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State-of-the-Art Report

IN SITU TEST AND INSTRUMENTATION IN SOFT CLAY DEPOSITS

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SUMMARY This state-of-the-art report describes some salient aspects of in-situ tests and instrumentation of soft clay deposits as experienced by the author in the last fifteen years. It contains a major contribution on the subject from the Norwegian Geotechnical Institute as well as the Asian Institute of Technology. The in-situ tests are classified as two groups; small scale tests and large scale tests. In the category of small scale tests, the usefulness of vane tests and Dutch cone tests are described in detail. In the case of large scale tests, full scale test embankments, test excavations and instrumented foundations are considered. Instrumentation aspects of the large scale tests are predominently obtained from the work of Dr. E. Dibiagio. Applications of probability techniques to quantify uncertainties in geotechnical engineering practice are emphasized.

INTRODUCTION

Soft clay deposits are extensively found in many countries in South and East Asia: Thailand, Malaysia, Singapore, Indonesia, India, Japan, Korea. There are also large deposits in Norway, Sweden, Zambia, Ghana, Canada, U.S.A. Mexico, Australia, New Zealand etc. During the last few decades extensive research has been carried out on the fundamental and applied aspects of soft clays in order that engineering structures can be built economically and safely on this soil profile. Typical structures include highway embankments, excavations, shallow and deep foundations for buildings, tunnels for transportation and water supply, slurry trench excavations for diaphragm walls, and deep foundations. In-situ tests and instrumentation have become an indispensible geotechnical tool for site investigation, evaluation of design parameters, monitoring of construction activities and for the evaluation of the performance of structures. In-situ tests have been used extensively to determine basic soil properties such as strength, compressibility, permeability, and the coefficient of earth pressure at rest. In addition, instrumentation had been used in the measurement of pore pressure, settlement, lateral movement, earth pressure, load, stress and strain. In this state-of the-art report, the author has chosen to discuss some special and salient aspects of the role of in-situ tests and instrumentation of soft clay deposits.

Field tests can be broadly divided into small scale tests and, perhaps large scale tests. In the cateogry of small scale tests, vane apparatus is used extensively for the measurement of in-situ strength to very great depths at close intervals. The data obtained from vane tests are used extensively in the analysis of embankments, excavations, and shallow and deep foundations. One advantage of vane testing is that several tests can be carried out economically at a site, compared to other types of field and laboratory tests. Thus, the apparatus can be used effectively in determining the spatial variation of the strength characteristics which can then be modelled as probability distributions. It is possible now to quantify the uncertainties involved in the evaluation of design parameters and their consequence on the safety of the designed structure. Vane tests are also used to evaluate specific characteristics of a deposit, such as anisotropy, where vanes of different sizes and shapes are used to determine the variation of strength with different inclinations of failure planes. Another in-situ test that is often carried out is the Dutch cone penetration test, wherein cone resistance and skin friction are measured separately. The data can be used directly to evaluate the carrying capacity of piles. Also, empirical correlations have been developed between cone resistance values and in-situ vane strengths. Piezometers of various kinds have been developed to measure in-situ permeability by both constant head and falling head methods. Recently, piezometers have been used in the hydraulic fracturing technique for the measurement of permeability as well as the coefficient of earth pressure at rest. Various kinds of pressuremeters have been developed in the past decade for the in-situ measurement of strength, deformation modulus and the coefficient of earth pressure at rest. These apparatus are increasingly used in South and East Asia, but their potential and usefulness have yet to be appraised critically.

In the category of large scale field tests, the Norwegian Geotechnical Institute under its first director, the late Dr. Laurits Bjerrum, pioneered in advocating large scale field tests for geotechnical engineering practice. For determining the basic properties, such as strength and in-situ stress, extensive work was conducted to develop a large scale direct shear apparatus in which a one-metre diameter clay core can be sheared at depths up to about 12 m in any desired plane of failure. A modified version of the slurry trench process has been used to isolate a large section of intact clay from its surroundings by the slurry which would subsequently fail under direct shear. Other common large scale field tests include test embankments and excavations, and full scale load tests on instrumented foundations. At the Asian Institute of Technology, Dr. Za-Chieh Moh and his colleagues, Dr. E.W. Brand and Dr. J.D. Nelson, conducted a comprehensive program of large scale field tests on Bangkok Clay. This work included test embankments taken to failure, and an embankment of sloping height where the settlements due to different amounts of surcharge were measured. A full scale test excavation was carried out at the same site.

Instrumentation of soft clay has been dealt with in an authoritative manner by Dr. E. Dibiagio and Mr.F. Myrvoll in their presentation of field measurement of soft clay during the International Symposium on Soft Clay held at Bangkok in 1977. Accordng to them the physical quantities measured in any instrumented project include force, stress, and strain, linear and angular displacements, and pressure. Using these quantities, the parameters evaluated include: surface and sub-surface ground movements; settlement, tilt and horizontal displacement of structures; stress, strain and load in structural elements; pore pressure in the ground and at boundaries of structures; and earth pressure at contact surfaces between soil and structures. The basic types of instruments for the meąsurement of geotechnical parameters are presented in relation to applications in strutted excavations, embankments, slurry trench excavations, piled foundations, tunnels, etc. Also, the selection and general arrangement of instruments for each project is outlined. The principal geotechnical problêms described.

Data from large scale field shear tests and from full scale instrumented tests on slurry trench excavations, embankments, excavations and piled foundations in soft clay deposits are presented to illustrate the role of in-situ tests and instrumentation in geotechnical engineering. Brief mention is made of methods of presenting data from large scale instrumented projects, and the usefulness of large scale tests in obtaining empirical correlations of performance in terms of safety factors with basic properties such as plasticity index of soft clays. The usefulness of numerical techniques in the analysis of performance of large scale instrumented test sections and their predictinns are compared to observed measurements. The role of probability theories in quantifying observed measurements in relation to design criteria is discussed.

BASIC PROPERTIES OF SOFT CLAYS

BJERRUM (1973) stated that during the early stage of the development of soil mechanics all soft clays were classified in one group, generally with the heading "normally consolidated clays" to distinguish them from "overconsolidated clays". The identification and classification of a soft clay depends on the geological history, the water content and the Atterberg limits, visual study based on clay fabrics, the in-situ strength and the compressibility characteristic based on oedometer results. BJERRUM (1973) classified soft clays as normally consolidated young clays, normally consolidated aged clays and overconsolidated clays. Most soft clays have a weathered upper crust and many of them can also be distinguished as quick clays, cemented clays, expansive clays etc. However, in all cases, the soft clays have low strength and high compressibility as a result of their high natural water content (see Fig. 1). The typical basic properties of a soft clay such as Bangkok Clay is given in Fig. 2. For all purposes of carrying out in-situ tests and instrumentation, the soft clays which are geographically distributed in all parts of the world need not be precisely distinguished.

SMALL SCALE IN-SITU TESTS

In the introductory remark of this state-of-the-art report the author

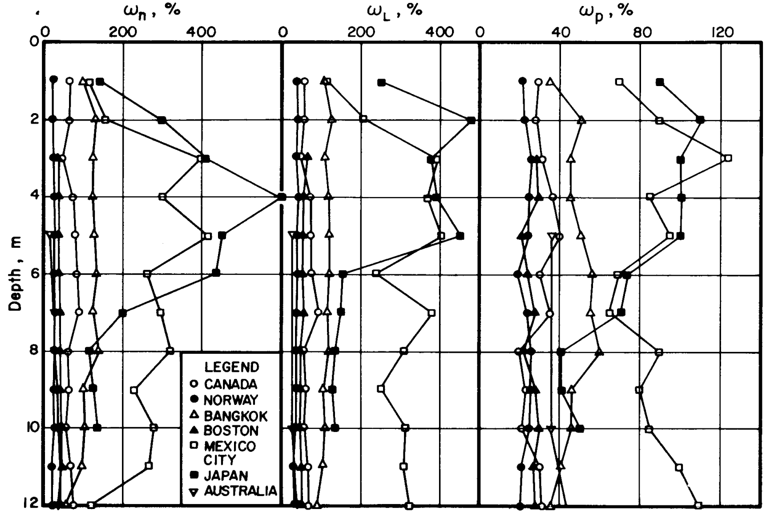


Fig. 1 - Index Properties and Natural Water Content of Soft Clays

distinguished in-situ tests as small scale tests and large scale tests. The commonly carried out small scale tests include vane tests, Dutch cone tests, pressuremeter tests and also the use of piezometers. Some salient features of each of this type of test will now be discussed.

