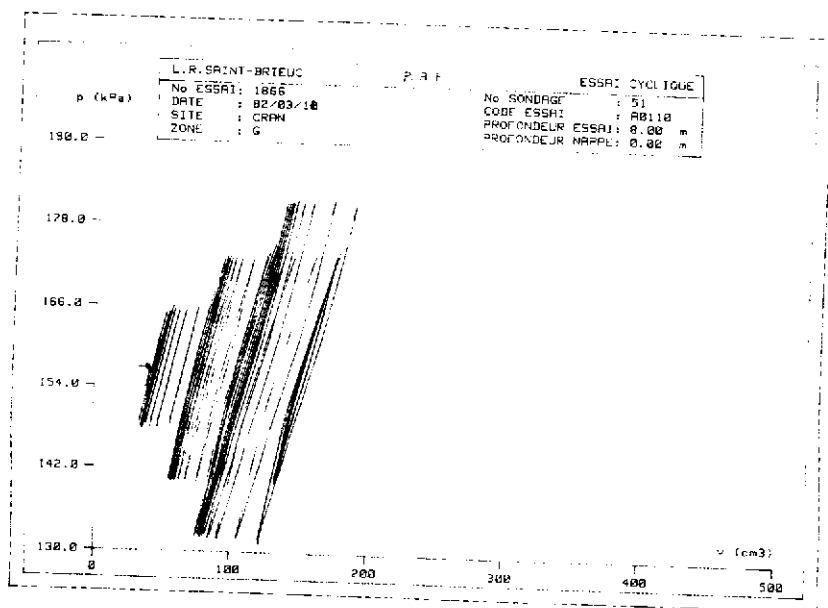


Figure 6

Cyclic pressuremeter test: example of a pressure/volume curve plotted in real time

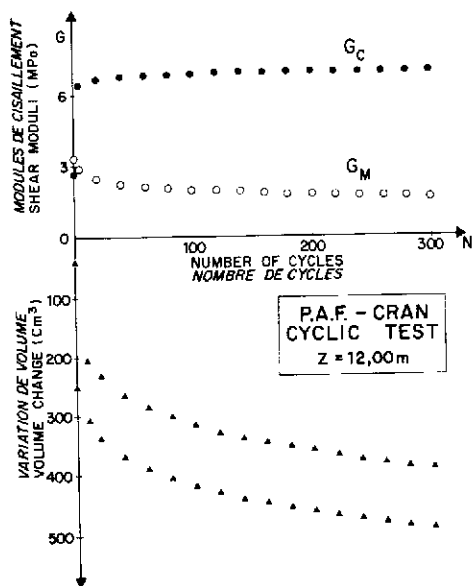


Figure 7

Type C cyclic pressuremeter test:
variation in parameters as a
function of cycles

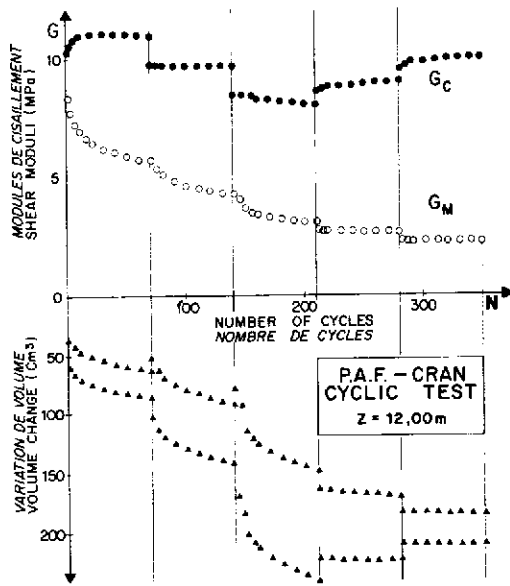


Figure 8

Type T cyclic pressuremeter test:
variation in parameters as a
function of cycles

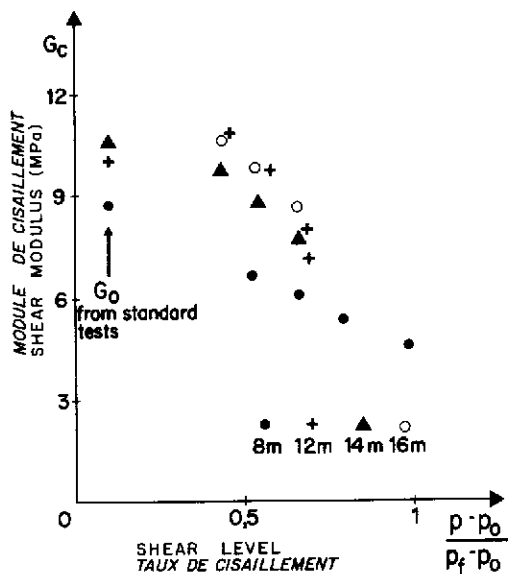


Figure 9

Variation in the cyclic pressuremeter
modulus as a function of shear level

TENSION PILE DESIGN USING SELF-BORING PRESSUREMETER TEST RESULTS

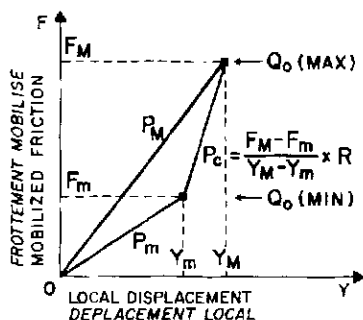


Figure 10
Definition of friction
moduli on pile

Figure 11
Cyclic loading test
on pile: variation
in friction moduli

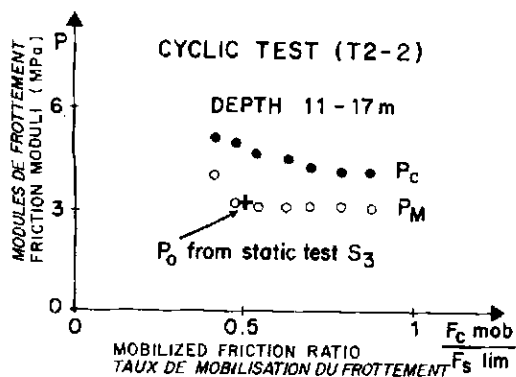
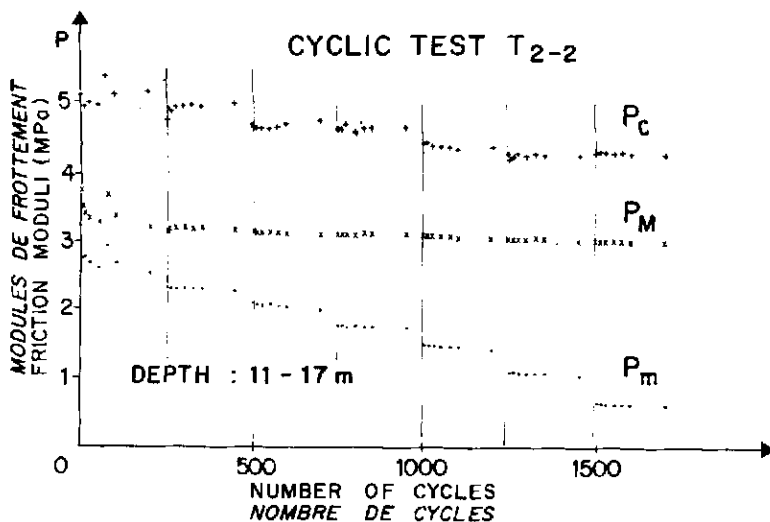


Figure 12
Variation in the cyclic friction
modulus as a function of
mobilized friction ratio

DESIGN OF Laterally Loaded PILES Using Pressuremeter Test Results

ABSTRACT

The possibility of using the pressuremeter test curve as a p - y curve for input in the finite difference program in order to predict the behavior of laterally loaded piles is investigated. The evaluation is made by using elasticity and plasticity theory, laboratory model tests in clay and in sand, full scale pile load tests at three stiff clay sites, two soft clay sites and one sand site. It is found that this method leads to results which are conservative if the pressuremeter is inserted in a hand augered hole and unconservative if soil displacement takes place during the probe insertion. It is shown that two curves are necessary to describe the behavior of a pile element: a friction-lateral displacement curve (f - y) and a front resistance-lateral displacement curve (q - y). The pressuremeter gives a good measure of the q - y curve and may provide elements of data necessary to obtain the f - y curve.

I. THE PROBLEM

Since 1950 about 1500 platforms have been built in the Gulf of Mexico in water depths varying from 0 to 300 m (1100 ft) of water. Most of them are of the steel jacket type with pile foundations. Approximately 50 new platforms are built every year; the cost of a platform varies greatly but probably averages about 10M\$. This makes the construction of offshore platforms in the Gulf of Mexico a 1M\$/day business.

The Mississippi Delta is one of the most prolific oil producing areas; the soil conditions are very poor and pile foundations are a must. The major lateral loads on the platforms are due to the wind, the waves, the currents and sometimes the mud flows; in most cases

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the lateral loads are about 50% of the vertical loads. A recently built mudslide resistant platform in 100 m (300 ft) of water had piles designed to resist each 2100 tons vertically and 1250 tons laterally.

The problem is to predict the lateral deflection at the mudline that the piles will experience at working loads and under extreme storm conditions as well as to predict the maximum bending moment in the pile. Tolerable deflections are of the order of 10% of the pile diameter.

II. THE SOLUTIONS

Many solutions to the above problem have been proposed; some predict the ultimate lateral load that the soil can resist, some predict the deflections under small lateral loads, some predict the full lateral load-lateral deflection curve for the pile.

Among the solutions used to predict the ultimate lateral load that the soil can resist are Hansen's method (11), Brom's method (5), Bierschwale, et al. method (3). Hansen's method is based on the theory of plasticity and is developed for rigid piles; Brom assumes a soil pressure distribution at failure and presents a method for both rigid and flexible piles, based on their measurements Bierschwale, et al. present an empirical procedure for rigid piles in clay; they also list and evaluate a number of other methods.

Among the solutions used to predict the deflections under small lateral loads are the subgrade reaction method (17), Brom's method (15) and the elasticity approach (17). The subgrade reaction method models the soil as a discrete number of springs; Brom presents a particular version of the subgrade reaction approach; the elasticity approach has the advantage of treating the soil as a continuous medium. Comparison of the subgrade reaction and elasticity method in one instance (17) showed that the subgrade reaction method predicts larger deflections, larger maximum moments and that the difference increases as the pile becomes more flexible.

Among the methods used to predict the complete load-deflection curve for the pile are the finite element method and the subgrade reaction method both using nonlinear model. The three dimensional finite element analysis (20) presents a number of advantages over the subgrade reaction method: it allows for realistic three-dimensional effects, computation of stresses and deformations in the soil and three-dimensional constitutive soil behavior. At present however the three-dimensional finite element method is not widely used because of the considerable human and computer effort required.

The nonlinear subgrade reaction method is widely used and

DESIGN OF Laterally Loaded PILES USING PRESSUREMETER

represents the current practice for offshore platforms. It consists of solving the governing differential equation by the finite difference method (20):

$$E_p I \frac{d^4 y}{dz^4} + Q \frac{d^2 y}{dz^2} + P = 0 \quad (1)$$

where $E_p I$ is the flexural stiffness of the pile, y the deflection, z the depth, Q is the axial load, and P is the resisting force per unit of pile length developed by the soil (fig. 1). The average pressure p is defined as:

$$p = \frac{P}{B}$$

where B is the width of the pile or later on the width of the pressuremeter. The parameter P depends upon z and y ; this led to the concept of the p - y curves which exist at various depths z . One of the major problems encountered with any of the above methods is the characterization of soil behavior; for the nonlinear subgrade reaction method, p - y curve criteria have been developed for soft clay (15), stiff clay above the water table (21), stiff clay below the water table (19), and sand (18). These criteria are presented and evaluated by Meyer (16). They are essentially based on empirical correlations with the results of lateral load tests on instrumented piles; the soil parameters necessary to develop the entire nonlinear p - y curves are: the angle of internal friction ϕ and a modulus of subgrade reaction k for the sand criterion, the undrained shear strength c_u and the strain at one-half the ultimate principal stress difference ϵ_{50} for the clays.

In current practice, ϕ is obtained from "in house" correlations with the driving resistance during sampling (blows per foot), k is chosen from three recommended values, c_u is obtained from unconsolidated undrained triaxial tests on samples or sometimes from in situ vane tests, ϵ_{50} is obtained from the same unconsolidated undrained tests or chosen from three recommended values. Meyer (16) showed that the predicted pile behavior is most sensitive to ϕ and c_u . Underestimating c_u by 50% may lead to an increase in maximum moment of 100% and an increase in deflection of 200%. Denk et al. (7) have shown that stress relief on some gassy marine sediments may decrease drastically c_u and the soil stiffness.