Vane Tests

In 1948 Cadling developed the vane test for in-situ measurements of the undrained shear strength. By carrying out in-situ measurement, most of the uncertainties expected in the measurement of strength from laboratory tests due to the disturbance of the clay during sampling can be removed. Also, vane tests can be carried out at close intervals as small as 0.5 m depth. It has now become good practice in most countries to include a number of vane borings in any site investigation program on soft clays. Also, the number of boreholes at any site can be greatly reduced by the addition of a few vane borings. A few examples of the application of vane tests in geotechnical engineering practice will now be presented.

Site investigation. - Figure 3 illustrates the map of the lower section of the Nakhon Sawan Highway and the Bangkok-Siracha Highway (see EIDE & ANDRESEN, 1977). The soft Bangkok clay is a marine clay and as such is rather homogeneous. It was thus possible to map the vane strength along 60 km stretch of the highway by carrying out nearly 100 vane borings. Typical contours of the strength along the longitudinal section extending to a depth of nearly 20 m can be mapped as shown in Fig. 4. With such mapping of soil conditions it is evident that the number of boreholes and laboratory test program can be greatly reduced.

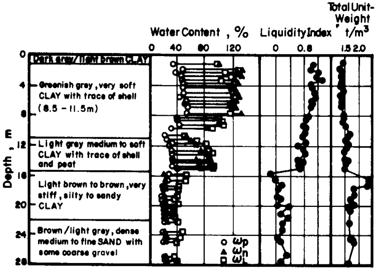


Fig. 2 - Index Properties of Bangkok Clay

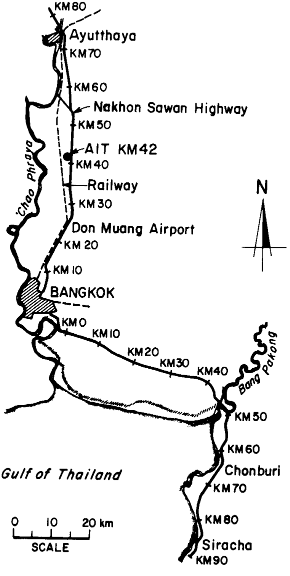


Fig. 3 - Nakhon Sawan Highway and Bangkok Siracha Highway
(After EIDE & ANDRESEN, 1977)

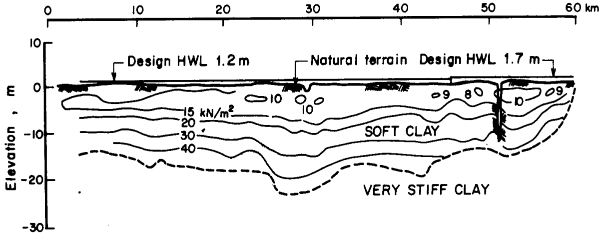


Fig. 4 - Contours of Vane Strength
(After EIDE & ANDRESEN, 1977)

Design of piled foundations. - The bearing capacity of a pile is usually estimated from end bearing and shaft adhesion, by using a static formula. Thus for piles driven only in clays, an expression of the form given in equation (1) can be used.

$$Q = N_c . s_u . A_p + \alpha \; s_u \; A_s \qquad \dots\dots (1)$$

when N_c is the bearing capacity factor normally assumed as nine (but taken as 10 for Bangkok Clays, see HOLMBERG, 1970) and α is the adhesion factor. s_u refers to undrained strength and A_p and A_s are the area of cross section of pile point and the shaft area respectively. Throughout the years a number of papers have been written to present the correlations between undrained shear strength and the adhesion factor, α. Figure 5 illustrates the relationships between the adhesion factor and the undrained vane strength as compiled by BJERRUM (1973). The design practice of piled foundations in the greater Bangkok plain area has been largely empirical. Load tests have been utilised for the carrying capacity of piles for important structures. A comprehensive study made recently on more than 30 test piles from eleven projects illustrate that vane strength data can be used in the soft clay for the estimation of point resistance and shaft adhesion. The details of the test piles are given in Table 1 and the computed values are presented in Figs. 6 and 7. The piles were separated into two groups: short pile group and long pile group. In both cases the test load values of the bearing capacity is 0.7 and 0.9 times the computed values. In the cases when the long piles were bearing in stiff clay and also in the sand layer, alternate expressions are used for the estimation of end bearing capacity and also the shaft carrying capacity. However, in the soft clay region, data from vane tests were used to estimate the point bearing load and also the shaft adhesion.

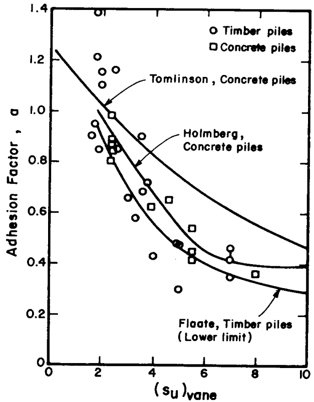


Fig. 5 - Adhesion Factors for Soft Clays (After BJERRUM, 1973)

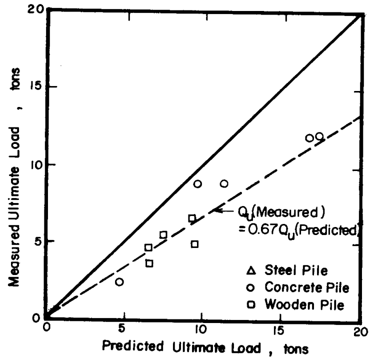


Fig. 6 - Measured and Predicted Bearing Capacity of Piles (Short Piles)

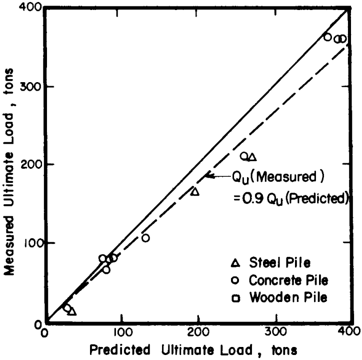


Fig. 7 - Measured and Predicted Bearing Capacity of Pile (Long Piles)

Bearing capacity of footings. - BJERRUM (1973) indicated that in principle the bearing capacity of footings in soft clay can be evaluated from the expression

$$Q_f = N_c s_u + \gamma D \quad \ldots\ldots \quad (2)$$

where N_c is the bearing capacity factor and s_u is the undrained strength average over a depth of about two-third the width of the footing. γ is the total unit weight of soil and D is the depth of the base of the footing. The N_c values depend on the shape of the footing and its depth below the surface and is taken from SKEMPTON (1951).

Stability of strutted excavations. - During the excavation in soft clays bottom heave failures should be avoided. A procedure for the calculation of the safety against bottom heave failure as proposed by BJERRUM & EIDE (1956) is

$$F = \frac{N_c . s_u}{\gamma D} \quad \ldots\ldots \quad (3)$$

where F = safety factor, D is the depth of excavation, γ is the total unit weight, s_u, is the undrained strength and N_c is a coefficient that depends on the dimensions of the excavations. Thus vane strength data could be used to investigate the safety of strutted excavations against bottom heave failure in soft clays.

Stability of embankments and excavations. - Vane strength data are also commonly used in evaluating the stability of embankments and excavations under total stress conditions. BJERRUM (1972) has proposed correction factors to be used in the field values of vane strength as

$$(s_u)_{field} = (s_u)_{vane}\ \mu_R\ \mu_A \qquad \ldots\ldots (4)$$

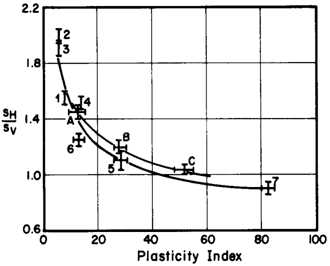
where μ_R is a factor that depends on time and μ_A is a factor which depends on the anisotropy of the clay.

Anisotropy of strength. - AAS (1965 and 1967) was perhaps the first author to have made a detailed study of the measurement of anisotropic strength in the field using vanes of different sizes and shapes. BJERRUM (1973) has subsequently compiled the data to show how the ratio S_h/S_V can be related to the plasticity index of clays (Fig. 8), S_h refers to the strength on a horizontal plane of failure and S_v refers to the strength on a vertical plane of failure. In the analysis of the stability of embankments and shallow foundations, a good proportion of the failure arc lies closer to the horizontal direction. Thus an estimation of the accurate undrained strength along a horizontal plane of failure in an anisotropic clay deposit is of considerable importance in the design of embankments and also shallow foundations.

Dutch Cone Tests

The cone penetration test was developed in the Netherlands as early as 1930 and has been in use extensively in many parts of the world. The cone penetration test has been in use for many years for the estimation of the extent of soft clay layers. Recent improvements with the use of the electrical measuring devices have enabled the successful correlation of the cone resistance to the undrained strength. The accuracy of the strain gauge type of penetrometer, however, depends on very careful temperature compensations and calibrations. A friction sleeve is often included in the standard cone penetrometer, which is primarily used to identify the soil (SANGLERAT, 1972).

The original Dutch method of cone penetration is a mechanical device which also incorporates a friction sleeve (BEGEMANN, 1951). Such a mechanical device is used extensively in the Bangkok plain by the Asian Institute of Technology. The advantage of the method is its simplicity and also the empirical experience that has been gathered over its use for many years. There are of course many disadvantages and the major one seems to be the friction which occurs between the inner rods and the outer pipes and also in the cone itself. However, empirical correlations can always be made with the measurements as obtained from a specific type of equipment and full scale tests data. It should then be understood that the correlation factor established incoporates the inaccuracies and as such should not be used when measurements are carried out with a different apparatus. Some of the experience gained in the use of Dutch cone data in evaluating the bearing capacity of driven piles in the Bangkok clays will now be presented.



PROCEDURE	SYMBOL	TYPE OF CLAY
FIELD VANE TEST WITH VANES OF DIFFERENT H/D RATIOS	1	MANGLERUD (SILTY QUICK)
	2	LIERSTRANDA (SILTY QUICK)
	3	KJELSÅS (QUICK)
	4	LEAN DRAMMEN (SILTY SENSIT.)
	5	PLASTIC DRAMMEN
	6	LEAN DRAMMEN (DEEP)
	7	BANGKOK
DIRECT SIMPLE SHEAR TEST	A	LEAN DRAMMEN
	B	PLASTIC DRAMMEN
	C	SKÅ - EDEBY

Fig. 8 - Anisotropic Strength of Soft Clays
(After BJERRUM, 1973)

Design of driven piles. - The methods by which the bearing capacity of a pile can be calculated from Dutch cone penetration tests have been modified with increasing experience. The end bearing is estimated from the product of the area of the pile tip and the average cone resistance around the pile tip. VAN DER VEEN & BOERSMA (1957) found that the end resistance is the average of the measured cone resistance over the average (a + b)d and the most probable values were a = 3.75 and b = 1. The skin friction on the pile can be estimated from the cumulative local friction at the pile tip multiplied by the pile perimeter. The formula used in calculating the ultimate load is

$$Q_u = \alpha . P . q_{Tf} + \lambda q_c A_p - W \qquad \text{..... (5)}$$

where P = perimeter of pile (m)
q_{Tf} = cumulative local friction (t/m perimeter)
α = friction factor (dimensionless)
λ = bearing factor (dimensionless)
A_p = area of pile tip (m2)
q_c = average cone resistance (t/m^2)
W = weight of pile (t)

Table 2 contains the friction factor α and bearing factor (λ) as obtained by various authors. Figures 9 and 10 contain the observed and measured pile loads as estimated from the Dutch cone results. Up to now several types of analysis have been conducted on the bearing capacity of driven piles in the Bangkok plain. They include total stress analysis using static formulae, the effective stress analysis, wave equation analysis, use of pile driving formulae, etc. However, the best correlation seems to be the one correlated with the Dutch cone test data. (PHOTA-YANUVAT & BALASUBRAMANIAM, 1979).

Table 2 - Friction Factors and Bearing Factors for Driven Piles in Bangkok Area

Investigator	α				λ	
	Soft Clay	Medium Stiff Clay	Stiff Clay	Sand	Clay	Sand
PHAM, (1972)	1.4	1.4	0.7	-	0.33	1.0
JUTA-SIRIVONGSE (1972)	1.0	1.0	1.0	1.0	0.33	1.0
CHOTIVITTAYATHANIN, (1977)	1.0	0.7	0.5	0.5	0.33	0.5
PHOTA-YANUVAT, (1979)	1.0	0.7	0.5	0.8	0.33	0.5

Use of Pressuremeters

The pressuremeter concept was perhaps first developed back in 1933 in the USSR. MENARD (1971) seemed to have developed an improved version and has also advanced the theoretical concepts. However, the equipment was little used in soft clays for the possible fear of disturbance due to its installation and operation. Self boring pressuremeters were subsequently developed by WROTH & HUGES (1973) and BAGEULIN & JEZEQUE (1973). The pressuremeters have so far been not used extensively in Southeast Asia and the author is not aware of any comprehensive study being made to make a critical evaluation of the potentials of this equipment. Recently, LLT pressuremeters developed by the Oyo Corporation are in use in Southeast Asia and it is hoped that enough

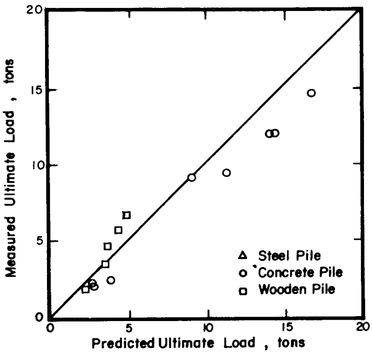


Fig. 9 - Measured and Predicted Bearing Capacity of Piles (Short Piles)

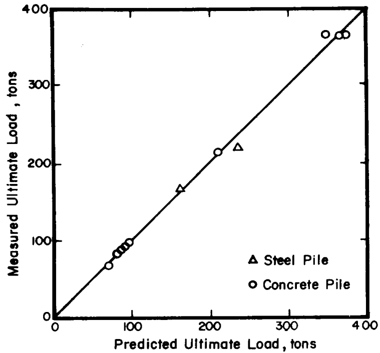


Fig. 10 - Measured and Predicted Bearing Capacity of Piles (Long Piles)

experience would be gained in the near future, so that contributions could be made in an authoritative manner to evaluate their performance. Self boring pressuremeters are claimed to be used successfully for the measurement of the coefficient of earth pressure at rest, K_o. However, in most instances, the measured K_o values seem to be in good comparison with the laboratory measured values of K_o which are known before hand prior to carrying out self boring tests.

Use of Piezometers

Permeability tests in situ are normally measured by installing piezometer Local disturbances around the piezometer can cause difficulty in making accurate permeability measurements. Often, the laboratory and in-situ values of permeability seldom agree. Also, the permeability in the field in the horizontal direction may be higher than that in the vertical direction. Care should be taken to note that moderate pressure heads are used to avoid the possibility of hydraulic fracturing. Expressions for the evaluation of permeability from piezometer tests seem to still rely heavily on the classical work of HVORSLEV (1951).

BJERRUM & ANDERSEN (1972), were the first to have used the hydraulic fracturing technique in soft Norwegian clays for the measurement of in-situ lateral pressure and hence, K_o, the coefficent of earth pressure at rest. The method can only be used in deposits where K_o values are less than one-namely normally consolidated clays. Recently, the method was used in Bangkok clays and is found to give values of K_o which are in close agreement to those measured by other laboratory methods. Figure 11 illustrates a schematic sketch of the arrangement used in hydraulic fracturing technique. The measured values of K_o for Bangkok clays are plotted in Fig. 12 with other values of K_o obtained from laboratory tested samples. Hydraulic fracturing techniques have previously been used in rock mechanics (see HAIMSON & FAIRHURST, 1970) and for permeability measurements at the Dead Sea, Israel by BJERRUM et al (1972).