III. THE PRESSUREMETER APPROACH

There is a definite analogy of loading between the pressuremeter test (PMT) and a laterally loaded pile (fig. 1); this is why the PMT appears to be ideally suited to characterize soil behavior in this

case. There are two ways of using PMT results for the design of laterally loaded piles: the first one consists of using the PMT curve to obtain a modulus of horizontal subgrade reaction, the second one consists of using the entire PMT curve as a p-y curve.

The Menard-Gambin method (2,10) makes use of a modulus of subgrade reaction k:

$$\frac{1}{k} = \frac{y}{p} = \frac{1.33 R_0}{3E} \left(\frac{R}{R_0} \times 2.65 \right)^\alpha + \frac{\alpha R}{3E} \quad \text{for } R \geq R_0 \quad (2)$$

$$\frac{1}{k} = \frac{y}{p} = \left(\frac{1.33}{3E} (2.65)^\alpha + \frac{\alpha}{3E} \right) \times R \quad \text{for } R < R_0 \quad (3)$$

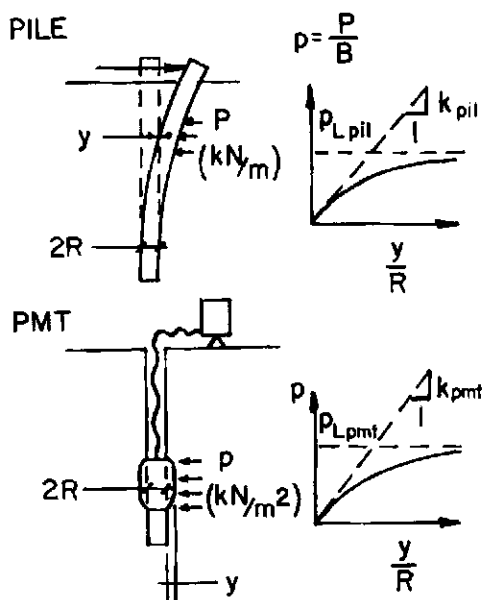


Figure 1: The Analogy

where E is the pressuremeter modulus, R_0 is a reference radius of 0.30 m, R is the pile radius, α is a soil structure factor varying from 1 for overconsolidated clays to 1/4 for sand and gravel. The p-y curve is then represented by an elastic plastic model (fig. 2). This method has been used with success (10) and has at least one advantage: it does not require the use of a computer if the soil is not stressed beyond the yield criterion and if the soil can be modelled by either a constant k value or linearly increasing k value with depth.

The second method consists of using the entire pressuremeter curve as a p-y curve (fig. 1) and will be called the pressuremeter p-y curve method. In this case p is the pressure applied to the soil by the probe and y is the

increase in radius of the soil cavity. This method makes complete use of the measured data; however it is probable that the pressuremeter p-y curve is different from the pile p-y curve because of the somewhat different modes of loading (2). The remainder of this article focuses on the evaluation of this method using elasticity and plasticity theory, laboratory model tests in sand and in clay, and full scale pile load tests at six different sites in Texas.

IV. THEORETICAL CONSIDERATIONS

A. ELASTICITY

According to the theory of linear elasticity the p - y/R curve would be a straight line; the slope of this straight line will be k_{pil} for the pile p - y/R curve (fig. 1) and k_{pmt} for the pressuremeter p - y/R curve. The values of k_{pil} and k_{pmt} can be calculated using elasticity theory and then the ratio k_{pil}/k_{pmt} can be obtained.

The expression of k_{pmt} is (2):

$$k_{pmt} = \frac{P}{\frac{y}{R}} = \frac{E}{1+\nu} \quad (4)$$

where E is the elasticity modulus and ν is Poisson's ratio. This expression assumes a plane strain condition around an infinitely long expanding cylinder.

Several expressions of k_{pil} can be obtained. Using the derivation published by Baguelin et al. (1) a first expression is obtained:

$$k_{pil} = \frac{E}{1+\nu} \times \frac{4\pi(1-\nu)}{\left[(3-4\nu)\ln\left[\frac{R_0}{R}\right] - \frac{2}{3-4\nu}\right]} \quad (5)$$

This expression assumes a plane strain condition around an indefinitely long cylinder of radius R moving laterally; it is also assumed that the pile adheres perfectly to the soil and that a fixed boundary exists at a radius R_0 from the pile. With $R_0 = 10R$ and for a Poisson's ratio of 0.33 the ratio k_{pil}/k_{pmt} is 1.29.

Douglas and Davis (8) integrated Mindlin's equation in order to obtain the displacements of a rectangular plate buried vertically and moving laterally. Using these results another expression of k_{pil} is obtained:

$$k_{pil} = \frac{E}{1+\nu} \times \frac{8\pi(1-\nu)}{[(3-4\nu)F_1 + F_2 + 4(1-\nu)(1-2\nu)F_3]} \quad (6)$$

where F_1 , F_2 , F_3 are influence factors given by charts (8). For a deeply buried rectangular plate with a length to width ratio of 10 and for a Poisson's ratio of 0.33, the value of k_{pil}/k_{pmt} is 1.25.

Imai (13) used an approximate solution to the problem in the case of linear elasticity and found:

$$k_{pil} = \frac{E}{(1+\nu)} \times \frac{\pi(3+2\nu)}{8} \quad (7)$$

For a Poisson's ratio of 0.33, the ratio k_{pil}/k_{pmt} is 1.44.

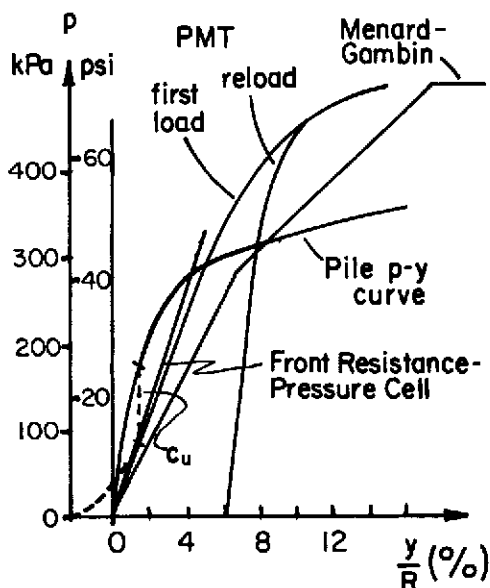


Figure 2: Field p-y curves

A finite element program was used to determine k_{pil} . The soil was linear elastic with a Poisson's ratio of 0.33 and was represented by a thick wall cylinder with an outside diameter equal to 10 times the pile diameter; this outside boundary was fixed and full pile-soil adherence was assumed. This analysis led to a ratio k_{pil}/k_{pmt} of 1.50.

Yet another way of estimating k_{pil}/k_{pmt} is to use the k of the Menard-Gambin method given in equations 2 and 3. Assuming a Poisson's ratio of 0.33, the following ratios were obtained:

for $2R = 0.30$ m and $\alpha = 1$

$$k_{pil}/k_{pmt} = 0.88$$

for $2R = 0.30$ m and $\alpha = \frac{1}{3}$

$$k_{pil}/k_{pmt} = 1.83$$

for $2R = 1.20$ m and $\alpha = 1$ $k_{pil}/k_{pmt} = 0.88$

for $2R = 1.30$ m and $\alpha = \frac{1}{3}$ $k_{pil}/k_{pmt} = 2.68$

It appears from all the above results that the slope k_{pil} is generally steeper than the slope k_{pmt} ; the ratio k_{pil}/k_{pmt} varies from 0.88 to 2.68 and averages 1.50.

B. PLASTICITY

The plasticity theory can be used to study the values of the pressuremeter limit pressure p_{Lpmt} (fig. 1) and the pile limit pressure p_{Lpil} ; then the ratio p_{Lpil}/p_{Lpmt} can be evaluated.

In the case of the undrained behavior of cohesive soils there

DESIGN OF Laterally Loaded PILES USING PRESSUREMETER

are several ways of estimating p_{Lpil} : Marti (14) uses a simple slip line analysis to find theoretically and confirm through laboratory experiments that:

$$p_{Lpil} = 11.42 c_u \quad (8)$$

where c_u is the undrained shear strength. For a deeply buried strip footing Hansen (11) finds:

$$p_{Lpil} = 8.0 c_u \quad (9)$$

In the p-y criterion currently used in practice p_{Lpil} at large depth is taken as

$$p_{Lpil} = 9 c_u \quad (10)$$

In the same case of undrained behavior, p_{Lpmt} is

$$p_{Lpmt} = c_u \left(1 + \ln \frac{G}{c_u} \right) \quad (11)$$

where G is the soil shear modulus. For a reasonable value of G/c_u equal to 20, p_{Lpmt} is equal to $4c_u$ and the ratio p_{Lpil}/p_{Lpmt} varies between 2 and 2.85.

Another way to estimate this ratio is to use the bearing capacity factor developed for the foundation design rules based on pressuremeter test results (2); p_{Lpil} is associated to the bearing capacity of a deeply buried strip footing.

$$\text{In this case } \frac{p_{Lpil}}{p_{Lpmt}} = 1.4 \text{ for soft clay}$$

$$\text{and } \frac{p_{Lpil}}{p_{Lpmt}} = 2.1 \text{ for stiff clay}$$

In the case of a cohesionless soil, and assuming no volume change, p_{Lpmt} is expressed as (2):

$$p_{Lpmt} = k_o \sigma'_v (1 + \sin \phi) \left(\frac{G}{s} \times \frac{1}{\cos \phi} \right)^{\frac{1-k_a}{2}} \quad (12)$$

where k_o is the coefficient of earth pressure at rest, σ'_v the effective vertical stress at the testing depth, ϕ the angle of

DESIGN OF Laterally Loaded PILES USING PRESSUREMETER

internal friction, s is the soil shear strength ($s = k_a \sigma'_v \tan \phi$) and k_a is the coefficient of active earth pressure ($\tan^2 \phi (45 - \phi/2)$).

Reese et al. (18), using a stress block compatibility method, arrive at:

$$P_{Lpile} = k_a \sigma'_v (\tan^8 (45 + \frac{\phi}{2}) - 1) + k_o \sigma'_v \tan^4 (45 + \frac{\phi}{2}) \tan \phi \quad (13)$$

this solution assumes that the active earth pressure is developed in the soil element behind the pile; this is probably an upper bound solution because of the arching phenomenon which may develop behind the pile. If instead the passive earth pressure is assumed to exist in the soil element in front of the pile, the expression obtained is probably a lower bound:

$$P_{Lpile} = k_p \sigma'_v (1 - \frac{1}{\tan^8 (45^\circ + \frac{\phi}{2})}) + k_o \sigma'_v \frac{\tan \phi}{\tan^4 (45^\circ + \frac{\phi}{2})} \quad (14)$$

For ϕ varying from 30° to 40° and for reasonable values of the other parameters the ratio P_{Lpile}/P_{Lpmt} varies from 13.5 to 36.5 when using equation 13 and from 1.4 to 1.57 when using equation 14. Equation 13 and 14 assume that k_p and k_a which have been developed for infinitely long walls are applicable to the lateral punch of a narrow pile. The exact solution probably lies between the two bounds.

Using Hansen theory (11) for values of ϕ ranging from 30° to 40° , the ratio P_{Lpile}/P_{Lpmt} varies from 4.18 to 14.16, respectively. Also, using the pressuremeter bearing capacity factor for a deeply buried strip footing in sand (2) gives:

$$\frac{P_{Lpile}}{P_{Lpmt}} = 2.1 \text{ for loose sand}$$

$$\frac{P_{Lpile}}{P_{Lpmt}} = 4 \text{ for dense sand}$$

It appears from all the above results that p_{Lpile} is larger than p_{Lpmt} ; the ratio p_{Lpile}/p_{Lpmt} varied from 1.4 and 36.5 and averaged 2.6 for clay and 9.9 for sand. Although many of the assumptions of the various theories cited can be challenged, the above theoretical considerations all tend to show that the pressuremeter p-y curve method will be conservative at both small and large strains unless the pressuremeter curve is corrected in some fashion.

V. LABORATORY TESTS

A 0.75 x 0.75 x 0.6 m (2.5 x 2.5 x 2 ft) container was built and filled with clay. A model pile instrumented with strain gauges was driven into the soil. The lid of the box was screwed on tight preventing any vertical movement of the soil and the pile top was clamped simulating a fixed head condition. The pile was pulled laterally while the following were recorded: strain gauge output, total lateral load, deflection and rotation at the ground surface. After emptying the container the soil was placed again in an identical manner and a pressuremeter probe identical in size to the pile was driven in the deposit; the lid was screwed and a pressuremeter test was run.