LARGE SCALE FIELD TESTS

As stated in the introduction of this state-of-the-art report large scale field tests can be of a type where basic property such as the shear strength is measured from large scale direct shear tests or they can be of the type when full scale field tests are carried out in relation to site investigation, design and construction of major projects. The latter type of full scale instrumented tests will of course involve a large number of sophisticated instruments and that is one of the reason why in this state-of-the-art report, in-situ tests and instrumentation are considered together as one topic. It is therefore, considered that large scale direct shear tests will only be considered in this section and subsequently at a later section full scale instrumented tests will be presented.

Large Scale Direct Shear Tests

A new piece of field equipment has been constructed at the Norwegian

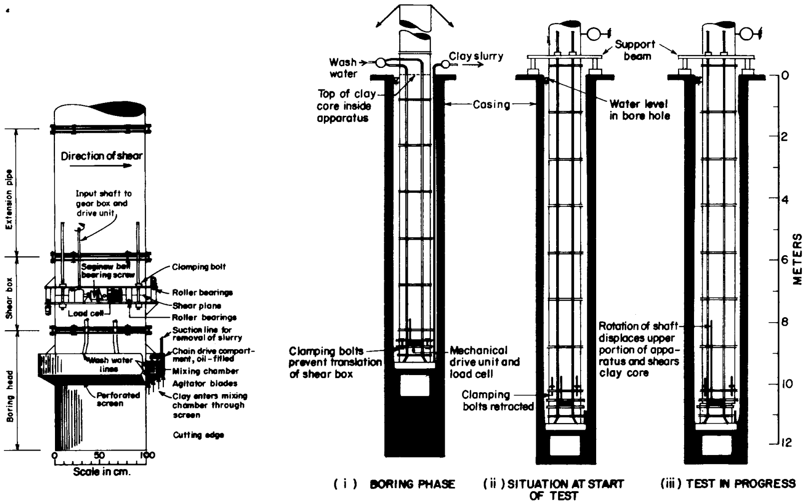


Fig. 13 - Large Scale Direct Shear Apparatus (After DIBIAGIO & AAS, 1967)

the upper half of the shear box can translate horizontally up to 5 cm relative to the bottom half. The force necessary to shear the specimen is developed by mechanical means from the ground surface. The force transmitted to the specimen is measured by vibrating wire gauge load cells.

The force displacement curve obtained from the apparatus at one site is shown in Fig. 14 (see BALASUBRAMANIAM & DIBIAGIO,1969). The shear strength obtained from the device is plotted with respect to vane strength in Fig. 15

While the apparatus offers a mean of measuring the shear strength on a horizontal plane of failure, errors can be involved in the testing due to friction in the apparatus. Also, the central portion of the clay core may have the possibility of standing like a tree without being sheared, while all the shear takes place along the circumference. Of course, the cost involved in carrying out such tests are enormously high and often far in out of proportion to the standard tests such as vane tests used in day to day practice.

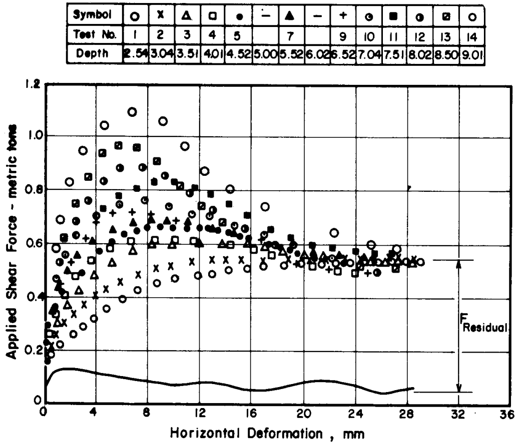


Fig. 14 - Force-Displacement Diagrams from Large Scale Direct Shear Apparatus

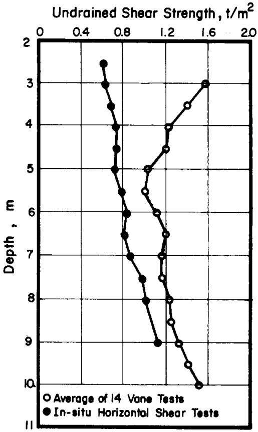


Fig. 15 - Undrained Strengths from Large Scale Shear Tests and Vane Tests

In that sense, if anisotropy is of major importance in soft clay deposits, a cheaper and simple device to measure the shear strength on a horizontal plane of failure need to be invented in a manner similar to the standard vane tests now carried out extensively for the measurement of the shear strength along a vertical plane of failure.

Slurry Trench Method of Shearing Clay Core

The late Dr. Laurits Bjerrum also initiated a program of tests in which a large section of clay core isolated from its surroundings by the slurry trench process was subsequently sheared under direct shear conditions. Recently, KARLSRUD (1979) have presented some data from these novel tests and compared them with the results obtained from triaxial tests. The conclusion was that the in-situ strength is significantly higher than those determined in the laboratory by stress-controlled triaxial tests.

It is of interest to mention here that when braced excavation in the Chicago clay was made as early as 1940 by Prof. Ralph Peck, the equipment used for the measurement of strut loads was simple hydraulic jacks. In the last four decades and especially between 1970 to 1980, instrumentation in soft clay deposits as well as in other geotechnical engineering fields has expanded both in quality as well as in quantity in an exponential manner. While such an advancement, certainly is of beneficial to the geotechnical engineer in enabling him to choose the most appropriate type of instrumentation for his project from a wide variety of alternatives, it also poses the serious question of how to decide on the selection of an equipment from such an enormous variety and quality. In that respect, instrumentation cannot be considered as an exercise where mechanical, hydraulic or any other sophisticated device is merely used for physical measurements. DIBIAGIO & MYRVOLL (1977) rightly attempts to define instrumentation as a combination of activities consisting of part philosophy and know-how, part insight and experience, and part measurement technology and devices, that are used to obtain qualitative and quantitative information needed to assess or solve a geotechnical problem on hand. They further go on to say that if the results of an instrumentation program are of any value to the practical engineer and also for the advance of the state-of-the-art, three essential conditions must be satisfied:

(i) The purpose of the instrumentation program in relation to the underlying geotechnical problems must be defined and understood.

(ii) The instrumentation program must be comprehensive enough and be carefully planned to include all necessary observations and measurements, and

(iii) The data collected during the course of an instrumentation program must be reduced to a convenient form, interpreted and analysed by competent engineers and the results must be made available to the profession.

The three conditions stated above by DIBIAGIO & MYRVOLL (1977) are indeed a necessity to the success of the use of instrumentation in any project not necessarily be in geotechnical engineering.

Objectives in Instrumentation

DIBIAGIO & MYRVOLL (1977) mention that instrumentation programs are generally initiated either as a part of a research project or as a means of obtaining information necessary for the successful design and excavation of a construction project, or to verify that the performance of a completed engineering work is acceptable and is in agreement with the predicted behaviour. Accordingly, instrumentation programs are used in site investigation and evaluation of soil parameters, construction control, performance measurement and full scale tests.

Parameters Measured with Instrumentation

According to DIBIAGIO & MYRVOLL (1977) the parameters generally measured in geotechnical problems related to soft clay are

(i) Surface and sub-surface movements.

(ii) Settlement, tilt and horizontal displacement of structures.

(iii) Stress, strain and load in structural elements.

(iv) Pore pressure in the ground and on boundaries of structures, and

(v) earth pressure on contact surfaces between soil and structure.

These parameters are evaluated from observations made on force, stress and strain, linear and angular displacement and pressure.

Selection of Instruments

With the abundance of geotechnical instruments now available, the selection of relevant instruments for any particulr project will depend on many factors. However, DIBIAGIO & MYRVOLL (1977) recommends that the selection of the most preferable instrumentation system and components will depend on (i) the cost involved, (ii) the required accuracy, (iii) the environmental and operational conditions and (iv) the qualifications of the personnel who are to install and use the proposed instrumentation. A comprehensive presentation of the basic instrumentation systems used is presented in DIBIAGIO & MYRVOLL (1977) and readers are requested to refer to this authoritative article on 'field instrumentation for soft clay'. Mentioned briefly here, tell-tale rods, Borros anchor, full profile measurements and borehole extensometers are some of the devices used in the measurement of vertical displacements (see Fig. 16a). Similarly, inclinometers, horizontal extensometers and off-set measurements are used in the measurement of lateral movements, (see Fig. 16b). Calibrated hydraulic jacks, strain gauges, extensometers, load cells and embedded strain gauges are used for the measurement of loads and strains in structural elements (see Fig. 16c). Pore pressures in the field are generally measured with open stand pipe, piezometers of various types such as the electrical piezometers, the pneumatic piezometers and the closed hydraulic piezometers (see Fig. 16d). Closed hydraulic piezometers have the added advantage to de-air them, especially when they are installed in soft clays with organic contents when often gas and air bubbles are evolved which clogg the measuring system. Finally, for the measurement of earth pressures, membrane type of earth pressure gauges and capsule-type of pressure gauges are available. Sometime integrated pressure measurements are made in the form of force, especially strut loads in braced excavations (see Fig. 16e).