The purpose of these tests was to compare the p-y curve of the pressuremeter with the p-y curve of a pile element having the same dimensions, embedded in the same deposit and displaced at the same rate.

The clay was a low plasticity clay with a liquid limit of 43.2%, a plastic limit of 20.4%, a natural water content of 28.2% and a total unit weight of 19.2 kN/m³ (122 lb/ft³). It was placed in the remolded state and tamped into place in thin layers; the uniformity of the deposit was checked by running small vane tests in the box. According to these tests the undrained shear strength of the clay was 24 kPa (500 lb/ft²).

The pressuremeter tests and the pile load tests were performed at the same rate of strain of: $dy/dt = 0.25$ mm/min. Figure 3 shows typical results. After driving the pressuremeter in place, the pressure in the probe corresponds to p_a and y is 0; curve 1 represents the result of the pressuremeter test. After driving the pile in place, the pile feels the same horizontal pressure as the pressuremeter; however the pile curve (curve 5) starts from 0 because no lateral load imbalance is created.

In order to compare the two p-y curves it is necessary to transform curve 1 so that it goes through the origin. Several solutions are shown: linear extension and horizontal translation (curve 2) and vertical translation (curve 3). A third solution consists of finding the equation of curve 1 by regression analysis extend it to $p=0$ and translate it (curve 4); in this case a hyperbolic model has been found to be very well suited (12). It does not seem appropriate to consider curve 3 as the PMT p-y curve because it ignores the part of the lateral soil resistance mobilized by the driving of the probe. Curve 2 and 4 compare favorably with curve 5. Tests performed with sand in the container instead of clay showed similar trends.

VI. FULL SCALE FIELD TESTS

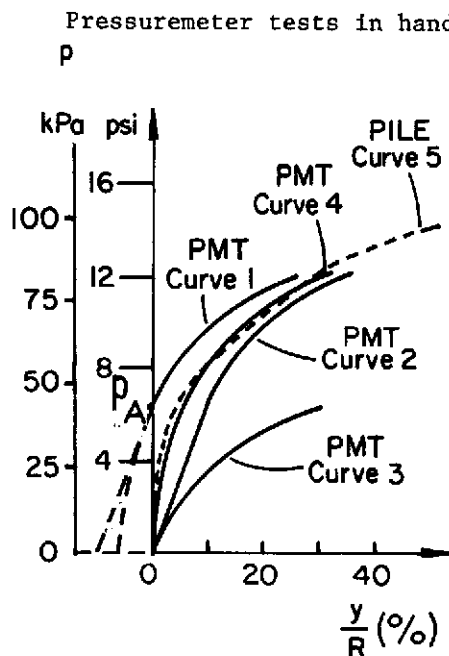


Figure 3: Laboratory Results

Pressuremeter tests in hand augered holes were performed at six sites in Texas where instrumented piles had been loaded laterally and where detail information was available from the load tests. The piles ranged from 0.30 m to 0.90 m (1 to 3 ft) in diameter, from 4.5 m to 21 m (15 to 70 ft) in length and the soil at the sites were soft clay (2 sites), stiff clay (3 sites), dense sand (1 site).

A. TEXAS A&M UNIVERSITY SITE

The soil at this site is a stiff clay with the following average characteristics: liquid limit 50%, plastic limit 20%, natural water content 25%, total unit weight 20.1 kN/m^3 (128 lb/ft^3). The undrained shear strength from unconfined compression tests varies from 48 to 191.5 kPa (1000 to 4000 lb/ft^2) and averaged 95.8 kPa (2000 lb/ft^2). The water table

is approximately 4.5 m (15 ft) deep.

Three concrete drilled shafts were tested at that site (3). Their diameters and embedded length are shown on figures 4, 5, and 6. Seven pressuremeter tests were performed in one hand augered borehole between the depths of 0 and 5.4 m (0 and 18 ft) with the pressuremeter described in reference 4. A typical result is shown in figure 2. Each pressuremeter test included the performance of a complete unload-reload cycle starting at the end of the pseudo elastic part of the PMT curve (fig. 2); each curve was corrected for volume losses and membrane resistance; then the straight line part of the curve was extended to $p=0$ and the origin was placed at point 0 on figure 2. A similar procedure was applied to the reload part of the curve.

These PMT p - y curves were inputed in the finite difference computer program in order to predict the load-displacement curve for the piles. Figures 4, 5 and 6 show the results as well as the curves obtained from the load tests. Also the p - y criterion used in current practice was applied to obtain the fourth curve on the figures. It can be seen that generally the current criterion leads to very good prediction, that the first loading PMT curves lead to a conservative

Lateral Load (tons)

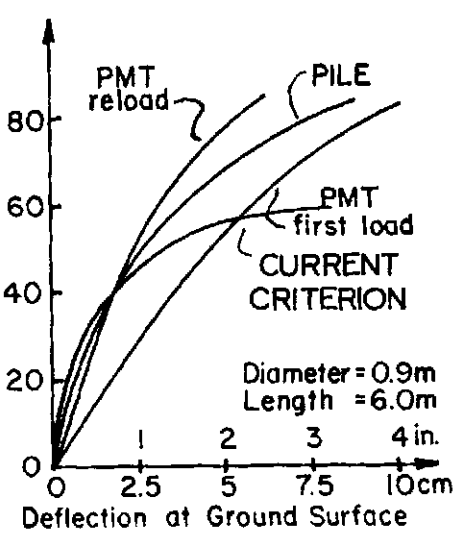


Figure 4: Texas A&M University 1977 drilled shaft

Lateral Load (tons)

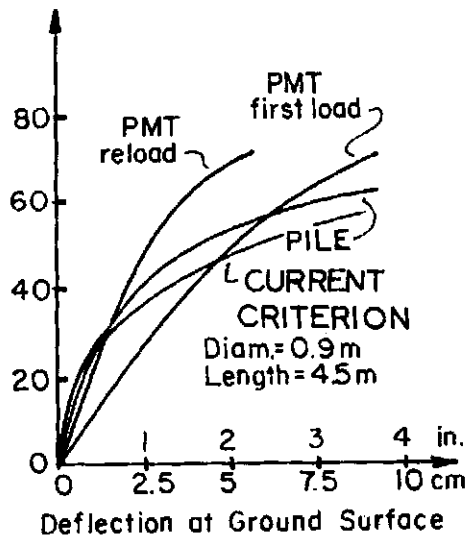


Figure 5: Texas A&M University 1978 drilled shaft

prediction, that closer agreement is obtained when using the reload PMT curves.

Lateral Load (tons)

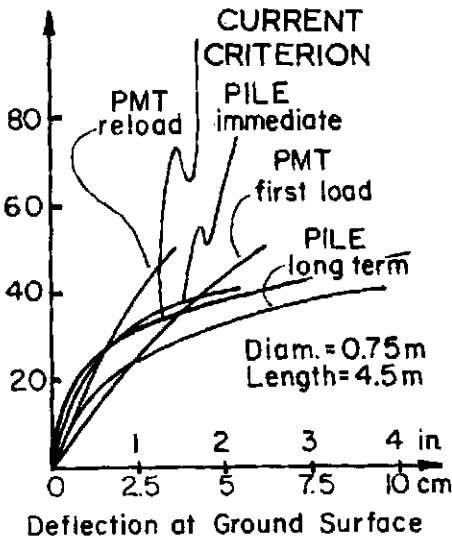


Figure 6: Texas A&M University 1979 drilled shaft

Since the current criterion gave such a close agreement, the p-y curves obtained by using this criterion were assumed to be the true p-y curves. These true p-y curves were compared with the first loading PMT curves (fig. 2). It turned out that consistently the maximum pressure difference between the two curves (c_u on fig. 2) was approximately equal to the soil shear strength. Furthermore, the drilled shafts were instrumented over their entire length with total pressure cells at the soil-shaft interface; the response of these cells led to p-y curves a typical example of which is given on figure 2. As was observed consistently on the test results of this site, the first loading PMT

p-y curves and the pressure cells p-y curves were in very close agreement. The two previous findings and common sense led the authors to believe that the following concept is valid and necessary to adequately describe the behavior of piles loaded laterally.

B. f-y/q-y MECHANISM FOR LATERALLY LOADED PILES

Consider a square pile for convenience (fig. 7). The resistance to the lateral displacement of the pile is made of two components: the front resistance described by the q-y curve and the friction resistance described by the f-y curve. The total resistance per unit length of pile P is then obtained for a given displacement y from:

$$P \text{ (kN/m)} = f \times 2B + q \times B \quad (15)$$

The entire p-y curve can be constructed from the f-y and q-y curves. For a pile having a cross-section different from square the coefficients 2B and B in equation 15 are different functions of the width B.

It is known from vertical load tests on instrumented piles that generally the displacement required to mobilize $f_{\max}$ on the f-y curve is smaller if not much smaller than the displacement required to mobilize $q_{\max}$ on the q-y curve. This means that for the small displacements occurring at working loads, the resistance due to friction may be a large part of the total lateral resistance, while the resistance due to front resistance is probably small. For example for the shaft of figure 4, at 1/3 of the ultimate lateral load, the front resistance accounted for only 30% of the total resistance and at 2/3 of the ultimate lateral load, the percentage of front resistance had become 50%.

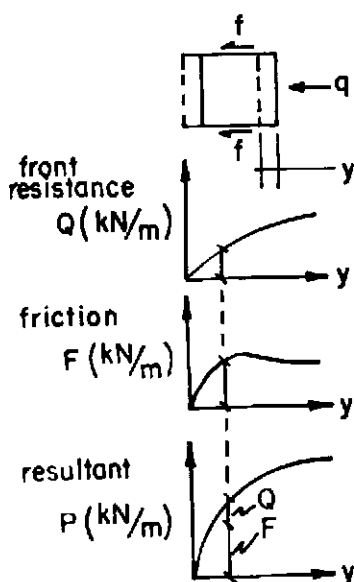


Figure 7: Lateral Load Mechanism

The close agreement between the first loading PMT curve and the pressure cell curve on figure 2 tends to show that the PMT curve gives a close estimate of the q-y curve. The f-y curve is similar in nature to the friction-vertical displacement curve (f-z) used for vertically loaded piles. One way of obtaining the f-y curve

DESIGN OF Laterally Loaded PILES USING PRESSUREMETER

would therefore be to use the f - z curves which have been developed for sand and clay (6). The pressuremeter can also provide elements of data necessary to construct the f - y curve; indeed, the shear stress-strain curve for the soil can be obtained from PMT or at least from selfboring PMT by a theoretical procedure in the case of the undrained behavior of cohesive soils (2); for cohesionless soils and for a circular pile the maximum friction f is related to the pressure p normal to the shaft through the angle of friction between the soil and the shaft material; the pressure p can be read on the PMT curve at the appropriate deflection y . Yet another way to obtain an f - y curve is to use the f - z curve based on PMT data (9).

C. HOUSTON SITE

The soil at this site is a stiff to very stiff clay with the following average characteristics: liquid limit 70% plastic limit 20%, natural water content 25%, total unit weight 19.6 kN/m^3 (125 lb/ft^3). The undrained shear strength from unconsolidated undrained triaxial tests varies from 57.5 to 143.6 kPa (1200 to 3000 lb/ft^2) and averaged 105.3 kPa (2200 lb/ft^2). The water table is approximately 5.4 m (18 feet) deep.

A 0.75 m (2.5 ft) diameter, 13.2 m (44 ft) long concrete drilled shaft was tested at that site (21)(fig. 8). Five pressuremeter tests were performed between the depths of 0 and 4.5 m (0 and 15 ft); they were reduced and input in the program as for the Texas A&M site and the curves obtained are shown on figure 8. A trend similar to the one observed at the Texas A&M University site exists.

Lateral Load (tons)

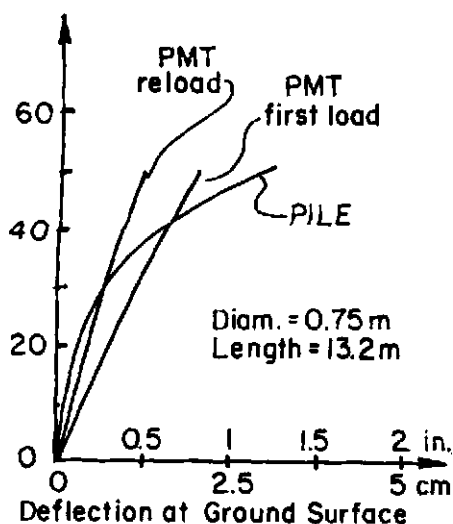


Figure 8: Houston drilled shaft

D. MANOR SITE

The soil at this site is a stiff to hard clay with the following average characteristics: liquid limit 77%, plastic limit 20%, natural water content 25%, total unit weight about 19.6 kN/m^3 (125 lb/ft^3). The undrained shear strength increases with depth from 95.8 kPa (2000 lb/ft^2) at the surface to 383 kPa (8000 lb/ft^2) at 4.8 m (16 ft) depth. The site was artificially flooded up to the ground surface four months prior to the load test.