DIBIAGIO & MYRVOLL (1977) have considered a wide range of projects in which instrumentation of various kinds has been used. These projects include strutted excavations, embankments, tunnels, tied back anchors, piles and slurry trench excavations. For each project they have also identified the major geotechnical problem. For example, the geotechnical problem related to strutted excavation can be bottom heave failure, strut loads, settlements, horizontal movements and pressure on the sheatings. Thus a typical instrumentation scheme may involve heave gauges, surface settlement gauges, subsurface settlement gauges, piezometers, inclinometer casings, strain gauges, earth pressure cells, etc.

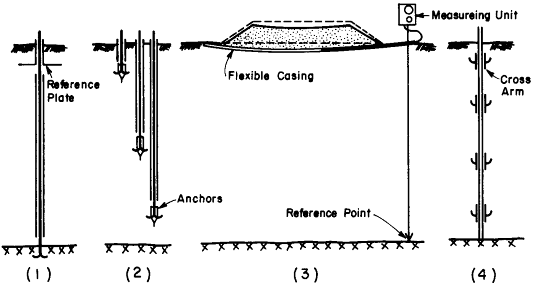


Fig. 16(a) - Gauges for Measuring Vertical Displacements (After DIBIAGIO & MYRVOLL, 1977)

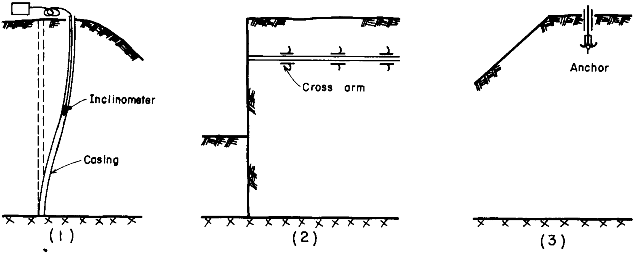


Fig. 16(b) - Measurements of Lateral Movements (After DIBIAGIO & MYRVOLL, 1977)

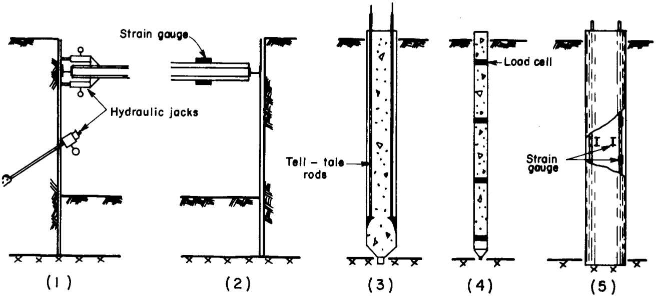


Fig. 16(c) - Gauges for Measuring Loads and Strains in Structural Elements (After DIBIAGIO & MYRVOLL, 1977)

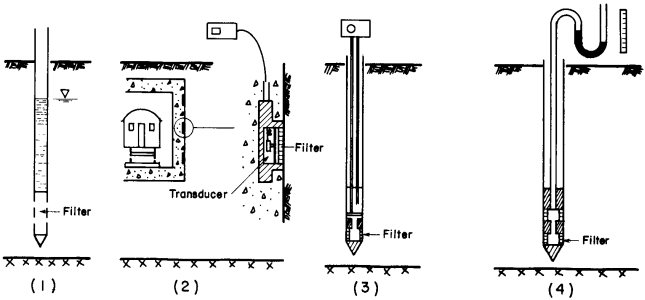


Fig. 16(d) - Piezometers for Pore Pressure Measurement (After DIBIAGIO & MYRVOLL, 1977)

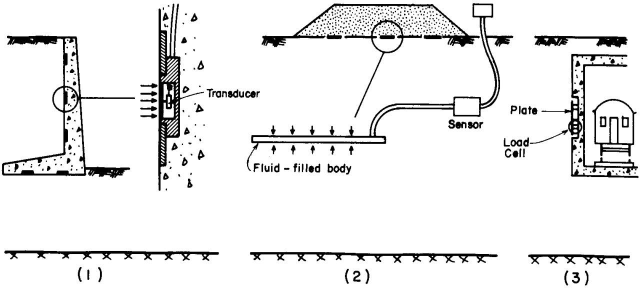


Fig. 16(e) - Earth Pressure Cells
(After DIBIAGIO & MYRVOLL, 1977)

In this section on instrumentation, the author has drawn his contribution mainly from the work of DIBIAGIO & MYRVOLL (1977) who have only recently completed a state-of-the-art report on field instrumentation for soft clay for the International Symposium held on Soft Clay in 1977 at the Asian Institute of Technology.

In the next sections the author will consider some case histories in which large scale instrumented projects were used and the data is expected to be used in site investigation and design of major projects related to a proposed new airport in Bangkok.

FULL SCALE TEST EMBANKMENTS AND TEST EXCAVATION

This section is fully devoted to the stabilty and settlement of two test embankments and a test excavation made in the soft Bangkok Clay. Generally, clay deposits known as Bangkok clay rest over sand and gravel beds with some sandy clay which occurs alternately to a depth of at least 300 m. Bangkok Clay can be divided into three different zones according to the strength and deformation characteristics. They consist essentially of an upper weathered zone of soft clay 4 to 5 m thick behaving as an apparently overconsolidated clay. This weathered crust is followed by a highly compressible soft clay occurring to an approximate depth of 10 m. The third zone is stiff clay occurring to an approximate depth of 15 m over sand and gravel beds. Several test embankments were built on Bangkok Clay to study the in-situ behaviour on a large scale. Of these embankments at least three of them are fully instrumented. Two embankments are shown in Figs. 17 and 18. These embankments are built in association with a proposed new international airport while the third embankment (see BALASUBRAMANIAM et al 1980) is constructed to obtain design information for a Naval Dockyard at Pom Prachul in the Samut Prakarn province approximately 20 km south of the city of Bangkok at the mouth of the Chao Phraya river. The embankment in Fig. 17 built at the Nong Ngoo Hao site (40 km from Bangkok) is of constant height and was instrumented with piezometers, settlement gauges and failure stakes. It has been taken to failure to study the stability of the embankment under different types of strength parameters. Embankment II in Fig. 18 is of sloping height and was built to study the effect of surcharge of different magnitude. The third embankment presented in BALASUBRAMANIAM et al (1980) was used to study the effectiveness of sand drains in Bangkok clay.

A test excavation was also constructed at the Nong Ngoo Hao Site. The test excavation was 80 m long x 25 m wide at the ground surface and was excavated to a depth of 4 m using a side slope of 2.5:1, as shown in Fig. 19. The ground was fully instrumented with settlement points, lateral deformation stakes and piezometers before the excavation was formed over a period of 31 days. A berm was provided on one side of the excavation to ensure failure on the side of the instrumentation.