DESIGN OF Laterally LOADED PILES USING PRESSUREMETER

A 0.63 m (2.1 ft) diameter, 15 m (50 ft) long steel pipe pile was driven into the clay and tested laterally (19). Five pressuremeter tests were performed between the depths of 0 and 4.5 m (0 to 15 ft); they were reduced and input in the program as for the Houston site and the curves obtained are shown on figure 9. A trend similar to the one observed at the Texas A&M University site exists.

E. SABINE SITE

The soil at this site is a soft marine clay of high plasticity with the following average characteristics: liquid limit 90%, plastic limit 25%, natural water content 70%, total unit weight 15.7 kN/m^3 (100 lb/ft^3). The undrained shear strength as measured in consolidated-undrained triaxial and miniature vane tests varied from 4.8 kPa (100 lb/ft^2) at the ground surface to 23.9 kPa (500 lb/ft^2) at a depth of 9 m (30 ft), and averaged 14.4 kPa (300 lb/ft^2). The sensitivity as measured by the vane test varied between 2 and 3. The water table was at the ground surface.

A 0.32 m (1.06 ft) diameter, 12.6 m (42 ft) long steel pipe pile was driven into the clay and tested laterally (15). Six pressuremeter tests were performed between the depths of 0 and 6.6 m (0 and 22 ft). As for all other sites the hole was first hand augered in the dry and then the probe was inserted; however in this soft clay some inward yielding of the borehole wall took place as the auger

was removed. As a result the PMT curves looked like curve 1 on figure 3 and were transformed into the pressuremeter $p - y/R$ curves by applying the method used to obtain curve 2 on figure 3. These transformed PMT $p-y$ curves were input in the program and the curves obtained are shown on figure 10. In this soft clay the behavior predicted by the PMT $p-y$ curves is stiffer than the measured behavior; this is contrary to the findings at the three stiff clay sites. This difference may be due to the difference in insertion procedure; the displacement taking place in the soft clay during the probe insertion probably led to a soil response stiffer than the one which would have been obtained in a cleanly augered borehole.

Lateral Load (tons)

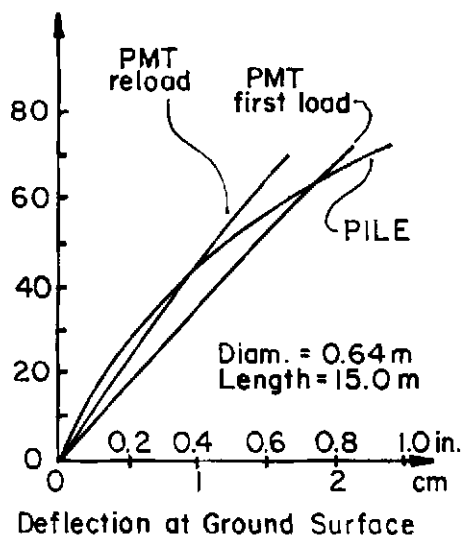


Figure 9: Manoir Steel Pipe Pile

F. LAKE AUSTIN SITE

Lateral Load (tons)

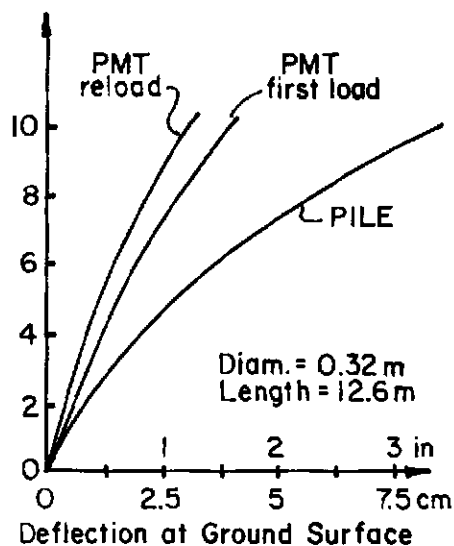


Figure 10: Sabine Steel Pipe Pile

The soil at the site is a soft clay with the following average characteristics: liquid limit 75%, plastic limit 25%, natural moisture content 45%, total unit weight 17.3 kN/m^3 (110 lb/ft^3). The undrained shear strength varies from 9.6 kPa (200 lb/ft^2) to 38.3 kPa (800 lb/ft^2) and averages 19.1 kPa (400 lb/ft^2) as measured in unconfined compression tests, and varies from 19.2 to 47.9 kPa (400 to 1000 lb/ft^2) averaging 38.3 kPa (800 lb/ft^2) in consolidated undrained triaxial tests. The water table was at the ground surface.

A 0.32 m (1.06 ft) diameter, 12.6 m (42 ft) long steel pipe pile was driven into the clay and tested laterally (15). Six pressuremeter tests were performed

between the depths of 0 and 5.4 m (0 to 18 ft). The same insertion problem as the one encountered at the Sabine site was encountered at Lake Austin; as a result the predicted behavior from PMT p-y input gives a stiffer response than the measured response (fig. 11); however because the inward yielding of the wall of the borehole at Lake Austin was less than the one at Sabine, the comparison between predicted and measured behavior is more favorable at Lake Austin.

G. MUSTANG ISLAND SITE

The soil at the site is a dense uniform fine clean sand with the following average characteristics: natural water content 25%, minimum dry density 12.4 kN/m^3 (79 lb/ft^3), maximum dry density 16.2 kN/m^3 (103 lb/ft^3) natural dry density 15.7 kN/m^3 (100 lb/ft^3). The blow count from Standard Penetration Tests increased almost linearly with depth from 10 blows/ft ($\text{blows}/0.3 \text{ m}$) at the surface to 60 blows/ft ($\text{blows}/0.3 \text{ m}$) at a depth of 12 m (40 ft). The water table was at the ground surface.

A 0.60 m (2 ft) diameter, 21 m (70 ft) long pile was driven into the sand and tested laterally (18). Only very shallow pressuremeter tests were performed at that site because of the difficulties encountered; deeper PMT p-y curves were obtained by using the results of the SPT which predicted a linear increase with depth. For example

Lateral Load (tons)

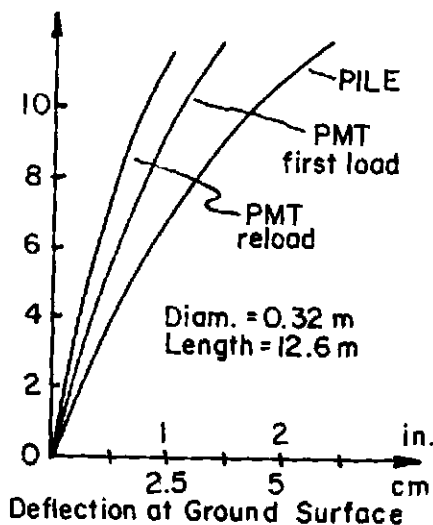


Figure 11: Lake Austin Steel Pipe Pile

been discussed. These include the effect on the p-y curves of: increasing pile diameter, proximity of the ground surface, rate of loading, cyclic loading, group action.

For the results presented, it can be concluded that:

1. The elasticity and plasticity theories as well as the established rules of foundation design on the basis of pressuremeter tests all indicate that the pressuremeter p-y curve will represent a conservative estimate of the pile p-y curve. The ratio k_{pil}/k_{pMT} was found to average 1.50 while the ratio p_{Lpil}/p_{LpMT} averaged 2.6 for clays and 9.7 for sands.

2. The way the pressuremeter probe is placed in the ground influences the results: if the

the PMT p-y curve at a depth of 3m (10 ft) was obtained from the measured PMT curve at a depth of 0.9m (3 ft) by multiplying the pressures p of the latter by 3.33. The predicted load-deflection curves are shown on figure 12. Here again the behavior predicted from the PMT p-t curves is conservative.

VI. CONCLUSIONS

This article has focused on the comparison of the pressuremeter p-y curve and the pile p-y curve, on the fundamental mechanism involved in the lateral movement of a pile element, and on the prediction of the lateral load-lateral deflection curve for the pile using the entire pressuremeter curve. Other important aspects of the design have not

Lateral Load (tons)

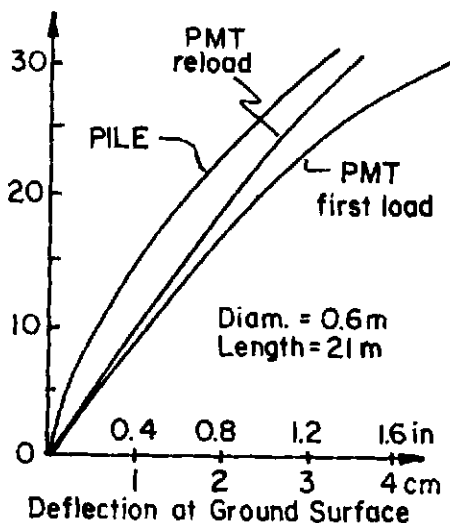


Figure 12: Mustang Island Steel Pipe Pile

DESIGN OF Laterally Loaded Piles Using Pressuremeter

probe is inserted in a hand augered hole the pressuremeter p-y curve method gives a conservative estimate of the load deflection curve for the pile at working loads, if the probe insertion creates displacement of the soil the pressuremeter p-y curve method gives a load-deflection curve which is stiffer than the real one for the pile.

3. Two curves are necessary to describe the behavior of a pile element loaded laterally: a friction-displacement curve (f-y) and a front resistance-displacement curve (q-y). It is an appropriate combination of these two curves which give the p-y curve. At working loads the friction contribution to the lateral resistance is large.

4. The pressuremeter gives a good measure of the q-y curve and may provide the information necessary to obtain the f-y curve.

ACKNOWLEDGMENTS

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FOUNDATION STABILITY CALCULATION METHOD USING CYCLIC PRESSUREMETER OR TRIAXIAL TEST RESULTS

ABSTRACT

For the overconsolidated clays in the North Sea, precision cyclic pressuremeter measurements performed from a gravity baseplate provided justification of an original method for computing the stability of a foundation raft under cyclic loading. These results can be transposed to a triaxial cell.

The method consisted in determining :

- an elasticity threshold below which irreversible cyclic deformations are negligible as are pore pressure variations ;
- a fatigue threshold above which there is no cyclic stability;
- between these two thresholds, the clay may take on pore overpressure and undergo considerable irreversible deformation for which stabilization probably occurs after a great many cycles.
- A gravity structure can be installed perfectly if the maximum cyclic stresses caused by storms do not go beyond the elastic limit.

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STABILITY CALCULATION METHOD

1 - INSTALLATION OF A SENSITIVE PRESSUREMETER BELOW A NORTH SEA GRAVITY STRUCTURE (FRIGG FIELD).

Equipment, soils and main results are described elsewhere [2,4]. We simply recall to mind that the originality and the quality of results proceed from the high sensitivity of cells (10^{-4} for pressure, $5 \cdot 10^{-5}$ for volume) actuated in situ by an electrovalve. An electrical transducer fixed in the middle of the flexible membrane continuously records pore pressure. The installation was expected to minimize soil disturbance : two probes with three cells (two active, one passive) were lowered by jacking into two different boreholes in predrilled 2" 3/8 (60.3 mm) holes. OD of cells is 65 mm. The lower part of the probe consists of a cutting head equipped with water nozzles. Sediments are drained off through the central hole of the probe. The active cells lie between 8 and 10 m below the slab/sea-bottom.

The surrounding soil is overconsolidated silty clay and clayey silt, separated from the slab by 3 to 4 m of dense fine sand. The main properties of this sediment are :

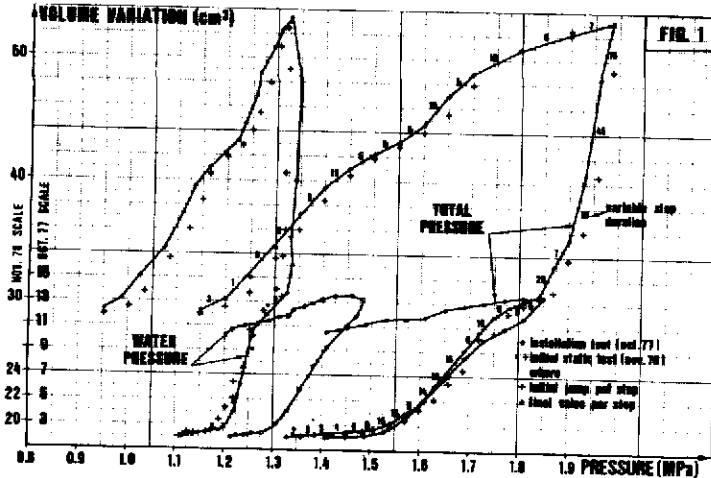
- mineralogy : 26 to 30% illite, 11 to 18% smectite, 55 to 62% kaolinite,
- carbonate content : 14 to 19%, effective phi angle : 24 to 28 degrees, plasticity index : 10 to 35%, liquid limit : 25 to 60%, water content 15 to 25%.
- average cohesion at cell levels : 190 kPa
- OCR at cell levels (including structure load) : 2 to 4 estimated $K_0 = 0.9$.

The structure is a Condeep type, three column structure, of which the maximum design loads are at sea-bottom level :

- vertically : $165000 \text{ t} / 9340 \text{ m}^2 = 177 \text{ kPa}$
- horizontally : $\pm 44000 \text{ t} / 9340 \text{ m}^2 = \pm 47 \text{ kPa}$

The guideline of this presentation is figure 1, which illustrates a complete static pressuremeter test, performed one year after the non-disturbing installation of the cells inside the investigated overconsolidated clay. This means that soil around the cells was perfectly at rest at the date of the test and presumably in the same state as before installation of the cells. The typical shape at the beginning of curves showing the lateral earth pressure p_0 (1.33 MPa on fig.1) indicates an average value for all tests of 1.3 MPa, whereas computations give 1.33 MPa.

STABILITY CALCULATION METHOD



Pressuremeter curves. Creep tests in steps

2 - ELASTIC BEHAVIOUR WITH PRESSUREMETER

2.1 - Description of elastic response to static loading

On figure 1, the beginning of the $p - v$ curve is represented by a straight segment almost parallel to the volume variation axis. We observe 0.5 or 0.8 cm³ of volume variation for about 1.53 - 1.33 = 2MPa of load increase. The corresponding modulus is about 300 MPa according to usual formula.

The curve $p - v$ is deduced from load steps of variable duration as mentioned in figure 1. For any step of load inside the range of elasticity, it is observed that (slides):

- pore pressure response is either an instantaneous stabilization or a response with a small peak.
- volumetric response is negligible
- elasticity is then defined according to observation by reversibility with almost no volume change, whereas the pore pressure variation is more changeable. This is in agreement with the theoretical formula describing the pressure p at the wall of the cavity [1].

$$p = p_0 + 2\mu e_{ro}$$

μ being Lamé's rigidity modulus

e_{ro} being the radial relative displacement at the wall of the cavity.

This is also in agreement with the fact that [1] $e_r = e_\theta$ for an elementary volume (e_θ = circumferential strain). As it is usually agreed that $e_z = 0$ (vertical strain), consequently, volume change $\Delta V/V = e_z + e_r + e_\theta = 0$. Only the rigidity modulus governs soil deformation.

The limit of elasticity corresponds to a "knee" of $p - v$ curves (fig.1) around 1.525 MPa. The difference with p_0 is about 2 MPa, whereas corresponding pore pressure build-up is about 1 MPa giving 1 MPa of effective stress. This value must be compared to the effective overconsolidation pressure of 0.5 to 1 MPa indicated by oedometer tests and, consequently, the initial elastic clay behaviour seems to have been generated by past overconsolidation.

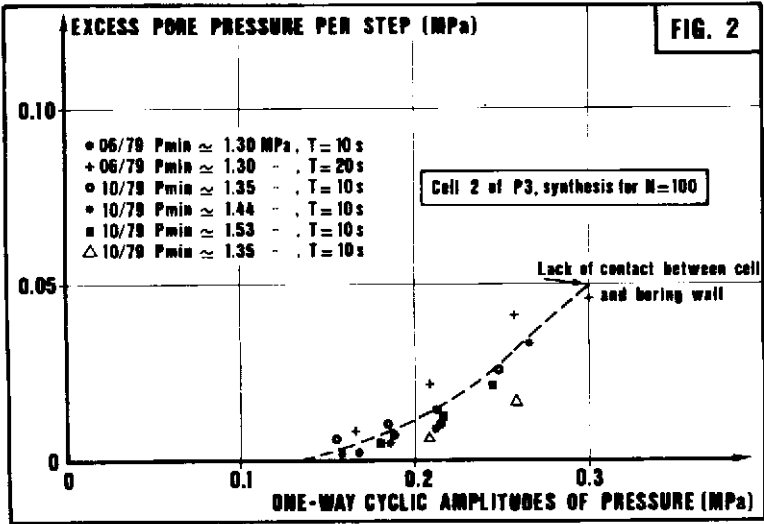
2.2 - Description of cyclic elastic behaviour and discussion

Among other cyclic tests, we have performed one way cyclic tests with increment by steps of 100 cycles starting from p_0 . From the first cycle of each step, we observe an instantaneous stabilization in cyclic volume variation and cyclic pore pressure in response to a step of cycles of applied pressure (slide). There is no excess pore pressure, in agreement with readings of piezometers installed below the gravity structure when (computed) cyclic loading does not exceed the elastic limit [4]. The same results are observed for two-way cycles around at rest pressure of soil (no figures).

Figure 2 illustrates the variation of excess pore pressures for one-way cyclic loadings by steps of 100 cycles. As expected from an elastic response, no excess pore pressure is observed before cyclic amplitude reaches 0.15 MPa which, added to initial pressure of 1.3 or 1.35 MPa, gives the limit of elasticity. Nevertheless, lack of excess pore pressure is also observed on this figure when cycles start from a minimum pressure of 1.44 or 1.53 MPa, so that the maximum amplitude becomes in excess of the elastic limit! The conclusion is thus that excess pore pressure essentially depends on the amplitude of one-way cycles, independently of the total pressure of reference, within the range of cyclic loads used in our tests. This is no longer true when soil approaches failure. The explanation is as follows :

The pore pressure variation (as well as the volume variation) is measured based on the value of p before the cyclic test. If this value is greater than p_0 there is an initial hardening which causes a variation of both u and v . Any increase of these two

parameters during the cyclic test will be influenced by this hardening and it has been shown [3] that under these conditions, the main parameter becomes the amplitude of the cyclic strain as long as we remain quite a long way from soil failure, as in the present case. In addition, as the clay is overconsolidated, low Δp values (here $\Delta p = 1.5 \text{ bar}$) cause little variation in the pore pressure. This can be observed on the pressuremeter results and we had previously noticed it on overconsolidated clays in triaxial tests [3].



Synthesis curve of pore pressure variations during cyclic pressuremeter tests in steps

3 - THE SECOND OR NON-REVERSIBLE LINEARITY

This is the usual linearity observed with pressuremeter curves and called "pseudo-elasticity" because volume variations are no longer reversible. It corresponds to the second linear part of our figure 1.

3.1 - Description of static behaviour

The clay modulus is 65 MPa. Contrary to the elastic case, the volume response to a step of pressure (0.02 to 0.03 MPa) cannot reach stabilization, despite step duration of up to 20 min, whereas pore pressure response U resembles the elastic feature, independently of peak or immediate stabilization (slides).

We did not apply steps of static pressure longer than 20 minutes, but cyclic tests - one of them lasting 7.5 hours and showing no stabilization in cumulated volume variation (see section 3.2). The drift of non-stabilization is so reduced that the linearity of p - V or p - U graph is not altered by the difference of duration of the successive steps (see fig. 1). The existing theories, synthesized in [1] can no longer apply because they do not take into account either changes of volume or non-reversibility.

3.2 - Description of cyclic behaviour

A one-way cyclic test with 4500 cycles between $p_0 = 1.25$ MPa and $p = 1.68$ MPa (i.e. more than elastic limit) is illustrated in figures 3a and 3b. Despite a decrease of average pore pressure as well as of amplitude of pore pressure cycles versus number of cycles, cumulated volume variation does not stabilize. Amplitude of cyclic volume variation decreases slightly with the increasing number of cycles.

More excess pore pressure variations for one-way cycles are illustrated in figure 4. Corresponding volume variations show a similar trend with increasing amplitude of cycles. Figure 5 illustrates the cumulated variations of volume per step of 100 one-way cycles : as soon as the elastic limit is exceeded, volume variations accelerate.

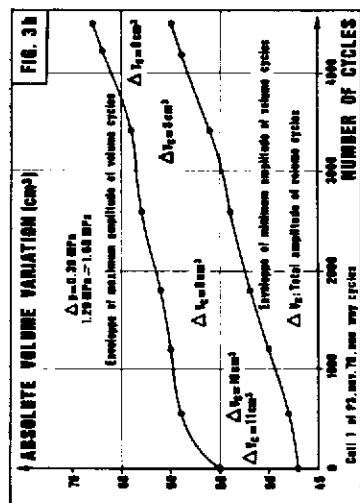
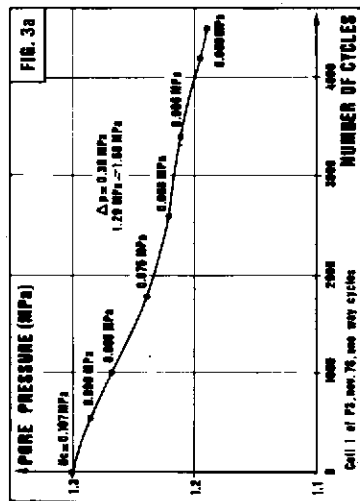
An example of a two-way cyclic test is as follows : the volume stabilizes when cycles reach the amplitude of ± 0.055 MPa around at-rest pressure whereas double amplitude induces cumulative volume variation. Despite this, amplitude varies between 1.2 and 1.4 MPa ($p_0 \pm 0.1$ MPa) i.e. lies below the elastic limit.

3.3 - Creep limit

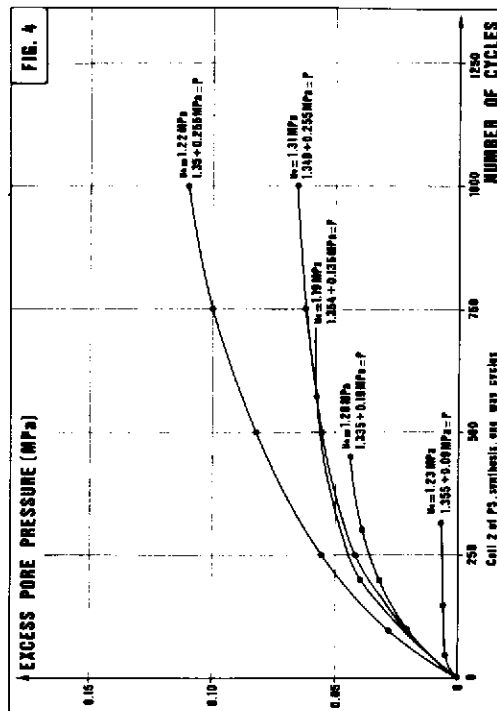
With the present sensitive cells, it is observed that, above the creep limit, cumulated steps of equal static pressure induce increasing volumetric creeps whereas pore pressure measured alongside the cell remains constant (ex. fig. 1) : this pattern corresponds to the propagation of plasticity from the borehole wall to the inside of the clay strata.

As soon as plastic creep is initiated, an immediate reduction of cell pressure does not stop it but only reduces its rate.

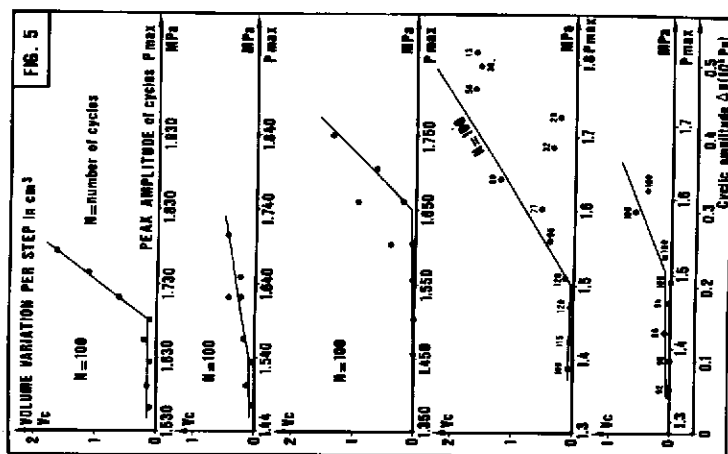
The same behaviour is observed with one way-cycles of which amplitude exceeds the static creep limit.



Fatigue test with cyclic pore pressure. Measurement of pore pressure and volume variation



Fatigue tests with cyclic pore pressure. Measurement of pore pressure.