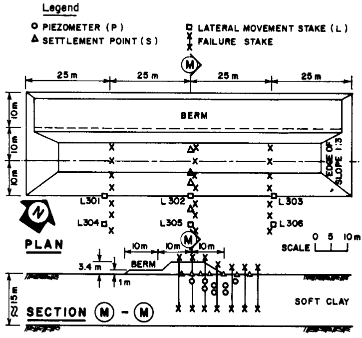


Fig. 17 - Full Scale Test Embankment I

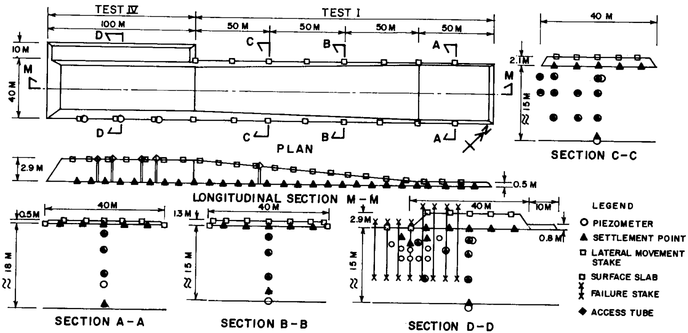


Fig. 18 - Full Scale Test Embankment II

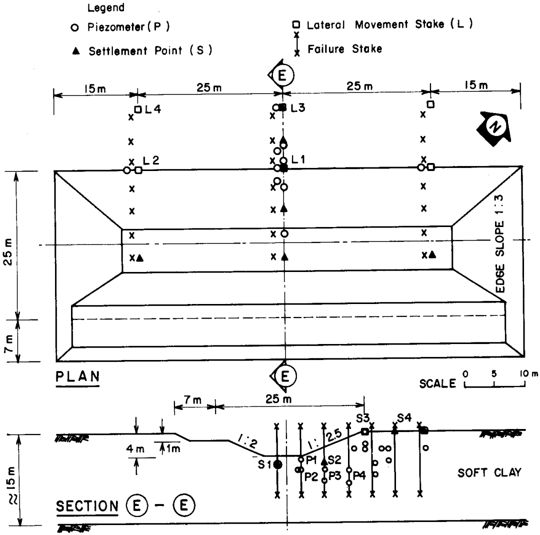


Fig. 19 - Full Scale Test Excavation

General Description of Nong Ngoo Hao Clay

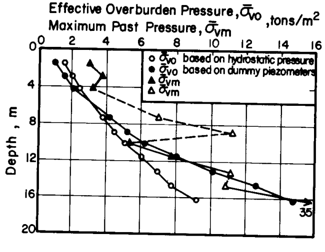
The sub-soil conditions at the Nong Ngoo Hao site is essentially the same as those already presented at other sites by MOH et al (1960), MUKTABHANT et al (1966) etc. According to the strength and compressibility characteristics the upper clay close to the surface has been divided into three zones, (i) a weathered zone extending to a depth of 4 m below the ground surface, (ii) a highly compressible soft clay occurring to a depth of about 10 to 12 m below the bottom of soft clay. The general properties and the strength and compressibility characteristics of weathered clay and soft clay are given below.

Property	Weathered Clay	Soft Clay
Natural Water Content (%)	133 ± 5	112 - 130
Natural voids Ratio	3.86 ± 0.15	3.11- 3.64
Degree of Saturation (%)	95 ± 5	80- 108
Specific Gravity	2.73	2.74
Liquid Limit (%)	123 ± 2	118± 1
Plastic limit (%)	41 ± 2	43± 1
Dry Unit Weight (lb/ft^3)	36.2	40.5
Organic Matter (%)	4.0	3.6
Grain Size Distribution		
Sand (%)	7.5	4.0
Silt (%)	23.5	31.7
Clay (%)	69.0	64.3

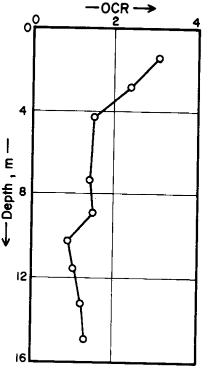
The in-situ stresses and the OCR values of the samples are plotted in Fig. 20. The variation of the voids ratio with respect to effective vertical stress for all three types of clays are presented in Fig. 21.

Statistical Evaluation of Strength and Compressibility

The uncertainties involved in soil engineering are due to many causes including variations in the soil properties, variation of loading conditions



(a) In-situ Stresses



(b) OCR Values

Fig.20 - In-Situ Stresses and OCR Values for Bangkok Clays

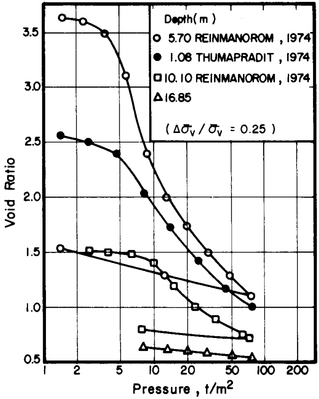


Fig. 21 - Voids Ratio-Pressure Relationships
for Bangkok Clays

and uncertainty in the mathematical model. Uncertainties in the soil property is by far the greatest no matter how comprehensive an investigation is made. Undrained strength measured from the field and laboratory tests were analysed see (BALASUBRAMANIAM et al 1979). The strength data were obtained from (i) field vane tests (ii) Dutch cone tests and laboratory tests.

Empirical distributions have been determined for Q_c and Q_{LF} (cone resistance and local friction) from Dutch cone tests and for s_u from field vane tests at intervals of 1 m up to 12 m depth. In an attempt to fit theoretical curves the χ^2 goodness of fit (at 5% significance level) test for normality rejected the normal distribution for all three parameters. Probable distributions were then determined using the Pearson's moment planes, where the moment of skewness is plotted against the coefficient of kurtosis. They indicate the possibility of the distribution being fitted to a J type β distribution for cone resistance, Q_c. A similar trend towards a β distribution was observed for the field vane strength too. Q_c, Q_{LF} and s_u are regressed with depth using a polynomial regression model to obtain a higher order equation to take into account non-linearity in the profiles. The regression

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Statistical Evaluation of Strength and Compressibility

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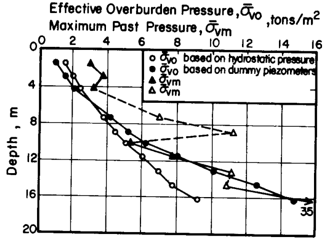
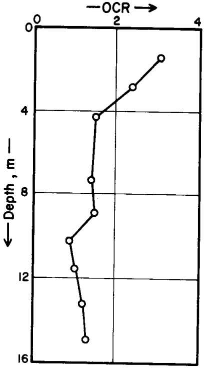


(a) In-situ Stresses



(b) OCR Values

Fig.20 - In-Situ Stresses and OCR Values for Bangkok Clays

Table 3 - Regression Equations for Q_c, Q_{LF}, and s_u at Nong Ngoo Hao site

Parameter			Regression Equation (d = depth in m.)	F-Value
$Q_c(t/m^2)$	Q_c	=	$30.02 - 5.89\,d + 1.19\,d^2 - 0.04\,d^3$	206.73
$Q_{LF}(t/m^2)$	Q_{LF}	=	$1.23 - 0.46\,d + 0.11\,d^2 - 0.009\,d^3 + 0.0003\,d^4$	70.87
$s_u\ (t/m^2)$	s_u	=	$2.393 - 0.564d + 0.091d^2 - 0.003\,d^3$	226.83

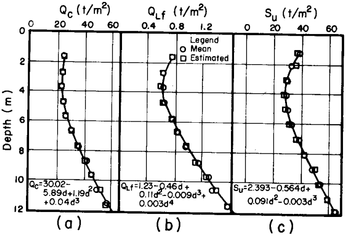


Fig. 22 - Cone Resistances and Vane Strengths for Bangkok Clays

equations at Nong Ngoo Hao for the three parameters are given in Table 3. Figure 22 shows the plot of actual mean values of the observations and the estimated values from the equations for Q_c, Q_{LF}, and s_u. From the above analysis of Q_c and s_u it is possible to find a regression equation relating cone resistance and vane strength. The results for Nong Ngoo Hao Clay can be expressed as

$$S_u(t/m^2) = 0.51 + 0.042\,Q_c(t/m^2) \qquad ..... \quad (6)$$

The coefficient of correlation is 0.93.

Bangkok Clay has been tested by various investigators to determine the compressibility characteristics. The main source of information is the research carried out at the Asian Institute of Technology. A typical (e, log $\bar{\sigma}_v$) curve for Bangkok Clay exhibits an initially less sloped portion which

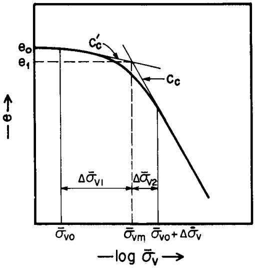


Fig. 23 - Voids Ratio-Pressure Model for Bangkok Clay

corresponds to an apparently overconsolidated state. This apparent overconsolidated range is well pronounced for weathered Bangkok Clay and is less prominent in soft Bangkok Clay. Since the common stress range for which settlement calculations are made falls in the shallow portion of the (e, log $\bar{\sigma}_v$) characteristic, a modified parameter C_c' corresponding to the slope in the overconsolidated range of stresses is also employed. Thus a typical (e, log $\bar{\sigma}_v$) model used in the analysis and containing the parameter C_c' and C_c are shown in Fig. 23, where $\bar{\sigma}_{vm}$ corresponds to the maximum past pressure.