Cyclic pressuremeter tests in steps. Measurement of different fatigue regimes.

STABILITY CALCULATION METHOD

3.4 - Unloading :

Unloading is governed only by pore pressure release, with or without cycles.

4 - COMPARISON WITH TRIAXIAL TEST

4.1 - General comparison

An elastic soil response is rarely seen as a result of usual static triaxial testing, even for overconsolidated clays. Generally, linearity observed during triaxial testing is non-reversible and is similar to the usual linear part of pressuremeter curves. Sections 3.1 and 3.2 apply directly to triaxial behaviour. The difference lies in the mechanisms of degradation. A sample in a triaxial cell is simultaneously subjected to distortion in the meridian plane (r, z in cylindrical coordinates) and to compression, then to dilatation, in sections (r, θ planes) where $\sigma_r = \sigma_\theta$ and $e_r = e_\theta$. Soil around the pressuremeter cell is distorted only in the horizontal (r, θ) planes where $e_r = -e_\theta$ because e is assumed to be zero [1]. This is enough to understand why, despite the above similarity, limits of behaviour and modulus will be numerically different even for identical consolidation pressures. Another discrepancy concerns pore pressure measurement and results from the different locations of the two sets of gauges. With the exception of these quantitative restrictions, the descriptions of elastic or non-elastic linear soil behaviour presented in sections 2 and 3 are also valid for similar triaxial testing and, particularly, for the following procedure of cyclic testing of clay in a triaxial cell suggested by Professor BIAREZ, which we call the "threshold test".

4.2 - The threshold test

This test consists of simulating cyclic pressuremeter testing with a triaxial cell, by successively applying equal pressure steps of 100 cycles and measuring volume and pore pressure response as illustrated in slides.

Results can be plotted :

1) as permanent vertical strain $\epsilon_p = \epsilon_{100} - \epsilon_1$ (ϵ_{100} = strain for 100th cycle, ϵ_1 = strain for initial cycle) as a function of the non-dimensional ratio q'_c / σ'_o ($q'_c = \sigma'_{1c} - \sigma'_3$ for effective cyclic amplitudes, $\sigma'_o = \frac{1 + 2 k_o}{3} \sigma'_c$, σ'_c is the consoli-

3

STABILITY CALCULATION METHOD

dation load and σ'_v is the effective overburden pressure). This is illustrated on figure 6A for one-way cyclic loading and 7A for two-way cyclic tests.

2) as simultaneous excess pore pressure $U_{100} - U_{50}$ versus the same ratio q'_c / σ'_c , as illustrated in figure 6B and 7B.

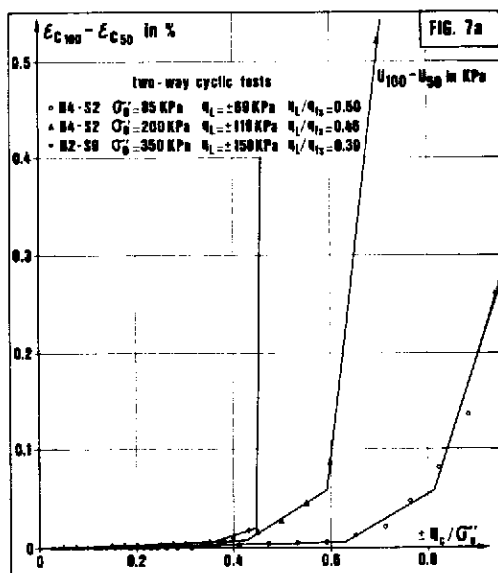
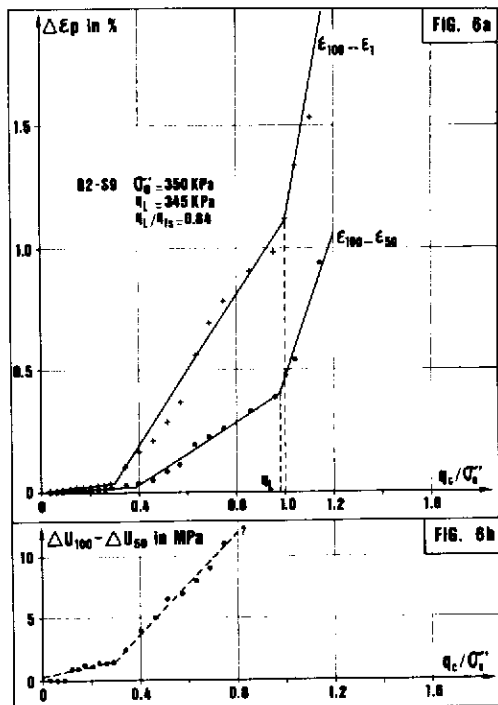
The results presented as examples correspond to a heterogeneous clayey silt from the North Sea, collected at the same depth as pressuremeter depth but at different locations. Nevertheless, the geotechnical properties listed below are fairly similar to those in chapter 1.

- water content = 21% - wet density = 2.01 - dry density 1.66 - liquid limit = 31 to 41% - plastic limit = 19% - plasticity index = 10 to 23
- mineralogy : illite 30%, kaolinite 20%, smectite 10%, CO₃Ca 1 to 6%, quartz and feldspar 20% to 25%
- deviator at failure q_{sf} sample B4 is 137 kPa = 33°
- deviator at failure q_{sf} sample B2 is 410 kPa = 29°
- OCR from oedometer test = 2 to 4

Let us now compare the two thresholds :

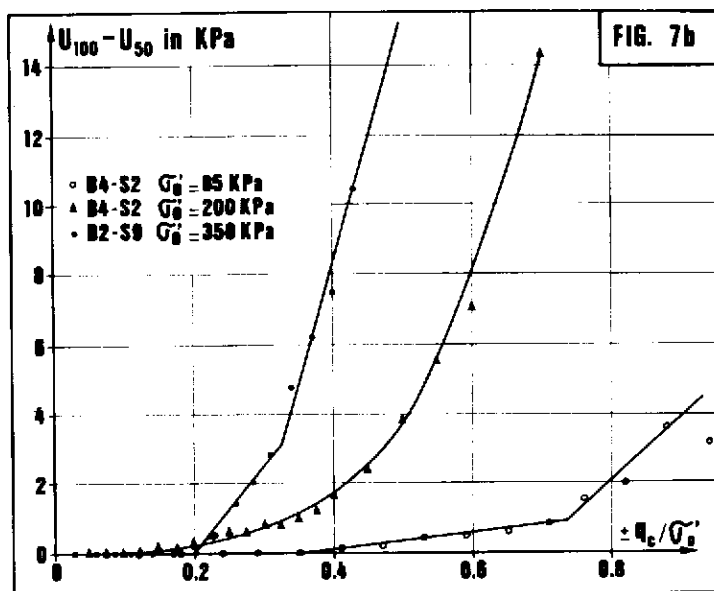
- one indicates the boundary between initial and second linearity, and can be compared to the pressuremeter elastic limit. Nevertheless for one-way cyclic tests with triaxial cell, we only observe lack of excess pore pressure associated with volume stabilization for the first steps but not for all steps of this first linearity. A possible explanation is that triaxial conditions are entirely undrained, contrary to pressuremeter conditions. In the laboratory, this phenomenon is found on normally consolidated as well as on overconsolidated clays.
- the second threshold limits the linearity and can be compared to that obtained with the pressuremeter because beyond this threshold, cyclic strains increase rapidly and lead to failure of the sample.
- between the two thresholds, pore pressure build-up and non-stabilized strain exist but their increase as a function of cyclic stress is relatively low.

However, one-way tests have been carried out on remoulded clays, prepared in the laboratory. We have shown that with these clays the cyclic strain values lower than the second "knee" lead to stabilization of both strain and pore pressure in undrained conditions. All the same, it should be noted that the stabilized value of axial strain can reach several percent which is not always acceptable with regard to the safety of the structure.



One-way triaxial cyclic tests in steps. Showing different fatigue regimes.

Two-way triaxial cyclic tests in steps. Showing different fatigue regimes.



Two-way triaxial cyclic tests in steps. Showing different fatigue regimes.

Figures 6 and 7 can be compared to figures 2 and 5 for pressuremeters but it was not possible to plot pressuremeter curves as far as limit pressure due to lack of contact between cell and boring wall (the soil does not follow the deflated cell during unloading when amplitude exceeds 0.3 MPa) and also because it was necessary to avoid disturbing the clay too much so that further tests could be performed. However, in respect of the second test in figure 5, if we mentally extrapolate to 100 cycles the last $\Delta v = V_n - V_i$ calculated for a small number of cycles in steps (13,38), we find a rate of increase in Δv per 100 cycles which is much higher than that drawn for amplitudes of less than 1.75 MPa (value of static creep pressure).

For the pressuremeter, the threshold under one way cyclic loading coincides numerically with the threshold during static loading (i.e. the elastic limit of about 1.515 MPa). In respect of triaxial testing, the cyclic stress limit q_L relative to the maximum stress is $q_L / q_{fs} = 0.84$ for one-way loading (fig. 6) but is much lower for two-way cycles (fig. 7). The tests carried out on remoulded normally consolidated samples have shown that q_L obtained from one-way tests is close to the undrained creep limit.

4 - CONCLUSION

More research is necessary in order to quantitatively relate pressuremeter and triaxial thresholds as well as static and one-way or two-way cyclic thresholds. At present, we are sure of the qualitative similarity of linear behaviours and of their boundaries for overconsolidated clays subjected to cyclic testing with pressuremeter (if sensitive enough) and triaxial cell. (See also ref. 4).

If the slab is designed so that the maximum cyclic shear induced by waves in the upper bearing clay layer never exceeds the elastic limit, no excess pore pressure or fatigue or permanent displacement will be observed. This is the case of the structure overlying the pressuremeters, as analysed in [4].

Conversely, if occasionally cyclic load induced by the severest storms exceeds the elastic limit, there is an intermittent settlement each winter season. Case history is presented in [5].

The so-called elastic limit for overconsolidated clays is thus evidenced by special triaxial cyclic tests, and can be introduced in classical computations for the design of foundations. It represents a limit of total safety against cyclic fatigue and cyclically induced displacements. The validity of such criteria has been

STABILITY CALCULATION METHOD

verified by in situ tests with sensitive pressuremeters as well as by the observations of the behaviour of a real structure on FRIGG FIELD.

ACKNOWLEDGMENTS :

We express our gratitude to ELF AQUITAINE NORGE, owner of the instrumentation of FRIGG FIELD and its results, for their permission which made this publication possible. We thank Mr. EL HOSRI (ECP) for his participation in this work.

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SESSION 4

ADDITIONAL CONTRIBUTIONS AND DISCUSSION

DISCUSSION

A. SACRE - (*d'Appolonia Brussels*) asked Mr. BRIAUD

The pressuremeter test closely simulates the behavior of the soil near a laterally-loaded pile. But a problem exists near the surface. Could I have an answer on this subject ?

J.L. BRIAUD - (*Texas A & M University*)

Shallow depth pressuremeter tests show that :

- . with small deformations, nearness to the surface has a negligible effect on the value of the modulus, for example ;
- . with large deformations, nearness to the surface exerts an effect on the limit pressure as shown by Ménard ; the problem is to know how much the limit pressure decreases.

At the usual design loads, one may well ask whether this problem really exists.

F. BAGUELIN - (*Laboratoire Central des Ponts et Chaussées*)
asked Mr. BRIAUD

How do you determine your experimental curves of lateral pile reaction (types of measurement, depth) ?

J.L. BRIAUD - *(Texas A & M University)*

I have shown two types of reaction curves obtained using pressure cells and extensometers (strain gauges) respectively. The former yield lower values than the latter. The curves obtained using the pressuremeter are closer to those obtained with pressure cells than with extensometers.

CONTRIBUTION

PREDICTION OF LOAD VS. SETTLEMENT CURVES FOR FRICTION PILES

S. HANSBO - *(AB Jacobson and Widmark, Lidings, Sweden)*

An extensive review and discussion of the pressuremeter method and its application for the prediction of pile behaviour, according to different authors, was recently published by one of my students in a doctoral thesis, (Sellgren, 1981). In this thesis, Sellgren presents a method of calculation, by means of which the full shape of the load vs. settlement curve is obtained. The settlement s of the pile head is related to the initial slope a of the load/settlement curve and the failure load p_f , according to the formula

$$s = \frac{aP}{1 - P/P_f} \quad (1)$$

where P equals the pile load.

For the calculation of the value a in Eq (1), Sellgren recommends the use of Cassan's method (Cassan, 1966). Sellgren further gives the following recommendations : for driven piles, the pressuremeter modulus should be taken as the cyclic modulus. If cyclic tests have not been carried through, the modulus should be taken as 3 times the standard value. However, if the pressuremeter tests are carried out after the piles have been driven (within the zone of influence of pile driving), then the measured standard modulus should be used.