The compession index C_c' of the overconsolidated range was determined for all the consolidation test data. The justification for the introduction of this index was tested statistically using correlation profiles and factor analysis. The data analysis of the index C_c showed a wide variation in the mean value of C_c with depth. Due to the lack of sufficient number of data at a particular depth, the determination of the distribution of C_c was found to be difficult. Thus the coefficient of variation approach given by KAY & KRIZEK (1971), is preferred in the analysis. The key factor in this approach for estimating the additional information from past data can be used effectively. For every set of experimental results on the parameter, e.g. C_c, the coefficient of varition (= ratio of the standard deviation and the mean) can be determined. There is experimental evidence to confirm that, if the soil within a given layer is of a similar type and has a reasonably consistent geological origin, then the coefficient of variation will be restricted to a narrow limit even though the mean value of the property under consideration may vary somewhat from location to location. Figure 24 shows the plot of the coefficient of variation versus the mean value of the compressive index determined by several researchers at different location in Bangkok. From Fig. 24 a value of 0.18 is obtained for the coefficient of variation using an averaging technique. These results are also summarized in Table 4.

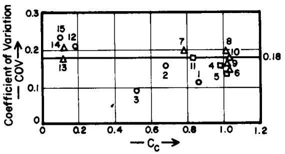


Fig. 24 - Coefficient of Variation of C_c

Table 4 - Coefficient of Variation of the Compressibility Index for Bangkok Clay (from past research)

Experimentalist	Site	C_c	V	Type
1. Teves	Bangkok	0.865	0.111	soft
2. Boripunt	Bangkok	0.680	0.189	soft
3. Boripunt	Bangkok	0.530	0.090	soft
4. Kim	Rangsit	0.988	0.163	soft
5. Kang	Rangsit	1.045	0.136	soft
6. Reinmanorom	Nong Ngoo Hao	1.121	0.133	soft
7. Reinmanorom	Nong Ngoo Hao	0.780	0.198	soft
8. Dang	Nong Ngoo Hao	1.076	0.195	soft
9. Dang	Nong Ngoo Hao	1.064	0.156	soft
10. Thumapradit	Nong Ngoo Hao	1.083	0.162	weathered
11. Kanjanaphos	Rangsit	0.844	0.176	weathered
12. Chuang	Bangkok	0.197	0.209	stiff
13. Towan	Nong Ngoo Hao	0.153	0.175	stiff
14. Dang	Nong Ngoo Hao	0.177	0.203	stiff
15. Kaweepoj	Bangkok	0.153	0.218	hard

In an attempt to study the distribution of the compression index, carefully selected data from about the same depth were analysed and Fig. 25 shows the empirical distribution so obtained for C_c at two different depths. The χ^2 goodness of fit test (at a 5% level of significance, see BENJAMIN & CORNELL, 1979) for these two distributions are not significant for the hypothesis that the distributions are normal.

Based on the normal probability distributions for C_c and the coefficient of variation, V, the probability distribution of the mean of the compression index $f(C_c)$, for the various layers at the chosen test site may be generated

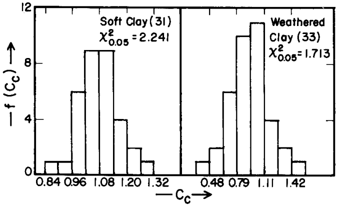


Fig. 25 - Histograms of C_c for Bangkok Clays

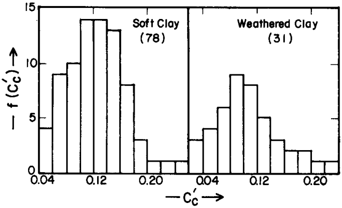


Fig. 26 - Histograms of C_c' for Bangkok Clays

from

$$C_c = \frac{\bar{C}_c}{1 - ZV/\sqrt{n}} \qquad \text{..... (7)}$$

where $\bar{C}_c$ is the mean compression index for the specific set of sample tested at this site for each layer. V is the coefficient of variation, n is the number of samples tested for each layer and Z is a standard normal deviate.

Unlike the compression index, C_c, the index C_c' in the overconsolidated range did not show any significant variation with depth, in a statistical analysis of the data. Thus two empirical distribution to represent the whole of weathered clay and soft clay were determined from available data. These distributions fail to be Gaussian by a χ^2 goodness of fit test (at 5% level). A positive skewness to the right is evident in Fig. 26.

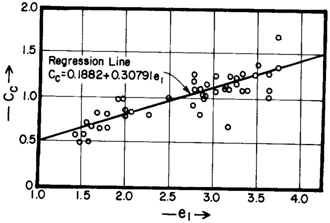


Fig. 27 - Relationships between C_c and e_1

Since the coefficient of variation approach was not valid for the parameter C_c' it was decided to use the empirical distribution itself for simulation purposes. Thus, a piece-wise linear probability distribution determined from the empirical distribution was incorporated in the simulation program.

Past researchers have found a significant correlation between e_o and C_c (ELNAGGER & KRIZEK, 1970) and have proposed many empirical relationships. Along a similar approach, regression analysis was performed between the voids ratio and the compression index C_c However, it is clear from Fig. 23 that there exists two sets of values in each case. The initial voids ratio (e_o) was regressed with the compression index (C_c'), while the voids ratio (e_1)^o corresponding to the maximum past pressure, was regressed with (C_c). This in turn gave linear regression models for the voids ratio in terms of the compressibility indices. Fig. 27 shows a typical scatter diagram obtained for the case of e_1 versus C_c in soft clay region with the estimated regression equation.

Details of Test Embankment

Two of the three full scale embankments given in Figs. 17 and 18 were analysed by various methods. Embankment I was 100 m long x 30 m wide and was built with dense sand up to a height of 3.4 m. The side slope being 1:2 as shown in Fig. 17. A berm of one metre width was provided on one side of the embankment to ensure that failure occurred on the other side which was fully instrumented. It was heavily instrumented as to get adequate information on pore pressures, settlements, lateral movements and mode of failure.

Embankment II in Fig. 18 is 300 m long and is of variable height. The height increase from 0.5 m at one end over a length of 200 m and then remains

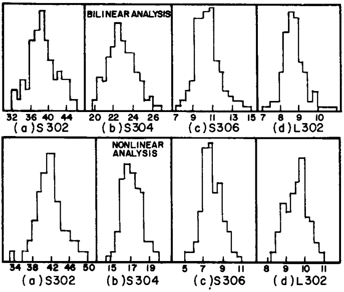


Fig. 28 - Histograms for Immediate Settlement and Lateral Movements

constant at 2.9 m height for the last 100 m length. The side slope in the variable height section is 1:1 and in the constant height section is 1:2. Embankment I was constructed as rapidly as possible until failure occurred. The fill material for both embankments was imported sand and the unit weight of the fill averaged to 1.8 t/m^2.

Settlement Analysis Using Fintie Element and Probabilistic Method

SIVANDRAN et al (1979) described the modification made in the standard finite element anaysis to accept simulated values of the elastic properties. The finite element analysis proceeds normally in the definition of data, the calculation of the finite element idealisation of the embankment, calculation of the initial stress distribution and the applicaton of the embankment load as nodal point forces. At this stage the stiffness metrix was assembled using the elastic properties simulated from the derived probability distribution. The analysis continued for the solution of displacements and finally the solution of stresses and strains in the elements. At this stage, the iteration was repeated to evaluate the elastic properties at the new stress levels using the simulation model. The cycle was continued for a number of times as in the classical finite element analysis. Sixty sets of runs were performed to determine the settlement distributions. Frequency distributions were constructed for four locations as shown in Fig. 28. These locations, S302, S304 and S306 refer to vertical settelments while location L302 refers to lateral movements away from the embankment. These locations are shown in Fig. 17. Measured data and results of the deterministic analysis at these

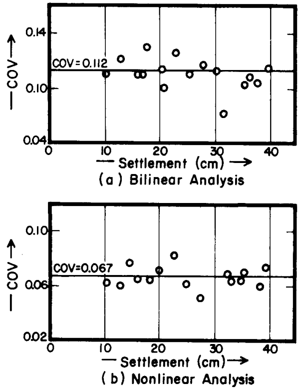


Fig. 29 - Coefficient of Variation of Calculated Settlements

locations are as follows.