The failure load P_f , which is the other parameter in Eq (1), was predicted according to the methods of different authors and compared with the results of both full-scale loading tests and large-scale model tests. In the full-scale tests, four cylindrical piles with square cross-section were tested. Two piles were driven through a 4 m thick layer of loose, organic silt to 6-7 m depth in dense sand (Luleå), and the other two piles through a 10-11 m thick layer of soft, organic clay to 6-7 m depth in sand (Järnbrott). In the model tests, piles, 0.15 m in diameter, were driven to a depth of 1-1.5 m in sand, with an overburden pressure on the sand surface of 0-250 kN/m². The results of the loading tests in respect of failure load are in good agreement with the predicted failure loads according to Centres d'Etude Ménard (1975) and Baguelin et al (1978), whereas the

SESSION 4 ADDITIONAL CONTRIBUTIONS AND DISCUSSION

predicted failure loads according to Bustamante & Gianceselli (1981) are too low. The best agreement is obtained by the method proposed by Baguelin et al, see Tables 1-2 and figure 1.

The load vs. settlement curves according to Eq (1), using the failure load P_f according to Baguelin et al, are compared with the results of the full-scale loading tests in Luleå and Järnbrott, figures 2-3. As can be seen, the agreement between predicted and measured pile behaviour in loading is good. Equally good agreement was obtained in respect of the model piles.

Table 1. Failure load P_f , in MN, in the Luleå pile loading tests.
Pile 0.270 m square

Measured value	Pile	
	A	B
Butler & Hoy Method	1.33	1.17
Brinch Hansen 80%-Method	1.41	1.23
Chin Method	1.60	1.42
Davisson Method	1.16	1.10
de Beer Method	1.25	1.13
Mazurkiewicz Method	1.46	1.37
Swedish Pile Commission Method	1.48	1.35
Van der Veen Method	1.59	1.48
Predicted value		
Centre d'Etudes Ménard	1.38	1.33
Baguelin et al	1.57	1.49
Bustamante & Gianceselli	1.24	1.14

Table 2. Failure load P_f , in MN, in the Järnbrott pile loading test.
Pile 0.235 m square

Measured value	Pile	
	C	D
Butler & Hoy Method	0.95	0.86
Brinch Hansen 80%-Method	1.26	0.89
Chin Method	1.54	1.39
Davisson Method	0.86	0.85
de Beer Method	1.00	0.85
Mazurkiewicz Method	1.20	1.10
Swedish Pile Commission Method	1.19	1.11
Van der Veen Method	1.35	1.30
Predicted value		
Centre d'Etudes Ménard	1.08	1.04
Baguelin et al	1.20	1.15
Bustamante & Gianceselli	0.84	0.79

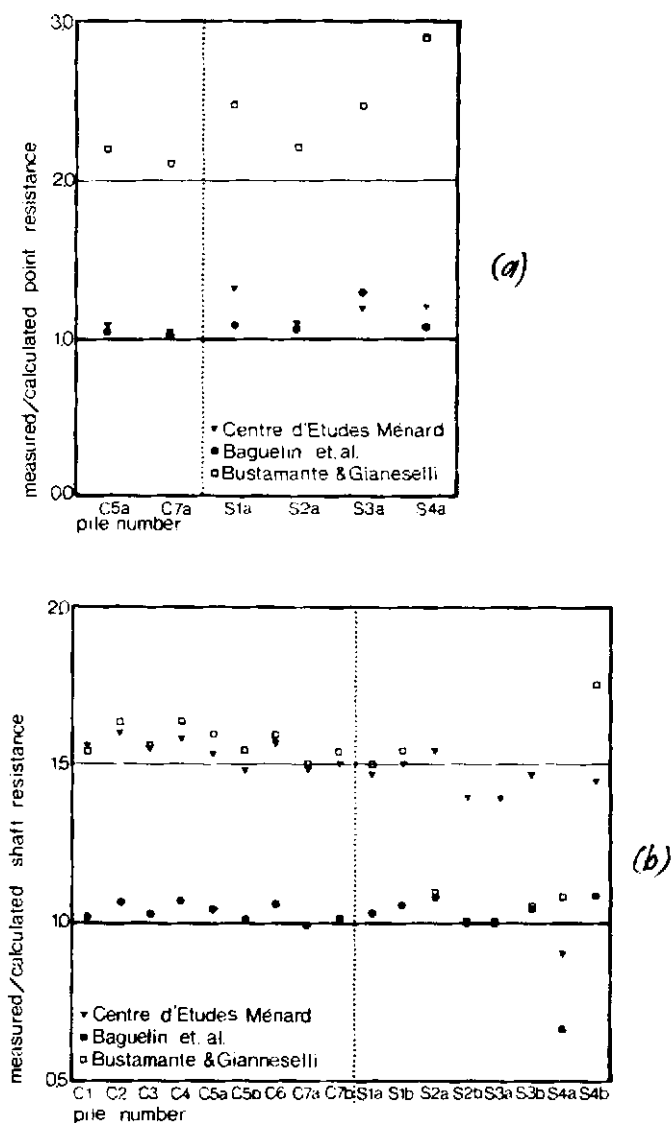


Figure 1. Ratio of measured and calculated bearing capacity of model piles. Pile diameter 0.15 m. The measured bearing capacity has been evaluated according to the method proposed by the Swedish Pile Commission.

- a) Point resistance (Data relate only to piles driven to 1 m depth in the sand)
- b) Shaft resistance

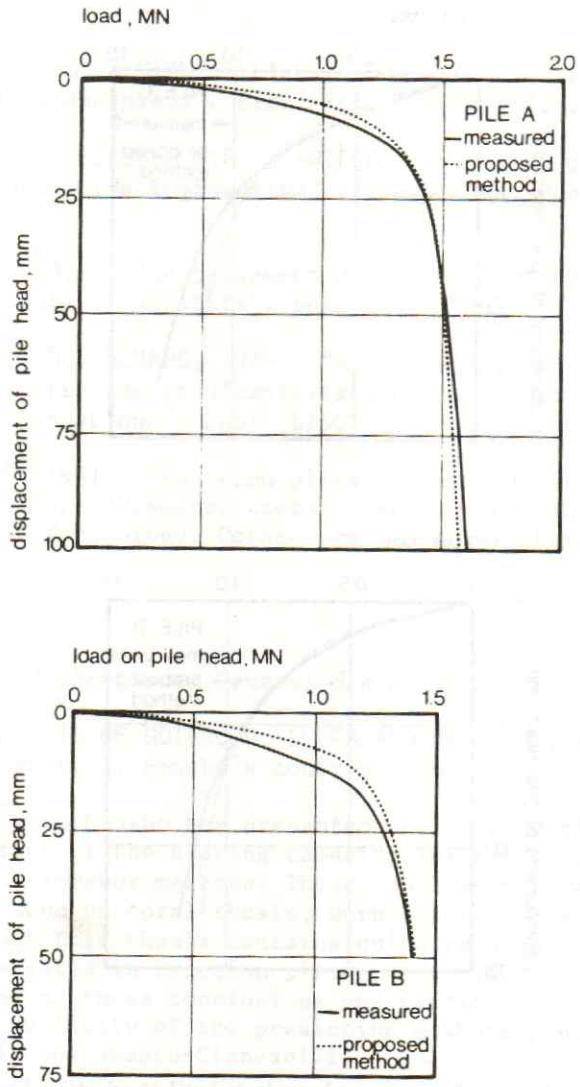


Figure 2. Comparison of measured and calculated displacements of pile head in the pile loading tests at Luleå.

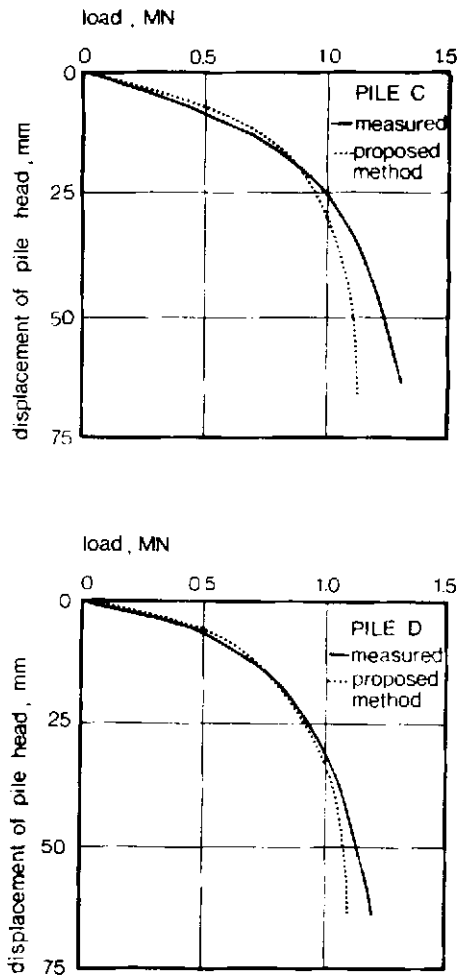


Figure 3. Comparison of measured and calculated displacements of pile head in the pile loading tests at Järnbrott.

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COMMENT by

M. BUSTAMANTE- (*Laboratoire Central des Ponts et Chaussées*)

A CRITICAL ANALYSIS OF GÖTEBORG, LULEÅ AND JARNBROTT PILE TESTING
(An answer to Prof. S. Hansbo's contribution).

Professor Sven Hansbo has presented the main conclusions related to the prediction of the bearing capacity for friction piles using different pressuremeter methods. These conclusions are based on a recently published Doctoral thesis, worked out by Eskil Sellgren (1981). Although this thesis contains quite remarkable results for engineers interested in friction piling, I would like to express my opinion on some of these conclusions and particularly on the ones concerning the validity of the predicting methods proposed by Ménard, Baguelin et al, Bustamante-Gianeselli.

GÖTEBORG MODEL TESTS

The shear data concerning the model tests obtained by Sellgren and presented by Prof. Sven Hansbo seem unquestionable. It is however highly dubious if these results can be directly extrapolated to the case of real piling.

What seems the most criticizable are the values of the bearing capacity factors k . Obviously they were greatly influenced by the

vicinity of the box walls and bottom. Sellgren is reporting that for piles driven to a depth of embedment of 1.5 m, the k values were abnormally high, ranging from 7.2 to 22.4.

To what extent the results obtained on piles driven only 1.0 m into the sand are exempt of wall and bottom effect seems rather a conjecture. The value k measured for this second series of piles, even if close of those proposed by Ménard or Baguelin et al. may not be necessarily representative. It will be even demonstrated in the next paragraph B, that to take into account these last k values can lead to large divergences between calculated Q_L and measured $\bar{Q}_L$ in some cases (see Luleå results in Table I).

At last, although not primary, another remark should be made concerning the calculation of unit friction according to Bustamante-Gianeselli proposals. These authors are also proposing a value of 1.2 MN for concrete driven pile into medium dense sand. It ought to be said however that the complete version of Bustamante method, published afterwards in Bulletin de Liaison number 113, was not available for Sellgren at the moment when he checked the validity and representativeness of proposals made by the different authors.

FULL SCALE TESTS

As mentioned by Prof. Sven Hansbo (see his contribution), 5 full scale tests (A_1 , A_2 , B, C, D) on 4 driven concrete piles (A, B, C, D) were carried out at Luleå and Järnbrott sites. A_1 , C and D were tests on isolated piles. A_2 and B were tests on pile set among a group for an abutment.

The observed load-displacement curves corresponding to the tests A_1 , A_2 , B, C and D are presented in Figures 2 and 3 of Prof. Sven Hansbo contribution.

A strict application of the Ménard, Baguelin-Jézéquel-Shields or Bustamante-Gianeselli predicting rules leads to the calculated bearing capacities Q_L presented in Table I, together with the measured bearing capacity $\bar{Q}_L$ obtained from testing.

Two examples of a detailed strict computation are given in the APPENDIX.

What should be emphasized in these two examples, corresponding to test B (Luleå site) and test C (Järnbrott site) is the fact that the measured soil density of the sands near the pile points exceeding $p_1 > 2.5$ MPa, the following values of k factor should be taken into account :

SESSION 4 ADDITIONAL CONTRIBUTIONS AND DISCUSSION

at least $k = 7$ according to Baguelin et al.

$k > 4.35$ (category 3) if the Bustamante proposals are strictly applied.

$k > 6$ (category III) according to Ménard.

Such k values considerably increase the ultimate point resistance Q_L^P in particular and are mainly responsible for a sizable increase of the total bearing capacity Q_L . It can be noticed that the predicted Q_L according to Bustamante-Gianeselli method, are in a quite good agreement with the measured corresponding ones, being not at all too conservative.

The Table I values are to be compared with the corresponding values in Table II, calculated by Sellgren and presented by Prof. Sven Hansbo. As it has been demonstrated above the calculated values of Table II do not seem to be the result of a strict application of the different authors proposals.

If strictly applied (see Table I) the Bustamante-Gianeselli prediction method appears at least for Luleå and Järnbrott piles as quite satisfactory and giving the best agreement between measured $\bar{Q}_L$ and calculated values, whereas the calculated Q_L according to Ménard or Baguelin-Jézéquel-Shields proposals can give sometimes too high values of Q_L , overestimating even dangerously the actual bearing capacity of a pile. Such surestimating is mainly due to taking into account too high values of k .

APPENDIX

LULEÅ - TEST B

Soils conditions :

$$p_{le} > 2,5 \text{ MPa} \quad p_o = 1,5 \quad p_{le}^* > 2,5 \text{ MPa}$$

Point resistance :

Ménard $k > 6,0$ (category III)

Baguelin et al. $k = 7,0$

Bustamante-Gianeselli $k = 4,35$ (category 3)

$$\text{Ménard } Q_L^P = 875 \text{ kN}$$

$$\text{Baguelin et al. } Q_L^P = 1097 \text{ kN}$$

$$\text{Bustamante-Gianeselli } Q_L^P = 790 \text{ kN}$$

TABLE I

BEARING CAPACITIES CALCULATED BY BUSTAMANTE

Site	Test	MENARD (kN)				BAGUELIN-JEZEQUEL-SHIELDS (kN)				BUSTAMANTE-GIANESELLI (kN)				Measured bearing capacity Q_L (kN)
		k	Q_L^P	Q_L^S	Q_L	k	Q_L^P	Q_L^S	Q_L	k	Q_L^P	Q_L^S	Q_L	
LULEÅ	(1) A1	5,7	669	882	1551	6,0	704	1125	1829	2,7	317	648	965	800 - 850
	A2	> 7,0	1276	742	2018	> 7,0	1276	1054	2330	4,35	790	930	1720	
	B	6,0	875	811	1685	7,0	1097	993	2090	4,35	790	872	1662	
JÄRNBROTT	(1) C	5,7	661	492 (632)	1153 (1293)	7,0	811	567 (707)	1378 (1518)	4,35	504	530 (670)	1034 (1174)	1250 - 1350
	(1) D	5,7	629	442 (582)	1071 (1211)	6,8	(751)	517 (657)	1268 (1408)	3,8	420	480 (620)	900 (1040)	

(1) Those values in brackets are taking into account an estimated (more probable) skin friction of 0.20 MN in the soft clay of Järnbrott.

TABLE II
BEARING CAPACITIES CALCULATED BY SELIGREN

Site	Test	MENARD (kN)			BAGUELIN-JEZEQUEL-SHIELDS (kN)			BUSTAMANTE-GIANESELLI (kN)		
		Q_L^P	Q_L^S	Q_L	Q_L^P	Q_L^S	Q_L	Q_L^P	Q_L^S	Q_L
LULEÅ	(*)									
	A1	580	760	1340	540	1030	1570	270	620	890
	A2	630	750	1380	620	950	1570	300	940	1240
JÄRNBROTT	B	630	700	1330	620	870	1490	300	840	1140
	C	580	500	1080	610	590	1200	360	480	840
	D	580	460	1040	610	540	1150	360	430	790

(*) The values for this test calculated by Selligren do not appear in the Prof. Sven Hansbo's contribution.

Skin friction :

$$\text{Ménard} \quad Q_L^S = 811 \text{ kN}$$

$$\text{Baguelin} \quad Q_L^S = 993 \text{ kN}$$

$$\text{Bustamante-Gianeselli} \quad Q_L^S = 872 \text{ kN}$$

JÄRNBROTT - TEST C

Soils conditions :

$$p_{1e} = 2,34 \text{ MPa} \quad p_o = 0,24 \text{ MPa} \quad p_1^* = 2,1 \text{ MPa}$$

Point resistance :

$$\text{Ménard} \quad k > 5,7 \quad (\text{category III})$$

$$\text{Baguelin} \quad k = 7$$

$$\text{Bustamante-Gianeselli} \quad k = 4,35 \quad (\text{category 3})$$

$$\text{Ménard} \quad Q_L^P = 661 \text{ kN}$$

$$\text{Baguelin et al.} \quad Q_L^P = 811 \text{ kN}$$

$$\text{Bustamante-Gianeselli} \quad Q_L^P = 504 \text{ kN}$$

Skin friction :

$$\text{Ménard} \quad Q_L^S = 492 \text{ kN}$$

$$\text{Baguelin et al.} \quad Q_L^S = 567 \text{ kN}$$

$$\text{Bustamante-Gianeselli} \quad Q_L^S = 530 \text{ kN}$$

A. PASQUET*

FINAL ADDRESS

Ladies and gentlemen,

As this Symposium comes to a close, I take pleasure in thanking all the participants (150 participants from 20 countries) and the authors of 22 papers, who so competently discussed the results of their research and experience. I also wish to congratulate the organizers, in other words the Institut Français du Pétrole and the Laboratoires des Ponts et Chaussées, as well as the Comité Français de la Mécanique des Sols et des Fondations, which sponsored the Symposium, because this event has been an undeniable success.

This success is due to the combination of two developments which have grown rapidly in the past two decades :

- . offshore oil production,
- . the pressuremeter method for soil reconnaissance.

I. OFFSHORE OIL PRODUCTION⁽¹⁾

In the past twenty years, offshore petroleum activities have grown to such an extent that world oil output from the ocean floor has been multiplied by a factor of 6, corresponding to an annual growth rate of over 9%. During the same period, onshore production was multiplied by 2.6, or an annual growth rate of 5%

⁽¹⁾Information provided at a Symposium on advanced techniques, held in Paris in March 1982.

* Conseil Général des Ponts et Chaussées, Paris, France.

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Henceforth, total output from offshore fields throughout the world (about 600 million tons in 1980) is equal to that of Saudi Arabia and Iraq combined, or half of the production of the OPEC countries.

According to apparently reasonable forecasts, offshore oil production will double by 1990, to reach 1.1 billion tons. It will account for 30% of total production instead of 21%. By this time, offshore oil worldwide will represent twice the power output of nuclear power plants (instead of five times today).

Moreover, geographic production areas will also be expanded. Until recently, production was concentrated on the continental shelf, in other words at a depth less than 200 meters (with 85% in less than 100 meters), and with the 100 to 200 meter depth interval located in the North Sea. Since 1975, exploration has begun on the continental slope, deeper than 200 meters, and depths up to 1000 meters are being planned for the future.

II. SOIL RECONNAISSANCE FOR THE INSTALLATION OF OFFSHORE STRUCTURES

The offshore installation of petroleum structures in increasing water depths requires the most accurate possible knowledge of the characteristics of the soils in place. Among soil reconnaissance techniques used on the continental shelf, drilling with core sampling is the most widespread. However, laboratory tests on core samples do not always serve to reproduce correctly the conditions in which the soils are found in place. In situ measurements offshore are even more valuable, because of the difficulty of obtaining undisturbed core samples. Hence the role of the pressuremeter which serves :

- . to establish stress/deformation relationship of the foundation soil to failure,
- . to analyze the behavior of the soil subjected to cyclic stresses produced by wave action.

III. BRIEF SUMMARY OF PAPERS IN THE SYMPOSIUM

In the First Session (Chairman Mr. Habib), various authors examined the evolution of the pressuremeter method, of which the principle was discovered in 1930 by Kogler and Scheidig, but which

was developed essentially by Louis Ménard who, starting in 1954, built and then improved the device, and in 1963 set forth semi-empirical rules applicable to foundation design. In particular, the results obtained using other instruments have been presented, including the single-cell pressuremeter (LLT) used by Oyo, the self-boring pressuremeter invented by Mori, and the automatic pressuremeter developed by the Research Institute on Foundations and Underground Structures in Moscow.

The Second Session (Chairman Mr. Parriaud) was devoted to the self-boring pressuremeter, which is inserted without significantly, disturbing the soil, thus improving results concerning deformability, both on land and on the seabed. The instruments discussed included the self-boring pressuremeter of the Laboratoires des Ponts et Chaussées, the self-boring pressuremeter probe of the Institut Français du Pétrole for offshore applications, and the Camkometer of the University of Cambridge. Other papers dealt with interpretation of tests by the Technical University of Torino and by the Norwegian Geotechnical Institute.

The Third Session (Chairman Mr. Delacour) dealt with various pressuremeters specially designed for offshore reconnaissance :

- . pressuremeter of the Building Research Establishment of the United Kingdom,
- . wire-line expansometer developed by the Syminex Company (Marseille),
- pressio-penetrometer of the Laboratoires des Ponts et Chaussées, used with an annular vibrodriver,
- . self-boring pressuremeter for shallow water, built by the Service Central Technique des Ports Maritimes et Voies navigables,
- . offshore self-boring pressuremeter for deep water, developed by the Institut Français du Pétrole, with the assistance of the Laboratoires des Ponts et Chaussées.

The Fourth Session (Chairman Professor Reese) showed how the pressuremeter results can furnish :

- . elementary mechanical parameters : horizontal soil pressure at rest, shear modulus, shear strength,
- . direct foundation design, using empirical formulas to evaluate the failure pressure and settlement, as well as piling design (lateral friction and reaction to horizontal forces).

This Symposium has pointed out several major factors.

- (a) The variety of research on the pressuremeter, both for the development of new instruments and for investigations into the

FINAL ADDRESS

interpretation of test results, is highly impressive today.

(b) Several types of pressuremeter are already operational for off-shore soil reconnaissance :

- . placed in a previously drilled borehole,
- . driven into the soil by jacking,
- . inserted into the soil by self-boring.

(c) Some opposition has emerged in this Symposium between researchers and practitioners :

- . the researchers appear to be too critical of various deficiencies in the accuracy of the processes used, but are perhaps not sufficiently aware of the limits of theoretical models ;

- . the practitioners have to solve complex problems, and new ones in relation to terrestrial geotechniques, such as the effect of cyclic stresses on soil behavior.

The solution to this dilemma certainly lies in the choice of realistic solutions between the desirable ideal and the practically feasible.

(d) The variety of instruments and pressuremeter tests incurs a potential risk for the future. An attempt to "standardize" routine tests, combined with a degree of "self-discipline" among researchers, seems indispensable to avoid the pitfalls which are detrimental to the spread of the pressuremeter method.

IV. SOME INDICATIONS CONCERNING COMPARATIVE COSTS OF OFFSHORE OIL PRODUCTION STRUCTURES FOUNDATIONS AND SOIL RECONNAISSANCE

The costs of offshore oil structures vary considerably in accordance with the water depth, type of platform and environmental conditions.

An example may be provided by considering only two jacket platforms of the North Sea and Persian Gulf type. The table given below offer only orders of magnitude.

	North Sea	Persian Gulf
Water depth	100 to 180 m	20 to 40 m
100-year wave	30 m	7 m
Jacket weight	20,000 t	5000 t
Piles :		
. number	20 (in groups)	4
. diameter	1.5 m	1.2 m
. shaft length	100 m	80 m
Cost of the structure :		
. jacket	\$100.10 ⁶	\$25.10 ⁶
. piles	\$5.10 ⁶	\$1.10 ⁶
. installation of piles	\$10.10 ⁶	\$1.10 ⁶
Ratio :		
<u>cost of foundations</u> cost of the structure	15%	7 to 10%

It may be noted that the cost of a jacket platform in the North Sea (without the deck and installations) amounts to around 100 million dollars, or about 700 million French francs. The foundations (piles and installation) represent 15 million dollars, or 105 million francs, or about 15% of the total cost of the structure.

Concerning the cost of soil reconnaissance before the installation of the structures, this amounts to 1 to 2% of the cost of the structures in the North Sea, but may be as high as 5% for small structures.

CONCLUSIONS

The few data that I have just given show that a saving in the cost of foundations can have a significant effect on the total cost of oil production structures. Since these structures apply to an activity which is already very important and growing rapidly, it is absolutely essential to improve soil reconnaissance methods in order to optimize foundation design and to evaluate the safety of these structures correctly.

FINAL ADDRESS

The papers presented and the discussions that ensued in the past two days prove that the pressuremeter has helped to achieve considerable progress in this direction. This is why I again wish to congratulate the organizers of this Symposium as well as all the engineers and researchers for their participation.

I declare the adjournment of the Symposium and thank you all for your attention.

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