	S302	S304	S306	L302
Measured (cm)	43.0	19.5	6.0	11.0
Deterministic				
Bi-linear	37.0	17.5	12.5	10.0
Non-linear	40.0	19.0	10.5	10.5
Probabilistic				
Bi-linear	38.2	23.4	10.7	8.7
Non-linear	42.1	17.9	7.8	9.6

The mean values from the frequency distributions compare well with the measured values.

The primary concern in this study was to determine the influence of uncertainty due to spatial variation on predicted settlements. This was by the coefficient of variation (COV), the ratio of the standard deviation and the mean. The COV was determined from the frequency distributions of settlement of several modes in the mesh. The COV of the bi-linear analysis was found to be 0.112, while that for the non-linear analysis was 0.067 (see Fig. 29). This can directly be attributed to the uncertainty introduced in the determination of moduli. The determination of the modulus from the probability distribution of the vane strength of the bi-linear anaysis involved a

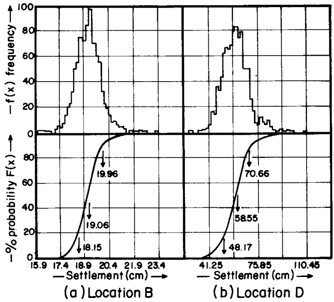


Fig. 30 - Frequency and Probability Distribution of Predicted Settlement

much larger scatter than the modulus determined from stress-strain relationships for the non-linear analysis.

Consolidation Settlement Using Probability Analysis

In this section, the probability analysis of the primary consolidation settlement under Embankment II is presented and discussed. The point D in Fig. 18 is chosen at a location where the height of the sandfill is 2.9 m while the second point B is located on the sloping section of the embankmennt at a location where the height of sand fill is 1.3 m.

By means of random number generations on a digital computer, it is possible to sample from any specified distribution for the parameters involved in consolidation settlement evaluation. The distributions of the predicted settlements are shown in Fig. 30 for both the locations B and D. Since the analysis was conducted for each layer separately it was possible to predict the settlements at various depths. The 10%, 50% and 90% settlement values down the centre line at B and D are shown in Fig. 31. A comparison of the predicted settlements with those evaluated using other methods is given below.

Method of Analysis	B(cm)	D(cm)
Consolidation Settlement by Stress Path Method	17.90	56.95
Elastic Method using Drained Modulus from Triaxial Tests	16.29	49.59
Critical State Theory (Modified)	-	58.88
Measured Values	8.20	7.50
Probability Analysis	19.06	58.50

The effect of scatter and dispersion of the soil parameters C_c, and p_c on the resulting distribution of settlement was also studied. Figure 32 shows the effect of variation of C_c for the hypothetical case chosen with mean values for $C_c = 1.1$, $p_o = 0.95$ t/m^2, $\Delta p = 1.0$ t/m^2, $p_c = 0.75$ t/m^2, when the settlement involved is in the normally consolidated range. Values chosen for the coefficient of variation are 0.05, 0.15 and 0.25 . Figure 32 seems to reveal that the mean value (50%) of the settlement is barely affected by the change in the coefficient of variation in C_c. Both the 10% and 90% values are changed. Two probability statements can now be made related to Fig. 32.

(i) 80% probability range for settlement ratio for the different cases are:

COV	Range	% increase
0.05	0.210 - 0.183 = 0.027	0.0
0.15	0.251 - 0.182 = 0.033	22.2
0.25	0.234 - 0.178 = 0.056	107.4

(ii) The probability that the settlement ratio may be greater than 0.22 (say) is

COV	Probability
0.05	2.0 %
0.15	6.5 %
0.25	20.0 %

These examples illustrate how the uncertainty in the soil parameters is transferred to the reliability of the predicted settlement.

CORRELATION OF SAFETY FACTORS WITH SIMPLE MEASUREABLE PARAMETERS

Large scale field tests are often expensive and are virtually impossible to be incorporated in most of the projects. It is thus important that the data obtained from such sophisticated projects be correlated with simple soil parameters which can be measured in most laboratory. In that sense, the correlations proposed by BJERRUM (1973) are most valuable for all practicing engineers. From a detailed study made on the failure of several test embankments on soft clays, BJERRUM (1973) proposed a linear correlation between the safety factor and the plasticity index of the clays (see Fig. 33). A similar correlation was also presented by BJERRUM (1973) between safety factors for shallow footings and the plasticity index of clays (see Fig. 34). In Fig. 35, a correlation is presented for safety factors of cuts and unsupported excavations with the plasticity index of soft clays. This state-of-the-art reporter is of the view that such correlations are of extreme importance for all soil engineers who are engaged in geotechnical engineering activities with soft clays.

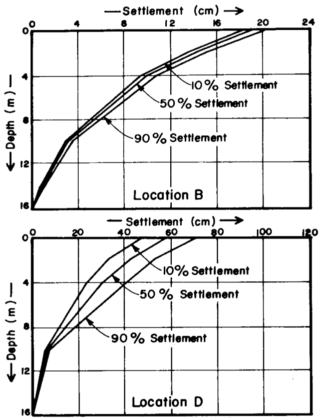


Fig. 31 - Predicted Settlements with Depth

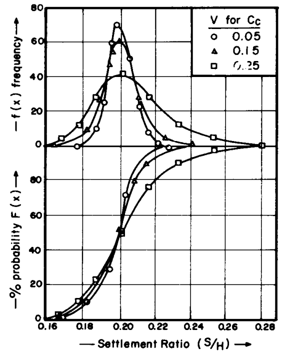


Fig. 32 - Effect of Dispersion in C_c on Predicted Settlement Distribution

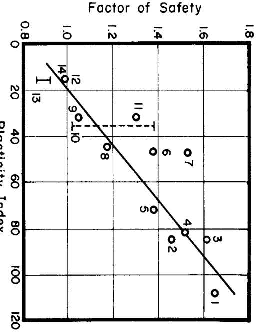


Fig. 33 - Correlation of Factor of Safety with Plasticity Index for Embankments (After BJERRUM, 1973)

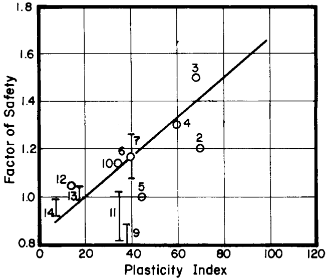


Fig. 34 - Correlation of Factor of Safety with Plasticity Index for Shallow Footings (After BJERRUM, 1973)

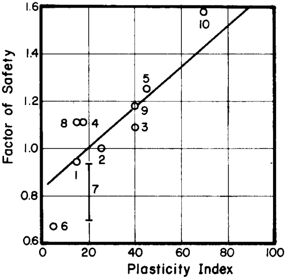


Fig. 35 - Correlation of Factor of Safety with Plasticity Index for Cuts and Slopes (After BJERRUM, 1973)

CONCLUSIONS

In the state-of-the-art report titled in-situ tests and instrumentation of soft clay deposits the author has selected certain salient aspects which he considers rather important from his experience in the last fifteen years. In-situ tests are broadly divided into two types; small scale tests and large scale tests. The usefulness of vane tests in site investigation and design aspects of embankments, excavations and piled foundations is described. Dutch cone tests are also recommended to be used in the design of piled foundations. In the category of large scale tests, full scale test embankments, test excavations and instrumented foundations are discussed. The instrumentation aspects of these large scale tests are discussed comprehensively by DIBIAGIO & MYRVOLL (1977). The author emphasizes the necessity to analyse fully the data obtained from full scale tests with soil parameters derived from simple laboratory and field tests. The role of statistics and probability theories in the analysis and interpretation of the data from all types of tests is of great importance in quantifying the scatter in the test data and in the uncertainties involved in analysis and design.

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